Earthquake Response of Concrete Gravity Dams

Deepti Wagle

A thesis submitted in total fulfillment of the degree of
Master of Engineering (by Research)

June 2010

Faculty of Engineering and Industrial Sciences
Swinburne University of Technology
Abstract

Nepal is set to construct a number of large dams to cater for its ever increasing power demand. One of the big dams in the pipeline is Arun III dam in the Sankhuwasabha district in eastern Nepal. The Arun III dam was designed in the mid 1980s and little is known about its capability to resist major earthquakes which are probable in this region. The Himalayan region is one of the most earthquake prone zones in the world but the seismic hazard level of the region is not well established. This study was undertaken with a purpose to investigate the seismic performance of the Arun III dam under site specific design ground motions.

In the first part of the research, a seismic hazard estimate was carried out for the eastern Nepal Himalayas where the Arun III dam is proposed. For the hazard study, two approaches were used:

- Probabilistic approach using Atkinson and Boore model of attenuation and
- Component Attenuation Model (CAM) methodology

The established seismic hazard was used to select design ground motions for the site. Four real earthquake accelerograms and one synthetic record were selected. While selecting the ground motions, special attention was paid to the scaling of the records to match the characteristics of the design level event.

The seismic performance of the dam was evaluated following the US Army Corps of Engineers guidelines, using both static and dynamic analysis approaches. Initially, the finite element model of the dam was developed and the seismic analysis was performed in a step by step manner starting from the preliminary level i.e. static analysis. The study the model was analysed using linear elastic dynamic analysis procedures – Response Spectrum Modal Analysis procedure and Time History Analysis procedure. The records selected from the seismic hazard study were applied as seismic loads to the finite element model of the dam structure in the dynamic analysis methodology. Then the results were analysed with the acceptance criteria established from the previous researches on gravity dams. In addition, a non-linear analysis was undertaken to evaluate the degree of cracking and damage to the dam structure under extreme earthquake excitation. Based on the analyses it was concluded that the Arun III dam is safe against collapse from the design level events but damage is probable primarily due to cracking relating to the localized high tensile stresses in the concrete.
**Preface**

During the course of this research a refereed conference paper to an international conference was written which has been accepted for publication. The details listed below:

Declaration

This is to certify that this thesis comprises:

- no material which has been accepted for the award to the candidate of any other degree, except where due reference is made in the text,
- solely of my original work and due acknowledgement has been made wherever other previously published material and references are used.
- less than 50,000 words in length, exclusive of tables, maps, charts and bibliographical references.

______________________________

Deepti Wagle

June 2010

(Amended September 2010)
Acknowledgements

I wish to express my profound gratitude to my supervisors Prof. John L. Wilson and Dr. Kamiran Abdouka for their untiring support, invaluable guidance and suggestions that enabled me to successfully complete my degree. The study was carried out in Faculty of Engineering and Industrial Sciences, Swinburne University of Technology, Melbourne, Australia. I am very grateful to the Faculty and the University for providing me with the scholarship to carry out my studies.

I thank my valued colleague Siva Sivanerupan and other researchers in the Faculty for their suggestions, support and encouragement. I also thank all the members of the Centre for Sustainable Infrastructure (CSI) for their encouragement and support. The academic environment and friendliness among fellow researchers played a major role in my productivity and eagerness to study. Also I would like to thank senior seismologist Gary Gibson for his extremely valuable suggestions.

I am deeply indebted to my family in Nepal for continuously encouraging me from thousands of kilometers away to stay engaged in my research. I extend deep gratitude to my father Baburam Bharadwaj who not only encouraged me but also provided me with the financial support for my stay in Melbourne. His help in obtaining the design documents of the Arun III project from the Library of the Department of Electricity Development, Ministry of Energy (Government of Nepal) enabled me to carry out my research successfully.
## Contents

Abstract ................................................................. i  
Preface ................................................................. ii  
Declaration ............................................................. iii  
Acknowledgements .................................................. iv  
Contents ............................................................... v  
List of Figures .......................................................... vii  
List of Tables ............................................................ x  
Abbreviations .......................................................... xii

1 INTRODUCTION .............................................................. 1  
  1.1 BACKGROUND .............................................................. 1  
  1.2 OBJECTIVES ................................................................. 1  
  1.3 OUTLINE ................................................................. 2  

2 HIMALAYAN SEISMICITY AND ARUN III DAM ................. 3  
  2.1 GEO-TECTONIC SETTING OF THE HIMALAYAS ..................... 3  
  2.1.1 Geological Framework .............................................. 5  
  2.1.2 Physiography ........................................................... 9  
  2.2 HISTORICAL SEISMICITY IN NEPAL ................................. 11  
  2.3 HYDROPOWER AND DAMS IN NEPAL ................................. 12  
  2.4 ARUN HYDROELECTRIC DEVELOPMENT PLANS ...................... 15  
  2.5 ARUN III HYDROELECTRIC PROJECT ................................. 16  
  2.5.1 Topography ............................................................. 16  
  2.5.2 Geology ................................................................. 18  
  2.5.3 Structural Details of the Dam and the Spillway ................. 20  
  2.5.4 Seismology ............................................................ 20  
  2.6 CONCLUSION ........................................................... 21

3 DAMS: CHARACTERISTICS AND METHODOLOGIES FOR PERFORMANCE ASSESSMENT .... 22  
  3.1 INTRODUCTION .............................................................. 22  
  3.2 TYPES OF DAMS ......................................................... 22  
  3.3 FORCES ACTING ON A CONCRETE GRAVITY DAM .......... 23  
  3.4 SEISMIC RISKS TO DAMS ............................................... 27  
  3.5 EARTHQUAKE RESPONSE OF GRAVITY DAMS .................. 30  
  3.5.1 Response to internal force or deformation controlled actions ........................................... 30  
  3.5.2 Response to stability controlled rigid body actions ....................................................... 31  
  3.6 DESIGN CRITERIA (USACE 2007) .................................... 32  
  3.7 PERFORMANCE EVALUATION .......................................... 35  
  3.8 PERFORMANCE ASSESSMENT METHODS ............................. 35  
  3.8.1 Numerical/Analytical methods .................................... 35  
  3.8.2 Physical modelling ................................................... 40  
  3.9 PREVIOUS RESEARCHES ................................................ 40  
  3.9.1 Analytical studies ...................................................... 41  
  3.9.2 Experimental .......................................................... 44  
  3.10 CONCLUSION ........................................................ 45
HAZARD ASSESSMENT AND REPRESENTATIVE DESIGN EARTHQUAKE GROUND MOTIONS

4.1 INTRODUCTION ........................................................................................................... 46
4.2 SEISMIC HAZARD ASSESSMENT ............................................................................... 46
4.2.1 Probabilistic Seismic Hazard Analysis (PSHA) ....................................................... 46
4.2.2 Methodology using the Component Attenuation Model (CAM) ............................... 57
4.3 SEISMIC HAZARD ANALYSIS FOR ARUN RIVER VALLEY, EASTERN NEPAL .......... 60
4.3.1 Design Response spectra .................................................................................... 60
4.3.2 Magnitude frequency relationship ..................................................................... 62
4.3.3 Return periods for design events ....................................................................... 62
4.3.4 M-R combinations ............................................................................................. 63
4.4 COMPARISON WITH PREVIOUS STUDIES ................................................................ 63
4.5 THE MAXIMUM CREDIBLE EARTHQUAKE (MCE) EVENT ..................................... 64
4.6 DISCUSSION AND CONCLUSION ........................................................................... 65

5 SELECTION OF REPRESENTATIVE GROUND MOTIONS ............................................. 67
5.1 MAXIMUM CREDIBLE EARTHQUAKE AND MAXIMUM DESIGN EARTHQUAKE .... 67
5.2 SELECTION OF GROUND MOTION TIME HISTORIES ............................................ 68
5.2.1 Use of recorded and synthetic time-histories ....................................................... 68
5.2.2 Criteria for selection .......................................................................................... 68
5.3 SELECTION AND SCALING APPROACH ................................................................. 69
5.3.1 Number of time-histories .................................................................................. 69
5.3.2 Scaling approach .............................................................................................. 69
5.4 SELECTED GROUND MOTIONS ............................................................................... 70
5.4.1 Design earthquake from seismic hazard analysis ................................................. 70
5.4.2 Tectonic environment and site condition ............................................................ 70
5.4.3 Selected records and scaling ............................................................................ 70
5.4.4 Response Spectra of selected records ................................................................. 72
5.5 CONCLUSION ......................................................................................................... 75

6 FINITE ELEMENT MODELLING AND PRELIMINARY SEISMIC ANALYSIS ..................... 76
6.1 INTRODUCTION ....................................................................................................... 76
6.2 NUMERICAL MODELLING ..................................................................................... 76
6.2.1 Expected Structural Behaviour ........................................................................ 76
6.2.2 Development of the Finite Element Model ......................................................... 77
6.2.3 Assumptions ..................................................................................................... 78
6.3 PRELIMINARY ANALYSIS .................................................................................... 80
6.3.1 Model validation and sensitivity analysis with mesh density .............................. 80
6.3.2 Sensitivity with foundation rigidity .................................................................. 83
6.3.3 Linear Static Analysis: Load cases .................................................................... 87
6.4 RESULTS AND DISCUSSION ................................................................................ 87
6.5 CONCLUSION ....................................................................................................... 91

7 DYNAMIC ANALYSIS OF ARUN III DAM ..................................................................... 92
7.1 INTRODUCTION ..................................................................................................... 92
7.2 RESPONSE SPECTRUM ANALYSIS ....................................................................... 92
7.2.1 Assumptions .................................................................................................... 92
7.2.2 Computation of earthquake response ................................................................. 93
7.3 LINEAR TIME HISTORY ANALYSIS ...................................................................... 103
7.3.1 Acceptance Criteria ......................................................................................... 103
7.3.2 Assumptions ................................................................................................... 104
List of Figures

Figure 1 Indian land mass moving towards the Eurasian landmass (Dahal R. K. 2006) .................................................. 3

Figure 2 Collision of the Indian plate with Tibetan plate and formation of the Himalayas (modified after USGS, 1999) .............................................................................................................................................................................. 4

Figure 3 Transverse division of the Himalayas (Gansser A. 1964) ......................................................................................................................... 5

Figure 4 Geological Map of Nepal modified from (Dahal R. K. 2006) ..................................................................................................... 6

Figure 5 Generalized cross section of the Himalayas, after (Dahal R. K. 2006) .................................................................................. 6

Figure 6 Aerial photograph of Udayapur district, the Main Boundary Thrust (MBT) can be observed passing through the middle of the photograph (Dahal R. K. 2006) ........................................................................................................... 8

Figure 7 MBT observed in Butwal-Tansen section of Siddartha Highway (Dahal R. K. 2006) ........................................................ 8

Figure 8 South Tibetan Detachment System (STDS) as seen in Gurudongmar Lake, Sikkim .............................................................. 9

Figure 9 Major historical earthquakes of Nepal (Google Earth image Viewed on 10/10/2008) .......................................................... 12

Figure 10 Seismic Hazard Map of Nepal based on GSHAP data (Amateur Seismic Centre Pune 2006) .................. 12


Figure 12 Trans-Himalayan stretch of the Arun river entering Nepal from Tibet looking South (Google Earth image Viewed on 10/10/2008) .................................................................................................................. 17

Figure 13 Schematic Geological Map of the Arun (Nepal Electricity Authority N. 1986) ................................................................. 19

Figure 14 Forces acting on a gravity dam (Directions shown are not necessarily critical) ....................................................... 24

Figure 15 Uplift Pressure Calculation (IS: 6512-1984 Bureau of Indian Standards) ................................................................. 26

Figure 16 Special cases uplift pressure calculation (IS: 6512-1984 Bureau of Indian Standards) ....................................................... 26

Figure 17 Gravity dam Subjected to earthquake Ground motions (USACE 2007) ...................................................................................... 30

Figure 18 Stress -Strain Relationship for Plain Concrete Structures (USACE 2007) ................................................................. 31

Figure 19 Dam Permanent Sliding Displacements (USACE 2007) ................................................................................................. 31

Figure 20 Equivalent Lateral Force Method- Step 1 (USACE 2007) ................................................................................................. 37

Figure 21 Equivalent Lateral Force Method- Steps 2 and 3 (USACE 2007) ...................................................................................... 37

Figure 22 Equivalent Lateral Force Method – Step 4 (USACE 2007) ................................................................................................. 38

Figure 23 Capacity Spectrum Diagram (Freeman S. 2004) ................................................................................................. 40

Figure 24 Variables in distance probability (USACE 1999) ................................................................................................. 49

Figure 25 Development of Equal hazard response spectrum from PSHA as per EM 1110-2-6050 (USACE 1999) .... 50

Figure 26 Interpreted active faults of Arun River valley and Eastern Nepal (Ministry of Physical Planning and Works G. o. N. 2001) ........................................................................................................................................................................ 51

Figure 27 Magnitude-Frequency relationship ................................................................................................................................. 53
Figure 28 Uniform Hazard Response Spectra .................................................................55
Figure 29 Uniform Hazard Response Spectra: ADRS format ........................................55
Figure 30 Disaggregation for PGA for 450 YRP event for Arun River valley ..................56
Figure 31 Disaggregation for PGA for 950 YRP event for Arun River valley ..................57
Figure 32 Response spectrum in ADRS format ............................................................59
Figure 33 ADRS diagram for Magnitude 5.5 across different source-to-site distances ..........61
Figure 34 ADRS diagram for Magnitude 6 across different source-to-site distances ..........61
Figure 35 ADRS diagram for Magnitude 6.5 across different source-to-site distances ..........61
Figure 36 ADRS diagram for Magnitude 7 across different source-to-site distances ..........62
Figure 37 ADRS diagram for Magnitude 7.5 across different source-to-site distances ..........62
Figure 38 Uniform hazard response spectrum for maximum design event (950 YRP) ........68
Figure 39 Selected records as representative design events (SeismoSoft Ltd 2010) .............72
Figure 40 Comparison of the scaled spectra with the design/target spectrum ....................74
Figure 41 Mean of the selected records' spectra with target design spectrum ....................74
Figure 42 Scaled and unscaled records of Landers earthquake in the form of response spectra 75
Figure 43 Arun III dam finite element model .............................................................77
Figure 44 Cross section of the nonoverflow monolith of Arun III dam ............................78
Figure 45 Horizontal plane under consideration and the loads associated .......................81
Figure 46 Comparison of the FEA and Manual calculation results .................................83
Figure 47 Extent of the foundation for FEA ...............................................................84
Figure 48 Kinks on the dam profile .............................................................................85
Figure 49 Normal stresses of the nodes along the dam-foundation interface .......................86
Figure 50 Sensitivity of the crest displacement with the elastic modulus of foundation material 86
Figure 51 Sensitivity of the peak normal stress with the elastic modulus of foundation material 87
Figure 52 Normal stress contour plots for different load cases ........................................89
Figure 53 Major Principal Stress contour plot for static analysis (G+P+E) .........................90
Figure 54 Nodes exceeding the tensile strength value of concrete (G+P+E) .......................90
Figure 55 Ten Mode Shapes for Arun III Dam ............................................................94
Figure 56 Location of critical nodes in the structure model ............................................95
Figure 57 Response Spectrum Modal Analysis results: Uttarkashi ................................97
Figure 58 Response Spectrum Modal Analysis results: Manjil ......................................98
List of Tables

Table 1 Transverse Division of the Himalayas ................................................................................................................................. 5
Table 2 Physiographical division of the Nepal Himalayas and corresponding Geological zones (Upreti 1999) ................. 10
Table 3 Notable Historical Seismic Events (National Society for Earthquake Technology Nepal - NSET 2009) .................. 11
Table 4 Inventory of Hydropower Projects and Dams in Nepal ........................................................................................................ 14
Table 5 Dam location coordinates .................................................................................................................................................... 17
Table 6 List of large dam projects with reservoir triggered seismicity (USCOLD Committee on Earthquakes 1997) ........... 28
Table 7 Seismic sources around Arun III dam site .......................................................................................................................... 51
Table 8 Earthquake events for Magnitude-Frequency study ......................................................................................................... 52
Table 9 Design events from disaggregation results ....................................................................................................................... 57
Table 10 M-R combinations and response spectral parameters for OBE .................................................................................. 63
Table 11 M-R combinations and response spectral parameters for MDE ............................................................................. 63
Table 12 PGA values comparison for selected return period events ............................................................................................ 64
Table 13 Design earthquake events for the Arun river valley ........................................................................................................ 66
Table 14 Details of the selected representative ground motions .................................................................................................. 71
Table 15 Loads acting on the plane .................................................................................................................................................... 82
Table 16 Moment relative to the centroid of the section ................................................................................................................. 82
Table 17 Comparison of FEA results with manual calculation .................................................................................................. 83
Table 18 Normal stresses on kinks with and without foundation material ..................................................................................... 85
Table 19 Peak Normal (SY) tensile stresses .................................................................................................................................... 88
Table 20 Modes of vibration ................................................................................................................................................................. 93
Table 21 Response maxima for different response spectra inputs (-ve Stresses - compression) .................................................. 95
Table 22 Response maxima for different time history seismic inputs (-ve Stresses - compression) .......................................... 107
Table 23 Spring elements states after nonlinear analysis ............................................................................................................. 114
Abbreviations

ADR S Acceleration Displacement Response Spectrum
BSI British Standards Institution
CAM Component Attenuation Model
CSM Capacity Spectrum Method
DCR Demand Capacity Ratio
ELF Equivalent Lateral Force
EM Engineer Manual
EPGA Equivalent Peak Ground Acceleration
ER Engineer Regulation
FEA Finite Element Analysis
FEM Finite Element Method
GoN Government of Nepal
GSHAP Global Seismic Hazard Assessment Program
HEP Hydro Electric Project
HFT Himalayan Frontal Thrust
HPP Hydro Power Project
HREP Hydro and Rural Electrification Project
ICOLD International Commission On Large Dams
INPS Integrated Nepal Power System
IS Indian Standard
ITSZ Indus Tsangpo Suture Zone
JICA Japan International Cooperation Agency
LDP Linear Dynamic Procedure
LSP Linear Static Procedure
MBT Main Boundary Thrust
MCE Maximum Credible Earthquake
MCT Main Central Thrust
MDE Maximum Design Earthquake
MFT Main Frontal Thrust
MoPPW Ministry of Physical Planning and Works
MW Megawatt
M_w Moment Magnitude
NDP Nonlinear Dynamic Procedure
NEA Nepal Electricity Authority
NSC National Seismological Centre
NSET National Society for Earthquake Technology
NSP Nonlinear Static Procedure
OBE Operational Basis Earthquake
PGA Peak Ground Acceleration
PSHA Probabilistic Seismic Hazard Estimation
RSA Response Spectral Acceleration
RSD Response Spectral Displacement
RSMA Response Spectrum Modal Analysis
RSV Response Spectral Velocity
STDS South Tibetan Detachment System
UHRS Uniform Hazard Response Spectrum
USACE United States Army Corps of Engineers
Chapter 1

Introduction

1.1 Background

Nepal is located at the boundary between the Indian and the Tibetan tectonic plates and therefore lies in a seismically active region. From historical data (National Seismological Centre Nepal) it is evident that great destructive earthquakes occurred in the past. According to the Plate Tectonics theory the Indian and the Eurasian (Tibetan) plates are converging towards each other with the Indian plate being plunged under a portion of the Eurasian plate. As an effect of the subduction, the portion of the Eurasian plate converging is raised and thus the Himalayas and the Tibetan plateau are formed. The Himalayas are believed to be in a continuous stage of formation as the area experiences frequent earthquakes.

With high rainfall and steep valleys, Nepal has great potential for hydropower, but is geographically located in one of the tectonically active regions of the world. In recent times, despite its vast hydro resources Nepal is crippled with power shortage due to increasing demand. Since hydro resource is the most abundant in Nepal it has to be utilised the most. Therefore, promotion of hydropower projects in Nepal is inevitable. A large number of massive hydropower projects are planned in Nepal for the coming decade. Hydro projects of massive size usually come with dams of huge dimensions which become a matter of concern for the severe consequences in case of a major earthquake.

The potential risk from high magnitude earthquakes on human lives and structures is a well known challenge to mankind. Mitigation of earthquake risk can be possible only with adequate assessment of seismic hazard which should be based on the evaluation of seismotectonic and geological processes. In this context it would be significant to quote the following from (Chopra A. K. 1967) and (Finn L. 1991). “The potential hazard of an earthquake inducing some of the quoted consequences depends on: the seismicity of the ground in the zone in which the dam is sited, the geological structure and the topographic conditions in the dam site, and the type, construction and size of the dam”. It is therefore of paramount importance that the upcoming hydropower projects in Nepal be designed to account for the effects of the appropriate level of earthquake actions.

1.2 Objectives

The main objective of this study is to undertake the following scope of works:

- Performance study of concrete gravity dams under earthquake ground motion.
- Establish the seismic hazard and find the representative design ground motions for Eastern Nepal.
Perform a case study on Arun III HEP (Hydro Electric Project) dam to determine its capacity and performance to a set of design earthquake ground motions.

1.3 Outline

The report documents a comprehensive seismic study of the Arun III dam. Arun III is a Pondage Run-of-River project, which is planned to be constructed in the Sankhuwasabha district of the Koshi Zone in eastern Nepal. The research focuses in the evaluation of the seismic response of the non-overflow monolith of the dam structure when subjected to a set of site specific design ground motions. This report is divided into three parts: (a) Introduction of dams and their seismic analytical aspects, review of Himalayan seismicity and introduction to Arun III dam (Chapters 1, 2 and 3), (b) Establishing the seismic hazard level for the region and preparing a set of design ground motions (Chapters 4 and 5) and (c) Evaluation of the response of the structure when subjected to the set of design ground motions including proposed recommendations and conclusions of the study (Chapters 6, 7 and 8).

The introductory Chapter 1 mentions the importance and objectives of the study. In Chapter 2, seismicity in the Himalayan region is discussed. In addition, the topographical, geological, structural and seismological aspects of the Arun III dam are presented. Chapter 3 reviews the methodologies for the performance assessment of dams. Also, the previous research works and literature are summarized. Chapter 4 presents the seismic hazard analysis conducted for establishing the hazard level in the eastern Nepal Himalayas. From the hazard estimate, a suit of design ground motions are prepared through selection and scaling in Chapter 5.

In Chapter 6, the finite element model of the non-overflow monolith of the dam is developed. In addition, preliminary seismic analysis is carried out including sensitivity studies in relation to the mesh density of the model and the foundation material properties. Chapter 7 presents the detailed dynamic analysis of the model with both response spectrum and time history methods.

Finally, the notable conclusions drawn from the study are presented in Chapter 8, along with recommendations.
Chapter 2

Himalayan seismicity and Arun III Dam

2.1 Geo-Tectonic Setting of the Himalayas

The Himalayan Range is the youngest mountain system in the world. It is a broad continuous arc along the northern fringes of the Indian subcontinent, from the bend of the Indus River in the northwest to the Brahmaputra River in the east. Between these bends the Himalayan range is approximately 2400 km long and 200 km to 300 km wide. The Himalayas cover an area of approximately 600,000 sq. km in south Asia.

According to the plate tectonic theory which is widely accepted around the world, the Himalayas were formed by the collision of the Indian Plate with Tibetan (Eurasian) Plate around 55 millions years ago (Klous W. J. 1996). A diagrammatic illustration is shown in Figure 1. Many scientists believe that at that time the northward moving Indian plate first touched the southern edge of the Tibetan (Eurasian) plate.

![Figure 1 Indian land mass moving towards the Eurasian landmass (Dahal R. K. 2006)](image)

Figure 1 Indian land mass moving towards the Eurasian landmass (Dahal R. K. 2006)
A continental plate colliding with another continental plate is a special case of convergent plate boundary. Since both the continental crusts are high in density and buoyant, subduction does not occur like in oceanic to continental crust collisions. Instead, the collision forms mountain ranges as the crust thickens and uplifts. The Himalayas were formed as the result of mountain building (orogenic) process which continues from the collision and the mountain is believed to be still on the process of its making. This is noticeable by present day northward movement of India at the rate of 2 cm per year and the occurrences of frequent seismic shakes all along the Himalaya and its surroundings (Jackson and Bilham 1994), (Pandey et al 1995), (Bilham et al 2001). Most part of the drift is accommodated within the Himalaya by various thrusts as well as rising peaks. The Himalayas and the Tibetan Plateau to the north have risen very rapidly. In just 50 million years, peaks such as Mt. Everest have risen to heights of more than 9 km. However, scientists believe that the Eurasian Plate may now be stretching out rather than thrusting up, and such stretching would result in some subsidence due to gravity (USGS 2010).

Significant research on the Himalayas commenced in the 1970s when the plate tectonics theory became popular among scientists and engineers. Eventually, the Himalayas became an ingenious natural laboratory to the researchers for assessing their hypotheses on earth surface and sub-surface dynamics. According to (Dahal R. K. 2006), the Himalayas are one of the rare mountain ranges on the earth where, in a single 50km long traverse, five major tectonic zones can be observed. Deep seated metamorphic rock sequences to the fossiliferous sedimentary rock make up these zones.

The first comprehensive picture of the Himalayas was provided by (Gansser A. 1964) dividing the whole Himalayan range transversely into the following five major groups (Refer Table 1 and Figure 3). A brief description of the same is presented in Table 1.
Figure 3 Transverse division of the Himalayas (Gansser A. 1964)

Table 1 Transverse Division of the Himalayas

<table>
<thead>
<tr>
<th>S.N.</th>
<th>Himalayan Division</th>
<th>Boundary West</th>
<th>Boundary East</th>
<th>Length</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Punjab Himalayas</td>
<td>Indus River</td>
<td>Sutlej River</td>
<td>550 km</td>
</tr>
<tr>
<td>2</td>
<td>Kumaon Himalayas</td>
<td>Sutlej River</td>
<td>Mahakali River</td>
<td>820 km</td>
</tr>
<tr>
<td>3</td>
<td>Nepal Himalayas</td>
<td>Mahakali River</td>
<td>Sikkim (Tista River)</td>
<td>800 km</td>
</tr>
<tr>
<td>4</td>
<td>Sikkim-Bhutan Himalayas</td>
<td>Sikkim (Tista River)</td>
<td>Bhutan eastern border</td>
<td>400 km</td>
</tr>
<tr>
<td>5</td>
<td>NEFA Himalayas</td>
<td>Bhutan eastern border</td>
<td>Tsangpo River</td>
<td>440 km</td>
</tr>
</tbody>
</table>

2.1.1 Geological Framework

Nepal occupies the central sector of the Himalayan arc. Nearly one third of the 2400 km long Himalayan range lies within Nepal. The Himalayas can be subdivided into the following five major tectonic zones (Dahal R. K. 2006).

- Gangetic Plain (Terai Zone)
- Sub-Himalayan (Siwalik) Zone
- Lesser Himalayan Zone
- Higher Himalayan Zone
- Tibetan-Tethys Himalayan Zone

Each of these zones is characterized by their own lithology, tectonics, structures and geological history. A generalized geological map is given in Figure 4.
These tectonic zones are separated from each other by thrust faults (See Figure 5).

- The southernmost fault, the Main Frontal Thrust (MFT) separates the Sub-Himalayan (Siwalik) Zone from Gangetic Plains.
- The Main Boundary Thrust (MBT) separates the Lesser Himalayan Zone from Siwalik.
- The Main Central Thrust (MCT) separates the Higher Himalayan Zone from the Lesser Himalayan Zone.
- The South Tibetan Detachment System (STDS) marks the boundary between the Higher Himalayan Zone and the overlying fossiliferous sequence of the Tibetan-Tethys Himalayan Zone.
- The Indus-Tsangpo Suture Zone is the contact fault between Indian plate and Tibetan (Eurasian) Plate in terms of plate tectonics.
The main geological zones of the Nepal Himalaya are described below.

**Gangetic Plain (Terai Zone)**

The Gangetic Plain (also called the Terai Zone which is the Nepalese segment of the Gangetic Plain) extends from the Indian Shield in the South to the Sub-Himalayan (Siwalik) Zone to the North. The plain is less than 200 meters above sea level and usually has thick (nearly 1500 m) alluvial sediments containing mainly boulder, gravel, silt and clay. The width of Terai Zone varies from 10 km to 50 km and forms a fairly continuous belt from east to west. To the north, this zone is separated from the Siwalik by an active thrust system called the Main Frontal Thrust (MFT).

**Sub-Himalayan (Siwalik) Zone**

The Sub-Himalaya Zone also called the Siwalik Zone is delimited on the south by the Main Frontal Thrust (MFT) and on the north by the Main Boundary Thrust (MBT). It consists basically of fluvial deposits of the Neogene age (23 millions years to 1.6 millions years old). This Zone extends all along the Himalaya forming the southernmost hill range with width of 8 to 50 km. The Siwalik Zone has a number of east-west running thrusts and is also rich with fossils of plants, pisces, reptiles and mammals.

**Lesser Himalayan Zone**

The Lesser Himalayan Zone is bounded to the north by the Main Central Thrust (MCT) and to the south by Main Boundary Thrust (MBT). MBT can be traced out in the whole Nepal Himalayas and it can also be well observed in aerial photographs (Figure 6 and Figure 7). The rocks of the Lesser Himalayan Zone have been transported southwards in several thrust slices. The Lesser Himalayas mainly have unfossiliferous, sedimentary, and meta-sedimentary rocks such as slate, phyllite, schist, quartzite, limestone, dolomite, etc, ranging in age from Precambrian to Eocene period. Some granitic intrusions are also found in this zone.
The Higher Himalayan Zone

The Higher Himalayan zone mainly consists of a huge pile of strongly metamorphosed rocks. Geologically, the Higher Himalayan Zone includes the rocks lying north of the Main Central Thrust (MCT) and below the highly fossiliferous Tibetan-Tethys Zone. This zone is separated from the Tibetan-Tethys Zone by normal fault system called the South Tibetan Detachment System (STDS). Higher Himalayan Zone consists of an approximately 10 km thick succession of strongly metamorphosed coarse grained rocks. It extends continuously along the entire length of the country as in the whole length of the Himalayas, and its width varies from place to place.

The Tibetan-Tethys Zone

The Tibetan-Tethys Zone lies in northern part of the country. It begins from the top of the STDS (Figure 8) and extends to the north in Tibet. The Tibetan-Tethys Zone is well-developed in Mustang, Manang and Dolpa area. In the eastern part, the amount of exposure of the Tibetan
Tethys Zone is almost negligible and found only in the upper heights of Mount Everest (Figure 4). Most of the other Great Himalayan peaks of Nepal such as Manaslu, Annapurna, and Dhaulagiri have rocks of Tibetan-Tethys Zone. This zone is composed of sedimentary rocks, such as shale, limestone, and sandstone, ranging in age from Cambrian to Eocene. This zone in some area is found as continuous deposits of Higher Himalayan Zone without normal fault.

2.1.2 Physiography

Physiographic division of Nepal has been in practice since 1950s. (Hagen T. 1969) divided Nepal into eight well defined physiographic units from south to north. These units are running east to west and can also be incorporated into the Indian Himalayan belt. The Hagen classification is the most appropriate classification and represents all characteristic physiographic zones of Nepal. A five fold classification is also in use (with the zones named Terai, Churia, Middle Mountain, High Mountain and High Himalayas). However, detail physiographical units of Nepal are as given in Table 2.
<table>
<thead>
<tr>
<th>SN</th>
<th>Geological Zones</th>
<th>Physiographic Units</th>
<th>Width (km)</th>
<th>Altitudes (m)</th>
<th>Main Rock Types</th>
<th>Main processes for landform development</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Gangetic Plane</td>
<td>Terai (Northern edge of the Gangetic Plane)</td>
<td>20-50</td>
<td>100-200</td>
<td>Alluvium: coarse gravels in the north near the foot of the mountains, gradually becoming finer southward</td>
<td>River deposition, erosion and tectonic upliftment</td>
</tr>
<tr>
<td>2</td>
<td>Sub-Himalayan Zone</td>
<td>Churia Range (Siwaliks)</td>
<td>10-50</td>
<td>200-1300</td>
<td>Sandstone, mudstone, shale and conglomerate.</td>
<td>Tectonic upliftment, erosion, and slope failure</td>
</tr>
<tr>
<td>3</td>
<td>Sub-Himalayan Zone</td>
<td>Dun Valleys</td>
<td>5-30</td>
<td>200-300</td>
<td>Valleys within the Churia Hills filled up by coarse to fine alluvial sediments</td>
<td>River deposition, erosion and tectonic upliftment</td>
</tr>
<tr>
<td>4</td>
<td>Lesser Himalayan Zone</td>
<td>Mahabharata Range</td>
<td>10-35</td>
<td>1000-3000</td>
<td>Schist, phyllite, gneiss, quartzite, granite and limestone belonging to the Lesser Himalayan Zone</td>
<td>Tectonic upliftment, Weathering, erosion, and slope failure</td>
</tr>
<tr>
<td>5</td>
<td>Lesser Himalayan Zone</td>
<td>Midlands</td>
<td>40-60</td>
<td>300-2000</td>
<td>Schist, phyllite, gneiss, quartzite, granite, limestone geologically belonging to the Lesser Himalayan Zone</td>
<td>Tectonic upliftment, Weathering, erosion, and slope failure</td>
</tr>
<tr>
<td>6</td>
<td>Lesser Himalayan Zone</td>
<td>Fore Himalayas</td>
<td>20-70</td>
<td>2000-5000</td>
<td>Gneisses, schists, phyllite and marbles mostly belonging to the northern edge of the Lesser Himalayan Zone</td>
<td>Tectonic upliftment, Weathering, erosion, and slope failure</td>
</tr>
<tr>
<td>7</td>
<td>Higher Himalayan Zone</td>
<td>Higher Himalayas</td>
<td>10-60</td>
<td>&gt;5000</td>
<td>Gneisses, schists, migmatites and marbles belonging to the Higher Himalayan Zone</td>
<td>Tectonic upliftment, Weathering, erosion (rivers and glaciers), and slope failure</td>
</tr>
<tr>
<td>8</td>
<td>Higher Himalayan Zone and Tibetan Tethys Himalayan Zone</td>
<td>Inner and Trans Himalayas</td>
<td>5-50</td>
<td>2500-4500</td>
<td>Gneisses, schists and marbles of the Higher Himalayan Zone and Tethyan sediments (limestone, shale, sandstone etc.) belonging to the Tibetan-Tethys Zone</td>
<td>Tectonic upliftment, wind and glacial erosion, and slope degradation by rock disintegrations</td>
</tr>
</tbody>
</table>
2.2 **Historical Seismicity in Nepal**

Historical Seismicity is the historical records of earthquakes preserved in different forms such as written history, chronicles, inscription, etc. which plays an important role in the seismic hazard assessment because instrumentally recorded earthquakes are lacking before the current century (National Seismological Centre (NSC) Nepal 2008). Historical events must be available for a long period of human civilization, which should throw light upon the extent of damage besides the date and place of occurrence. The earthquake of 15 January 1934 A.D. is the most devastating earthquake that ever occurred in the territory of Nepal. With a magnitude of 8.3 and casualties of more than 16,000 people from Nepal and India (National Seismological Centre Nepal) this earthquake is remembered every year as the national earthquake day in Nepal. The rupture length for this event is estimated to be 200 km ± 100 km (Molnar and Pandey 1988).

A list of the notable earthquake events in the history of Nepal is presented below (National Society for Earthquake Technology Nepal - NSET 2009).

*Table 3 Notable Historical Seismic Events (National Society for Earthquake Technology Nepal - NSET 2009)*

<table>
<thead>
<tr>
<th>Date</th>
<th>Epicentre</th>
<th>Intensity/Magnitude</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Latitude</td>
<td>Longitude</td>
<td></td>
</tr>
<tr>
<td>1255</td>
<td>27.7</td>
<td>85.3</td>
<td>X Modified Mercalli</td>
</tr>
<tr>
<td>1408</td>
<td>27.7</td>
<td>85.3</td>
<td>X Modified Mercalli</td>
</tr>
<tr>
<td>1681</td>
<td>27.7</td>
<td>85.3</td>
<td>IX Modified Mercalli</td>
</tr>
<tr>
<td>1810</td>
<td>27.7</td>
<td>85.3</td>
<td>IX Modified Mercalli</td>
</tr>
<tr>
<td>1833/08/26</td>
<td>28</td>
<td>85</td>
<td>7.8 Richter</td>
</tr>
<tr>
<td>1869/07/07</td>
<td>28</td>
<td>85</td>
<td>7 Richter</td>
</tr>
<tr>
<td>1934/01/15</td>
<td>27.6</td>
<td>87.1</td>
<td>8.3 Richter</td>
</tr>
<tr>
<td>1936/05/27</td>
<td>28.5</td>
<td>83.5</td>
<td>7 Richter</td>
</tr>
<tr>
<td>1954/09/04</td>
<td>28.3</td>
<td>83.8</td>
<td>6.5 Richter</td>
</tr>
<tr>
<td>1988/08/20</td>
<td>26.8</td>
<td>86.6</td>
<td>6.8 Richter</td>
</tr>
</tbody>
</table>

During the past fourteen years since the NSC started the seismological measurements, there have been around 50 earthquakes of magnitude greater than 5 with the largest one measuring 5.9 which occurred in Gorkha. A location map of the historical earthquakes is presented in Figure 9.
According to the GSHAP seismic data, Nepal lies in a region with high to very high seismic hazard. A seismic hazard map for Nepal was developed by (Amateur Seismic Centre Pune 2006) which is shown in Figure 10 also illustrating the approximate location of the proposed Arun-3 dam site.

### 2.3 Hydropower and Dams in Nepal

Nepal has over 6000 big and small rivers that originate in the high Himalayas and flow towards the south to join the tributaries of the Ganges. A vast majority of these rivers are snow fed and are therefore possible permanent sources for hydroelectricity. Also Nepal has a topography varying from 8848m (Mt. Everest) to 50m (Kechana Kalan) above the sea level within a very
short width ranging from 150 km to 250 km (North to South). This imparts a steep mountainous terrain. Thus, Nepal has a vast hydropower potential.

There are currently around 74 power stations in total, out of which 70 are hydropower plants (Nepal Electricity Authority 2008). The total installed power generation is 687.5 MW in the Integrated Nepal Power System (INPS) out of which 634.23 MW is from hydro resources. This accounts to 92.26% of the total generation (Nepal Electricity Authority 2008).

However, Nepal is crippled with power shortage these days due to increasing demand. Since hydro resource is the most abundant in Nepal it has to be utilised the most. It is a notable fact that India has a fast growing economy and has a rapidly increasing demand for power and is also currently facing harsh situations of power deficit. With limited sources of coal and other natural resources, India has to rely on Nepal for its increasing power needs. According to a survey conducted by the Ministry of Water Resources, GoN, the techno-economically feasible hydropower generation capacity of Nepal is 43,000 MW. It is quite ironical that the country is currently generating less than 2% of the techno-economically feasible capacity. If the generation of the full capacity were to be achieved then Nepal will not only be self-sufficient in energy but also be catering the increasing demands of the Indian power market. Therefore, promotion of hydropower projects in Nepal is inevitable. There is a large number of massive hydropower projects planned in Nepal for the coming decade. The following inventory of dams and weirs in Nepal were retrieved from various sources including (Nepal Electricity Authority 2008) and other related websites (Bhattarai D. 2007).
<table>
<thead>
<tr>
<th>SN</th>
<th>Name</th>
<th>Status</th>
<th>Power (MW)</th>
<th>River</th>
<th>Type</th>
<th>Ht. of Dam/Weir</th>
<th>Lt. of Dam/Weir</th>
<th>Dam</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Middle Marshyangdi HEP</td>
<td>Commissioned</td>
<td>70</td>
<td>Marshyangdi</td>
<td>RoR</td>
<td>62</td>
<td>95</td>
<td>Concrete</td>
</tr>
<tr>
<td>2</td>
<td>Chilime HPP</td>
<td>Commissioned</td>
<td>20</td>
<td>Chilime</td>
<td>RoR</td>
<td>3</td>
<td>13</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>Kali Gandaki ‘A’ HEP</td>
<td>Commissioned</td>
<td>144</td>
<td>Kali Gandaki</td>
<td>RoR</td>
<td>44</td>
<td>105</td>
<td>Concrete</td>
</tr>
<tr>
<td>4</td>
<td>Modi Khola HEP</td>
<td>Commissioned</td>
<td>14</td>
<td>Modi</td>
<td>RoR</td>
<td>7.5</td>
<td>33</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>Ilam (Puw Khola) HPP</td>
<td>Commissioned</td>
<td>6.2</td>
<td>Puwa</td>
<td>RoR</td>
<td>15</td>
<td>60</td>
<td>Concrete</td>
</tr>
<tr>
<td>6</td>
<td>Upper Bhotekoshi HEP</td>
<td>Commissioned</td>
<td>36</td>
<td>Bhotekoshi</td>
<td>RoR</td>
<td>2.5</td>
<td>17</td>
<td>Low weir</td>
</tr>
<tr>
<td>7</td>
<td>Khimti I HPP</td>
<td>Commissioned</td>
<td>60</td>
<td>Khimti</td>
<td>RoR</td>
<td>10</td>
<td>30</td>
<td>Concrete</td>
</tr>
<tr>
<td>8</td>
<td>Kulekhani II HPP</td>
<td>Commissioned</td>
<td>32</td>
<td>Kulekhani</td>
<td>Storage</td>
<td>114</td>
<td></td>
<td>Rock filled</td>
</tr>
<tr>
<td>9</td>
<td>Kulekhani I HPP</td>
<td>Commissioned</td>
<td>60</td>
<td>Kulekhani</td>
<td>Storage</td>
<td>114</td>
<td></td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>Marshyangdi HPP</td>
<td>Commissioned</td>
<td>69</td>
<td>Marshyangdi</td>
<td>RoR</td>
<td>3 gates 13.8: 2 gates 14.8</td>
<td>102</td>
<td>Concrete</td>
</tr>
<tr>
<td>11</td>
<td>Mailung Khola HPP</td>
<td>Ongoing</td>
<td>5</td>
<td>Mailung</td>
<td>RoR</td>
<td>2.5</td>
<td>20</td>
<td>Weir</td>
</tr>
<tr>
<td>12</td>
<td>Upper Modi HEP</td>
<td>In pipeline</td>
<td>14</td>
<td>Modi</td>
<td>RoR</td>
<td>10</td>
<td>30</td>
<td>Concrete</td>
</tr>
<tr>
<td>13</td>
<td>Indrawati III HEP</td>
<td>Commissioned</td>
<td>7.5</td>
<td>Indrawati</td>
<td>RoR</td>
<td>5</td>
<td>62</td>
<td></td>
</tr>
<tr>
<td>14</td>
<td>Aandhi Khola Project</td>
<td>Commissioned</td>
<td>5.1</td>
<td>Aandhi Khola</td>
<td>RoR</td>
<td>6</td>
<td>60</td>
<td></td>
</tr>
<tr>
<td>15</td>
<td>Jhimruk HREP</td>
<td>Commissioned</td>
<td>12</td>
<td>Jhimruk</td>
<td>RoR</td>
<td>2</td>
<td>205</td>
<td>Weir</td>
</tr>
<tr>
<td>16</td>
<td>Trishuli HPP</td>
<td>Commissioned</td>
<td>24</td>
<td>Trishuli</td>
<td>RoR</td>
<td>10</td>
<td>130</td>
<td></td>
</tr>
<tr>
<td>17</td>
<td>Gandak HEP</td>
<td>Commissioned</td>
<td>15</td>
<td>Narayani</td>
<td>RoR</td>
<td>6</td>
<td>60</td>
<td></td>
</tr>
<tr>
<td>18</td>
<td>Sunkoshi HPP</td>
<td>Commissioned</td>
<td>10.05</td>
<td>Sunkoshi</td>
<td>RoR</td>
<td>10</td>
<td>30</td>
<td></td>
</tr>
<tr>
<td>19</td>
<td>Chamelia</td>
<td>Ongoing</td>
<td>30</td>
<td>Chamelia</td>
<td>RoR</td>
<td>10</td>
<td>30</td>
<td></td>
</tr>
<tr>
<td>20</td>
<td>Devighat</td>
<td>Commissioned</td>
<td>14.1</td>
<td>Trishuli</td>
<td>RoR</td>
<td>10</td>
<td>30</td>
<td></td>
</tr>
<tr>
<td>21</td>
<td>West Seti</td>
<td>Ongoing</td>
<td>750</td>
<td>Seti (West)</td>
<td>RoR</td>
<td>195</td>
<td></td>
<td></td>
</tr>
<tr>
<td>22</td>
<td>Upper Karnali</td>
<td>Ongoing</td>
<td>300</td>
<td>Karnali</td>
<td>RoR</td>
<td>27</td>
<td>60</td>
<td>Concrete</td>
</tr>
<tr>
<td>23</td>
<td>Budhi Gandaki</td>
<td>In pipeline</td>
<td>600</td>
<td>Budhi Gandaki</td>
<td>Storage</td>
<td>225</td>
<td></td>
<td>Zoned</td>
</tr>
<tr>
<td>24</td>
<td>Arun III</td>
<td>In pipeline</td>
<td>402</td>
<td>Arun</td>
<td>RoR</td>
<td>65</td>
<td>155</td>
<td>Concrete</td>
</tr>
<tr>
<td>25</td>
<td>Kali Gandaki II</td>
<td>In pipeline</td>
<td>660</td>
<td>Kali Gandaki</td>
<td>Storage</td>
<td>177</td>
<td></td>
<td></td>
</tr>
<tr>
<td>26</td>
<td>Lower Arun</td>
<td>Proposed</td>
<td>308</td>
<td>Arun</td>
<td>RoR</td>
<td>10.5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>27</td>
<td>Upper Arun</td>
<td>Proposed</td>
<td>335</td>
<td>Arun</td>
<td>RoR</td>
<td>37</td>
<td></td>
<td></td>
</tr>
<tr>
<td>28</td>
<td>Pancheswar Multi Purpose</td>
<td>Proposed</td>
<td>6480</td>
<td>Mahakali</td>
<td>Storage</td>
<td>315</td>
<td></td>
<td></td>
</tr>
<tr>
<td>29</td>
<td>Koshi High Dam Project</td>
<td>Proposed</td>
<td>5500</td>
<td>Koshi</td>
<td>Storage</td>
<td>269 to 335</td>
<td></td>
<td></td>
</tr>
<tr>
<td>30</td>
<td>Upper Tamakoshi</td>
<td>Ongoing</td>
<td>309</td>
<td>Tama Koshi</td>
<td>PRoR</td>
<td>177</td>
<td></td>
<td></td>
</tr>
<tr>
<td>31</td>
<td>Koshi Barrage</td>
<td>Existing</td>
<td>15</td>
<td>Saptap Koshi</td>
<td>Flood control</td>
<td>177</td>
<td></td>
<td>Concrete</td>
</tr>
<tr>
<td>32</td>
<td>Dudh Koshi Project</td>
<td>Proposed</td>
<td>300</td>
<td>Dudh Koshi</td>
<td>Storage</td>
<td>180</td>
<td></td>
<td></td>
</tr>
<tr>
<td>33</td>
<td>Karnali Chisapani</td>
<td>In pipeline</td>
<td>10800</td>
<td>Karnali</td>
<td>Storage</td>
<td>255</td>
<td></td>
<td>Zoned</td>
</tr>
</tbody>
</table>
2.4 Arun Hydroelectric Development Plans

Water resources of the Himalaya are often viewed as the key to successful development of much of the Indian subcontinent. Year-round flow in the snow fed rivers provides water for agriculture during the long dry-season and also represents enormous hydroelectric potential of the region. The Arun River drains a large area of the Tibetan Plateau before crossing the Himalayas into Nepal where its discharge increases dramatically. The steep gradient and relatively high dry-season flow of the Arun have led to plans for major hydroelectric project (HEP) development. Several hydropower projects have been planned for implementation in the river viz.

- Upper Arun Hydroelectric Project (335 MW)
- Lower Arun Hydroelectric Project (308 MW)
- Arun III Hydroelectric Project (402 MW)

The immense hydropower potential of the Arun river has long been recognised from considerations of the large discharge and steep gradient. The first detailed assessment of the basin estimated that more than 1100 MW of electricity could be generated in a cascade of six power stations (JICA Japan International Cooperation Agency 1984). One proposed site known as Arun-3 was particularly attractive because of an S-shaped curve in the channel around a ridge of resistant rock. Because a tunnel could cut off the bend and drop more than 200 m in 11 km, this project was judged the most efficient development (Nepal Electricity Authority N. 1986).

The Project was first identified in 1984. NEA (Nepal Electricity Authority) carried out a detailed feasibility study in 1986. The study was based primarily on the field topographical and geological investigations, load forecast and supply programme, and preliminary design of the civil structures and electric facilities. A concrete gravity dam would be built across Arun just upstream of Num and a powerhouse at Pikhuwa to generate a capacity of 400 MW. Another site with characteristics similar to Arun-3 was identified in 1985 near the border with China. This project was called Upper Arun. It also takes the advantage of a dramatic curve in the channel to minimise the length of the headrace tunnel. This project would generate a capacity of 335 MW.

Of the above mentioned three projects, this report summarises the study of the seismiological assessment of the concrete gravity dam of the Arun III HEP (402 MW). The study was performed as a step by step analysis starting from linear static analysis to linear dynamic modal analysis (Response Spectrum Modal Analysis) and finally to dynamic transient analysis (Time History Analysis).

A schematic view of the recognized hydroelectric development projects along the Arun river is shown in Figure 11.
Arun III is a Pondage Run-of-River project, which is planned to be constructed in the Sankhuwasabha district of the Koshi Zone in eastern Nepal. According to the feasibility study report (Nepal Electricity Authority N. 1986) the dam is proposed to be 65m high concrete gravity structure with a total concrete volume of 150,000 m$^3$. The reservoir is estimated to have an effective storage of 2,500,000 m$^3$ of water. A brief description of the topography, geology and seismology of the region is discussed below.

### 2.5.1 Topography

The Arun River is the largest trans-Himalayan river passing through Nepal and has the greatest snow and ice-covered area of any Nepalese river basin. The Arun drains more than half of the area contributing to the Sapta Koshi river system but provides only about a quarter of the total discharge. This apparent contradiction is due to the fact that the location of more than 80
percent of the Arun’s drainage area of about 30,000 km² is in the rain shadow area of the Himalayas. It should be noted that the average annual precipitation in Tibet is about 300 mm (Liu G. 1989)

According to the design documents from Nepal Electricity Authority (1986), the river is known as Men Qu in Tibet in its upper reaches north of Xixabangma and then as the Peng Qu for most of its course north of the Himalayan crests. After progressing eastward through the arid grasslands, the Peng Qu turns south at the 4050 m elevation with the Yaru Qu. The Peng Qu then flows through the narrow Yo Ri gorge and a broad valley before entering the Longdui gorge at 3500 m at a point below Kharta. The climate changes abruptly in this area from rain shadow to monsoon-soaked (Howard-Bury C. K. 1922). This portion of the Peng Qu basin may generate much of the stream flow that crosses the border. The Peng Qu crosses the Himalayan crest at an elevation of 2175 m which is the Arun in Nepal. A Google Earth image of the river indicating its Trans-Himalayan stretch is shown in Figure 12. Seen in the figure is the river Arun (in red stretching roughly top-bottom) across the snow-capped Himalayas along the Tibet-Nepal border (yellow line across the middle of the figure stretching left-right). The location of the dam site is indicated in the upper middle third of the picture (yellow pin). The approximate location of the site as per the design document from GoN is given in Table 5.

![Figure 12 Trans-Himalayan stretch of the Arun river entering Nepal from Tibet looking South (Google Earth image Viewed on 10/10/2008)](image)

<table>
<thead>
<tr>
<th>Table 5 Dam location coordinates</th>
</tr>
</thead>
<tbody>
<tr>
<td>Latitude</td>
</tr>
<tr>
<td>Longitude</td>
</tr>
<tr>
<td>Elevation</td>
</tr>
</tbody>
</table>

*masl: meters above sea level
South of the Himalayan crest, the flow of the Arun increases rapidly downstream in the seasonally humid environment of eastern Nepal. The 5000 km² of land contributing water to the Arun inside Nepal is only 17 percent of the total basin area, but it provides more than 70 percent of the Arun total flow at its confluence with the Sapta Koshi (JICA Japan International Cooperation Agency 1984). The landscape south of the border tends to be steep with less than 15 percent of the area having sustained slope of less than 15° and is profusely dissected by stream channels. Many of the hill slopes are structurally unstable, and the region is seismically active (Kansakar D. R. 1988). An earthquake in August 1988 with an epicentre more than 50 kms south of the Arun basin had a magnitude of 6.7 on the Richter scale and resulted in more than 100 deaths in the Arun basin alone (Dunsmore J. R. 1988). Soil tends to be shallow and stony (generally less than 20 cm deep) (Goldsmith P.F. 1981). The alpine zone above 4000 m covers about 5-10 percent of the lower Arun basin (Shrestha T. B. 1988). Several large glaciers are found in the Barun river tributary near 8000 m peak Makalu.

The northern third of the Nepalese portion of the Arun basin supports a rich, though human-modified, forest of mixed hardwoods, Chir pine, fir, and rhododendron at elevations of 1000 to 4000 m (Cronin E. W. 1979); (Shrestha T. B. 1988). The vegetation in the southern two thirds of the area has been extensively modified for subsistence agriculture. Most of the people in the Arun river basin live in this southern area between 300 and 2200m in widely scattered villages near the slopes they farm (Shrestha T. B. 1988).

2.5.2 Geology

a) Regional geology

With reference to the feasibility study report the geological formation of the entire project area is gneiss in general and mica schist belonging to the Himalayan group.

The Himalayan group is intruded by Granitic rock and amphibolite whose dip and strike is parallel to the stratification of the gneissocity of the Himalayan gneiss. Although Himalayan gneiss is generally massive and compact, it exhibits various facies depending on the degree of mica content. Gneiss distributed in the dam site is of high mica content.

- **Geo-structure**

The subject area is situated on the eastern wing of the Arun anticline. The anticlinal axis extends roughly north south (Refer Figure 13 ). As a result, the Himalayan gneiss of the subject area dips to the east. The dip angle increases away from the dam axis.

- **Landslide and slope failure**

Since the area lies in an active uplift zone, there are a large number of massive landslide zones in the area. From the reference of the feasibility study report of the Arun III project conducted the slope failures and landslide zones are of the following types:
• Those resulting from the mass movement along bedding plane on the right bank of Arun river
• Those resulting from toppling occurring in areas of concentrated jointing due to prolonged action of tectonic stress
• Those resulting as a combination of above two types; and
• Those resulting from the secondary failure of talus slope at the lower section of slopes

![Figure 13 Schematic Geological Map of the Arun (Nepal Electricity Authority N. 1986)](image)

**b) Geology in the vicinity of the dam site area**

The Arun River is one of the major rivers of Eastern Nepal. Originating in the Himalayas and Tibet highlands, it flows southwards through the Himalayas. The ample discharge of the river has caused erosion through the Himalayas forming one of the deepest gorges in the world.

Referring to the feasibility study report (Nepal Electricity Authority N. 1986), the Arun River valley generally comprises of a steep “V” shaped gorge, with riverbed terraces occurring only at confluences with the tributaries. On the left bank downstream of the dam site, the river terrace, which is 25m deep, forms a relatively flat area. An old terrace gravel bed is observed at about a height of 100m from the riverbed on the left bank of the dam site area. However due to limited distribution and excessive erosion, this formation is not sufficiently widespread to form a topographical unit.
According to the feasibility study report, this area contains slope failure zones governed by
geological composition. These include land sliding where slippage occurs along bedding planes,
topping where slippage occurs along joint planes, and talus slope failure of secondary
sediments resulting from the first two types of slippage. Landslides failure is present on the
right bank at the dam axis which is a typical example of bedding plane slippage where the
present topographical surface and bedding plane surface are the same.

2.5.3 Structural Details of the Dam and the Spillway

With reference to the feasibility study report (Nepal Electricity Authority N. 1986), the
proposed dam is located approximately 250 m upstream of the junction of the Khoktak Khola
and Num Khola with the mainstream of the Arun River.

According to the feasibility study report, currently the riverbed is at an elevation of EL 793m to
795m and the river has a width of 50m to 60m covered with riverbed deposit of 13m thickness.
The bedrock underlying the riverbed deposit was found at EL 781m to EL 782m according to
the results of drilling investigation carried out in the vicinity as stated in the feasibility study
report. The report judges that the bedrock has adequate strength for the dam foundation.

The dam is designed to be 155m long, with its main monolith 65m high which would create a
reservoir area of 50 ha. The dam has two sections, one is a non-overflow section and the other
section serves as a spillway for discharging floodwater. The report states that an optimisation
study was performed for the dam and the results indicated that the normal water level of EL
840m was optimum with a high water level of EL 844m.

Furthermore, the dam is designed with its crest elevation for the non-overflow section at EL
846m (this includes the surcharge of 2m for design flood, 1.5m for wind-induced wave and
0.5m for earthquake-induced wave). This adds up to a total height of 65m for the non-overflow
section. The width of this section is 75m, including three sections where flushing facilities are
envisioned.

2.5.4 Seismology

It has been mentioned in the seismological section of the feasibility study report that for the
Arun III project study, direct geological exploration involving core drilling survey was carried
out together with seismic prospecting. According to the definition provided in (Mining Basics
2008), seismic prospecting is the most widespread geophysical method in underground
exploration. Small artificial shock waves are generated at a selected point by either firing a
charge of explosives in a shallow drill hole or dropping a heavy weight. The speed of the shock
waves is measured by timing their arrival at sensitive receivers called geophones placed along
the survey line.
As explained in the feasibility study report, the equipment used in the seismic prospecting consists of 24 channel fieldgraphs, 24 channel amplifiers and a vertical moving coil type geophone. The survey team carried out seismic prospecting along the right bank above dam axis, left bank and on the proposed dam foundation. The result from the seismological study was used to prepare a geological map of the survey area which is included in the feasibility study report.

2.6 Conclusion

In this chapter the seismicity in the Himalayas was discussed including the topographical, geological and seismological aspects of the Arun III dam site located in the eastern Nepal Himalayas. From past studies and on the basis of design documents from DoED, it is clear that the site is in a seismically active region. With concern to the grave consequences following a potential earthquake, Arun III dam should be assessed relating to its performance against major earthquakes which are likely in the Himalayan seismic environment. For this purpose, a deep understanding of the characteristics of a concrete gravity dam including the study of performance assessment methods is necessary which is discussed in Chapter 3.
Chapter 3

Dams: Characteristics and Methodologies for Performance Assessment

3.1 Introduction

According to the definition provided by Tancev (2005), civil engineering structures carried out for solving specific water related tasks are called hydraulic structures, while the applied science dealing with their general theory, design, construction and operation is hydraulic engineering. Dams are the hydraulic structures for blocking a riverbed in order to raise the water level and to create an artificial lake (an impounding reservoir). According to their importance, dimensions, the complexity of the problems to be solved during their design and construction, their influence on the environment, etc., dams are considered one of the most significant engineering structures. Dams are built either with local material – clay, loam, sand, gravel, crushed stone, or with composite materials like concrete and reinforced concrete; particular structural elements require asphalt, steel, wood, plastic materials, etc. The most widespread are embankment dams, which are built with local materials followed by various kinds of concrete dams. The construction of dams dates back in the history as far as 2900 BC when the first dam was built in Egypt which was called the Sad el-Kafara (Tancev L. 2005). In modern times due to development of soil mechanics, manufacture of contemporary plants and machinery to carry out large and complex civil engineering tasks, hydraulic engineering has experienced an overwhelming development and likewise the construction of dams. The development of modern methods for static and dynamic analyses, along with model testing investigation of structures also helped foster dam engineering. According to ICOLD (2008) (International Commission on Large Dams), today there are more than 45,000 large dams (higher than 15 m) listed in the World Register of Dams, with more than half in China.

The only notable damage on dams from seismic load has been to the Koyna dam in India which sustained the Koyna earthquake but suffered major cracks primarily on its non-overflow monoliths. This however does not mean that dams are immune from seismic events. They pose a great risk to the downstream settlements and therefore require due consideration for appropriate seismic demand to exhibit desirable level of seismic performance. This chapter will illustrate the aspects of dam design and performance assessment methodologies

3.2 Types of Dams

Dams can be classified as embankment dams and concrete dams on the basis of the construction material used ICOLD (2008).
a) Embankment Dams:

These are built of earth and/or rock fill and resist the water pressure by their weight. If the material is not inherently watertight, they are faced with an impervious material or have a watertight core. This is the oldest and most widespread type of dam, accounting for 83 per cent of the world total. They are usually trapezoidal in section.

Embarkment dams can be of the following types

- Homogeneous embankment dams are more or less made up of impervious material and the material is same throughout the cross section of the dam.
- Zoned embankment dams have different zones constructed of various materials inside the dam wall and impermeability is ensured by means of a relatively thin zone of cohesive material.
- Impervious facing embankment dams with diaphragm walls imparting impermeability to the dam are the third type of embankment dams.

b) Concrete Dams:

Types are as follows:

- Concrete Gravity dams have a roughly triangular cross section and also resist water pressure by their weight. This is the most widespread type of concrete dam, and the type proposed for the Arun III dam.
- Arch dams transmit most of the water load into the valley sides or large concrete thrust blocks.
- Buttress dams have the water load transmitted to triangular buttresses parallel to the direction of river flow
- Multiple arch dams consist, as their name implies of a number of small arches bearing on buttresses.
- Barrages are a special case, consisting of a line of large gates which, when closed, transmit the water load to flanking piers. Barrages are built on wide, slow moving rivers.

3.3 Forces acting on a Concrete Gravity Dam

A gravity dam is so called because it is designed to resist the forces acting on it with its self weight (gravity). The section of a gravity dam is more or less triangular. In preliminary design the dam is designed as a triangular section with a vertical face upstream. For the purpose of inspection the apex is then modified to include a road. This modification increases weight on the downstream side of the dam. To counterbalance this additional load due to the modified apex, the lower part of the upstream is modified to add more weight with an inclined face below
some level. In general, the forces associated with a gravity dam are as follows (Refer Figure 14):

![Figure 14 Forces acting on a gravity dam (Directions shown are not necessarily critical)](image)

a) Weight of the dam
The main stabilising force for a gravity dam is its weight. It comprises the weight of the concrete/masonry along with the weight of the appurtenances (gates, piers, etc). The weight of the dam per unit length is equal to the product of the area of the dam and the specific weight of the material. The total weight of the dam acts at the C.G. of the section.

The specific weight of concrete is usually taken 24 kN/m$^3$. However the actual specific weight may vary depending upon water-cement ratio, compaction of concrete and the unit weight of the aggregate used.

b) Water Pressure
The water pressure intensity varies linearly with the depth of the water measured below the free surface and is expressed as

$$p = \gamma_w h$$  \hspace{1cm} (1)

where,

$\gamma_w$ is the specific weight of water (9.81 kN / m$^3$)

The water pressure always acts normal to the surface. In the case of inclined face of the dam, the pressure is computed as vertical and horizontal components which are given below:
\[ P_n = \frac{1}{2} \gamma_w H^2 \]  

(2)

and it acts horizontally at a height \( H/3 \) above the base of the dam.

The vertical component is equal to the weight of the water column above the inclined portion of the upstream face of the dam and will act through the centroid of the area associated with the water column.

c) Uplift Pressure

Water has a tendency to seep through the pores and fissures of the foundation material. The seeping water exerts pressure and should be included in the stability calculations. The uplift pressure is defined as the upward pressure of water as it flows/seeps through the dam or its foundation. A portion of the dam weight will be effectively supported by this uplift thus reducing the net vertical force.

According to the Indian Standard (IS: 6512-1984 Bureau of Indian Standards), the method for the calculation of the uplift pressure is illustrated in Figure 15. According to the IS code, the uplift pressure distribution in the dam body shall have an intensity, at the line of the drains exceeding the tail water pressure by one-third the differential of the reservoir level and the tail water level. The pressure gradient shall extend linearly to heads corresponding to reservoir level and tail water level, the uplift acting over 100% area. In case of absence of the line of drains and for extreme loading conditions, the uplift shall be varying linearly from reservoir water pressure in the upstream to tail water pressure in the downstream. If the reservoir water pressure exceeds the vertical normal stress then a crack will be assumed from the upstream face to the point where the vertical normal stress is equal to the reservoir pressure at the elevation. The uplift pressure shall be equal to the reservoir pressure throughout the assumed crack length and then vary linearly to the tail water pressure level in the downstream.
d) Silt/Earth Pressure

According to the IS code, the horizontal silt and water pressure is equivalent to that of a fluid with a mass of 1360 kg/m$^3$, and the vertical silt and water pressure is determined as if silt and water together have a density of 1925 kg/m$^3$.

e) Seismic Forces

The seismic load is primarily accounted for as a coefficient along with the weight of the dam for preliminary analysis. This is shown as the seismic inertia in Figure 14. However for further analysis response spectrum method or time history analysis is required. These methods are
described in subsequent chapters. Also the dam experiences inertia force exerted by the mass of water which is adjacent to the dam, termed as the hydrodynamic interaction effect.

The hydrodynamic interaction effect of the water adjacent to the upstream face of the dam is expressed by an equivalent added mass of water. This concept assumes the water is incompressible, and provides added hydrodynamic mass functions that represent the inertial effects of water interacting with the dam face. In the analysis, the added hydrodynamic mass model developed by Westergaard (1933) was used. The Westergaard’s added hydrodynamic mass model is based on the assumptions of a rigid dam, straight in plan with a vertical upstream face to accelerate in an upstream-downstream direction only. The water in the reservoir is assumed to be incompressible with constant depth and infinite reservoir length. The model for the hydrodynamic pressure is a parabolic function as given below

\[ p = \frac{7}{8} \rho \sqrt{h(h-z)} \ddot{u} \]  

Where,

- \( p \) is the hydrodynamic pressure at height \( z \) above the reservoir bottom
- \( h \) is the depth of the reservoir
- \( \rho \) is the mass density of water and
- \( \ddot{u} \) is the horizontal ground acceleration

From equation (3), it follows that, the hydrodynamic pressure is maximum at the reservoir bottom (where \( z = 0 \)) and decreases parabolically to zero at the reservoir surface (where \( z = h \)). The hydrodynamic pressure on the dam is equivalent to the inertia force exerted by the mass of water “attached” to the dam and accelerates together with the dam in the upstream-downstream direction which is the concept behind the term “added mass” as introduced by Westergaard.

f) Other forces (Ice pressure, Wave pressure, etc)

Various other forces may be considered as per the site specific requirement like Ice pressure, Wind pressure, wave pressure etc.

### 3.4 Seismic Risks to Dams

Earthquakes are a major problem for humankind, killing thousands each year. Earthquakes do not only take lives but are also responsible for other losses ranging from collapse of structures to fire, tsunamis and landslides. Structures are linked directly to human lives and are hence required to be safe. Although no concrete dam has failed because of earthquake activity, there have been instances of significant damage to some dams viz. Koyna Dam, Maharastra, India; Hsinfengkiang, China; Sefid Rud, Iran. According to Wieland (2004) these dams suffered
considerable level of damage (cracks appeared in the Koyna and the Sefid Rud dams, one monolith was displaced permanently due to fault movement in Hsinfengkiang dam).

The hazard posed by large dams has long been known. Concerns about the seismic safety of concrete dams have been growing recently because the population at risk in locations downstream of major dams continues to expand and because the seismic design concepts in use at the time most existing dams were built were inadequate. As time passes by humankind has faced more and more earthquakes and the knowledge about the seismic demand and capacity of structures is getting better with each of these events.

**Reservoir Triggered Seismicity**

Lately there have been concerns about the increase in the order of seismic activities around large reservoir induced by water level periodic variation. It is believed that large reservoirs located in seismically active zones are the cause of earthquakes around the area. According to (Wieland M. 2004) the basic requirements for reservoir-triggered seismic events are:

- The existence of active faults,
- The existence of faults near failure limit

It implies that reservoir triggered seismic events can only occur in regions with high tectonic stresses in the earth crust. The stresses increased due to the reservoir impounding may trigger the release of seismic energy. This however does not mean that triggering due to impounding can change the underlying tectonic processes and long term seismic hazard at the dam site (Wieland M. 2004).

The best example to this case may be the Enguri high dam in Western Georgia where an the increase in seismic activities were reported during the periodic variation of the water level in the reservoir (Teimuraz Matcharashvili et al. 2007).

<table>
<thead>
<tr>
<th>Reservoir</th>
<th>Country</th>
<th>Depth (m)</th>
<th>Volume</th>
<th>Magnitude/Intensity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Akosombo</td>
<td>Ghana</td>
<td>109</td>
<td>148,000</td>
<td>MMI V</td>
</tr>
<tr>
<td>Almendra</td>
<td>Spain</td>
<td>185</td>
<td>2,649</td>
<td>3.2</td>
</tr>
<tr>
<td>Aswan</td>
<td>Egypt</td>
<td>90</td>
<td>160,000</td>
<td>5.2</td>
</tr>
<tr>
<td>Benmore</td>
<td>New Zealand</td>
<td>96</td>
<td>2,040</td>
<td>5.0</td>
</tr>
<tr>
<td>Blowering</td>
<td>Australia</td>
<td>142</td>
<td>2,559</td>
<td>3.5</td>
</tr>
<tr>
<td>Camarillas</td>
<td>Spain</td>
<td>43</td>
<td>37</td>
<td>4.1</td>
</tr>
<tr>
<td>Canelles</td>
<td>Spain</td>
<td>132</td>
<td>678</td>
<td>4.7</td>
</tr>
<tr>
<td>Capivara</td>
<td>Brazil</td>
<td>60</td>
<td>10,500</td>
<td>4.4</td>
</tr>
<tr>
<td>Location</td>
<td>Country</td>
<td>Year</td>
<td>Volume</td>
<td>Magnitude</td>
</tr>
<tr>
<td>---------------</td>
<td>---------</td>
<td>------</td>
<td>--------</td>
<td>-----------</td>
</tr>
<tr>
<td>Cenajo</td>
<td>Spain</td>
<td>1997</td>
<td>472</td>
<td>4.2</td>
</tr>
<tr>
<td>Danjiangkou</td>
<td>China</td>
<td>1997</td>
<td>16,000</td>
<td>4.7</td>
</tr>
<tr>
<td>El Grado</td>
<td>Spain</td>
<td>1997</td>
<td>400</td>
<td>MMI IV</td>
</tr>
<tr>
<td>Eucumbene</td>
<td>Australia</td>
<td>1997</td>
<td>4,761</td>
<td>5.0</td>
</tr>
<tr>
<td>Furnas</td>
<td>Brazil</td>
<td>1997</td>
<td>22,950</td>
<td>MMI V</td>
</tr>
<tr>
<td>Grandval</td>
<td>France</td>
<td>1997</td>
<td>292</td>
<td>MMI V</td>
</tr>
<tr>
<td>Hoover</td>
<td>U.S.A.</td>
<td>1997</td>
<td>36,703</td>
<td>5.0</td>
</tr>
<tr>
<td>Jocassee</td>
<td>U.S.A.</td>
<td>1997</td>
<td>1,431</td>
<td>3.8</td>
</tr>
<tr>
<td>Kariba</td>
<td>Zambia</td>
<td>1997</td>
<td>160,368</td>
<td>6.25</td>
</tr>
<tr>
<td>Kastraki</td>
<td>Greece</td>
<td>1997</td>
<td>100</td>
<td>4.6</td>
</tr>
<tr>
<td>Khoa Laem</td>
<td>Thailand</td>
<td>1997</td>
<td>7,000</td>
<td>4.5</td>
</tr>
<tr>
<td>Koyna</td>
<td>India</td>
<td>1997</td>
<td>2,780</td>
<td>6.3</td>
</tr>
<tr>
<td>Kremasta</td>
<td>Greece</td>
<td>1997</td>
<td>4,750</td>
<td>6.3</td>
</tr>
<tr>
<td>Kurobe</td>
<td>Japan</td>
<td>1997</td>
<td>199</td>
<td>4.9</td>
</tr>
<tr>
<td>Manicouagan 3</td>
<td>Canada</td>
<td>1997</td>
<td>10,423</td>
<td>4.1</td>
</tr>
<tr>
<td>Marathon</td>
<td>Greece</td>
<td>1997</td>
<td>41</td>
<td>5.75</td>
</tr>
<tr>
<td>Monteynamd</td>
<td>France</td>
<td>1997</td>
<td>275</td>
<td>MMI VII</td>
</tr>
<tr>
<td>Mossyrock</td>
<td>U.S.A.</td>
<td>1997</td>
<td>1,957</td>
<td>4.3</td>
</tr>
<tr>
<td>Nurek</td>
<td>Tajikistan</td>
<td>1997</td>
<td>11,000</td>
<td>4.5</td>
</tr>
<tr>
<td>Oroville</td>
<td>U.S.A.</td>
<td>1997</td>
<td>4,400</td>
<td>5.7</td>
</tr>
<tr>
<td>Parairebuna</td>
<td>Brazil</td>
<td>1997</td>
<td>4,740</td>
<td>3.2</td>
</tr>
<tr>
<td>Piastra</td>
<td>Italy</td>
<td>1997</td>
<td>13</td>
<td>MMI V</td>
</tr>
<tr>
<td>Preve Di Cadore</td>
<td>Italy</td>
<td>1997</td>
<td>69</td>
<td>MMI V</td>
</tr>
<tr>
<td>Voltagrande</td>
<td>Brazil</td>
<td>1997</td>
<td>3,760</td>
<td>5.1</td>
</tr>
<tr>
<td>Pukaki</td>
<td>New Zealand</td>
<td>1997</td>
<td>10,500</td>
<td>4.6</td>
</tr>
<tr>
<td>Shenwo</td>
<td>China</td>
<td>1997</td>
<td>790</td>
<td>4.8</td>
</tr>
<tr>
<td>Swift</td>
<td>U.S.A.</td>
<td>1997</td>
<td>932</td>
<td>5.0</td>
</tr>
<tr>
<td>Srinagarind</td>
<td>Thailand</td>
<td>1997</td>
<td>17,745</td>
<td>5.9</td>
</tr>
<tr>
<td>Vouglans</td>
<td>France</td>
<td>1997</td>
<td>605</td>
<td>4.4</td>
</tr>
<tr>
<td>Hsingfengkiang</td>
<td>China</td>
<td>1997</td>
<td>13,896</td>
<td>6.0</td>
</tr>
<tr>
<td>Zhelin</td>
<td>China</td>
<td>1997</td>
<td>7,170</td>
<td>3.2</td>
</tr>
</tbody>
</table>

However the same theory may not be applicable in the cases of a number of large reservoirs where no such seismic activities have been noticed or else, if noticed the shocks have been deemed not being related to the reservoir.
An earthquake in the vicinity of a dam may lead to dam leakage or even failure of the dam. Since it is impossible to predict about when, where and of what magnitude an earthquake will occur, it is necessary to assume the necessary seismic design load during the design of the structure.

3.5 Earthquake Response of Gravity dams

3.5.1 Response to internal force or deformation controlled actions

The response of a gravity dam to earthquake ground motions is illustrated in Figure 17. For earthquake motion cycles in the upstream direction, potential cracking usually occurs at the heel of the dam at the maximum expected water levels. For downstream cycles, potential cracks may occur at the slope discontinuity under the minimum expected water level conditions and near the toe of the dam.

As earthquake motion cycles swing towards the upstream direction, the potential cracking shifts to the upper part and the base of the dam. In general, the tensile stress-strain results from linear elastic FEM are used to determine if the structure meets established project performance requirements. For serviceability design, i.e. under OBE (Operational basis Earthquake) loading conditions the performance should be under the linear elastic range (Figure 18). For strength design (MDE or Maximum Design Earthquake loading condition) the performance should be within the non-linear strain hardening range (i.e. 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 for the MCE (Maximum Credible Earthquake) event.
3.5.2 Response to stability controlled rigid body actions

Rotational failure of a massive concrete structure is unlikely due to wedging action that limits the rotation. However, sliding due to shear failure can occur, leading to notable permanent displacements. The estimate of permanent displacements can be made using the upper bound sliding displacement methods or some non-linear analysis methods.

Sliding may occur in gravity dams along the construction joints, cracked sections within the dam, dam-foundation interface, weak planes within the foundation, or any combination of the aforementioned. If the joints are as strong as the parent concrete the displacement may take place at the dam-foundation interface.
3.6 Design Criteria (USACE 2007)

a) Design Earthquake Ground motions

Earthquake ground motions for the design and evaluation of Concrete Hydraulic Structures are Operational Basis Earthquake (OBE) Maximum Design Earthquake (MDE) and Maximum Credible Earthquake (MCE) ground motions. Seismic forces of the OBE level are considered unusual load whereas those associated with MDE and MCE are considered to be extreme loads. For design purposes, earthquake loads are to be combined with other loads that are expected to be present during routine operations.

- Operational Basis Earthquake (OBE)

The ER 1110-2-1806 Earthquake Design and Evaluation for Civil Works Projects (USACE 1995) defines OBE as a level of ground motion that is reasonably expected to occur within the service life of the project, i.e. with a 50 percent probability of exceedance during the service life. (This corresponds to a return period of 145 years for a project with a service life of 100 years). The related performance requirement is that the structure functions with little or no damage at all. It is a probabilistic event.

- Maximum Design Earthquake (MDE)

The MDE is the maximum level of ground motion for which a structure is designed or evaluated. As a minimum for other than critical structures, the MDE ground motion has a 10 percent chance of being exceeded in a 100 year period, (or a 1000 year return period). It can either be a probabilistic or a deterministic event.

- Maximum Credible Earthquake (MCE)

According to the definition provided in ER 1110-2-1806 (USACE 1995) the MCE is the greatest earthquake that can reasonably be expected to be generated by a specific source on the basis of seismological and geological evidence. Since a project site may be affected by earthquakes generated due to various sources each with their respective fault mechanisms, maximum magnitudes and distance from the site, there may be a large number of MCEs that could be defined. However these MCEs will be unique in terms of their characteristic ground motion parameters and spectral shapes. It is a deterministic event.

For critical structures, the MCE event must satisfy the damage control performance limit state to ensure that there is no risk of collapse. Critical structures are defined in ER 1110-2-1806 (USACE 1995) as structures that are part of a high hazard project whose failure will result in loss of life.

b) Performance levels

The following performance levels are commonly used, for the evaluation of concrete hydraulic structures according to (USACE 2007).
• **Serviceability Performance**
The structure is expected to be serviceable and operable immediately following earthquakes producing ground motions up to the OBE level.

• **Damage Control Performance**
Certain elements of the structure can deform beyond their elastic limits (non-linear behaviour) if non-linear displacement demands are low and load resistance is not diminished when the structure is subjected to MDE events. Damage may be significant, but it is generally concentrated in discrete locations where yielding and/or cracking occur. The designer should identify all potential damage regions, and be satisfied that the structure is capable of resisting static loads and if necessary can be repaired to stop further damage by non-earthquake loads.

• **Collapse Prevention Performance**
Collapse prevention performance requires that the structure does not collapse regardless of the level of damage. Damage may be unrepairable. Ductility demands can be greater than those associated with the damage control performance. If the structure does not collapse when subjected to extreme earthquake events, resistance can be expected to decrease with increasing displacements. Collapse prevention performance should only be permitted for MCE events. Collapse prevention analysis requires a Nonlinear Static Procedure (NSP) or Nonlinear Dynamic Procedure (NDP).

The (USACE 2007) states that in the design of structures both serviceability and strength should be considered. Structures must have adequate strength against failures by shear, flexure, tension and compression as in the case for plain concrete and some additional failure mechanisms namely bond failure, buckling and tensile failure in case of reinforcing steel. Failures due to lack of sufficient strength may result in loss of lives and property.

Structures should also exhibit serviceability in case of sustained and frequent loads. Usually it refers to limiting the structural displacement under static load condition. For OBE events, the serviceability requirement indicates the uninterrupted functioning of the structure with minute or no damage. For proper assessment of complex structures, the following information is necessary:

- Loading history
- Changes in system stiffness and damping along with the deformation
- Redistribution of resisting loads
- Path of the structure from initial elastic stage to the collapse prevention limit state.
The above information can be obtained from nonlinear static or dynamic analyses. However, in many cases a joint approach of engineering analysis and judgement may be necessary to decide whether the performance objectives have been met.

c) Design Approaches

- **Serviceability Design**

Serviceability design for Dams subjected to earthquake ground motions is achieved by reducing the possibility of structural damage to a negligible level. Similar to the strength performance, a suitable design basis earthquake event has to be selected along with appropriate evaluation and design procedures. The design basis earthquake event used for serviceability evaluation is the Operational Basis Earthquake (OBE).

The loading combination is as follows:

\[
Q_S = Q_D + Q_L + Q_{OBE}
\]

where:

- \(Q_S\) = Combined action due to dead, live and OBE loads for use in evaluating damage control performance
- \(Q_D\) = Dead load effect
- \(Q_L\) = Live load effect + uplift
- \(Q_{MDE}\) = Earthquake load effect from OBE ground motions including hydrodynamic and dynamic soil pressure effects

The loading combinations represent the total demand (dead load + live load + earthquake demand) for which the structure must be evaluated. The live load effect is the structural response to live loads such as hydrostatic, earth pressure, silt and temperature loads. The earthquake load effect is the response of an elastic structure to design earthquake ground motions. The earthquake load may involve multi-component ground motions with each component multiplied by +1 and -1 to account for the most unfavourable earthquake direction.

- **Strength Design**

Strength design for Concrete Hydraulic Structures subjected to earthquake ground motions is achieved by reducing the probability of structural failure to an acceptable level. This is accomplished by selecting an appropriate design basis earthquake event to be used in combination with specific design and evaluation procedures that assure the structure will perform as intended.

Seismic design and evaluation is usually based on linear-elastic response spectrum or time-history analysis procedures, although nonlinear analysis procedures can be used for evaluation.
of certain nonlinear mechanisms. The design basis earthquake event used for strength evaluation is the maximum design earthquake (MDE).

The loading combination for strength design is as follows:

\[ Q_{DC} = Q_D + Q_L + Q_{MDE} \]  

where;

- \( Q_{DC} \): Combined action due to dead, live and maximum design earthquake loads for use in evaluating damage control performance
- \( Q_D \): Dead load effect
- \( Q_L \): Live load effect + uplift
- \( Q_{MDE} \): Earthquake load effect from MDE ground motions including hydrodynamic and dynamic soil pressure effects

In the case of critical structures i.e. those which are part of a high hazard project whose failure will result in loss of life (USACE 1995), the structure is checked against the extreme event (MCE) to ensure that the collapse is prevented.

### 3.7 Performance Evaluation

Large concrete structures are usually susceptible to damage from tensile stress and cracking of concrete. The actual response of massive concrete structures to earthquake ground motions is very complex. Loading histories and rapid seismic strain rates have an important influence on structural performance. The ultimate tensile strength of concrete is especially sensitive to strain rate. Usually a concrete gravity dam is analysed using linear elastic finite element method (FEM). The results of the FEA in terms of stress demands combined with engineering judgement and past experience are used to assess the performance.

Demand Capacity ratios are used to evaluate the seismic performance of reinforced concrete structures. Depending on whether the response is a displacement controlled (flexure) action or a force controlled action (shear), demands and capacities will be expressed in terms of forces, displacement ductility ratios, or displacement. Under the Linear Dynamic Procedure, performance goals are met when all DCRs (Demand Capacity Ratios) are less than or equal to allowable values established.

### 3.8 Performance assessment methods

#### 3.8.1 Numerical/Analytical methods

Various methods are used for seismic analysis including linear static procedure (LSP), linear dynamic procedure (LDP), nonlinear static procedure (NSP) and nonlinear dynamic procedure (NDP). The most common method is the Linear Dynamic Procedure (LDP) in which seismic
demands are computed by response spectrum or time history analysis procedure. Following are the methods in practice.

a) LSP - Seismic Coefficient Method
This is the simplest method of analysis for the earthquake response estimation of structures. In this method, the earthquake load is treated as a static force and is included in the form of a seismic coefficient with the inertial forces. The seismic coefficient is code specific which is predetermined according to the seismic demands in the site. The analysis is primarily concerned about the stability of the structure against rotation and sliding. The inertia forces acting on the body are arrived upon as the product of structural mass, added mass of water and the effects of dynamic soil pressure, times a seismic coefficient. The magnitude of the seismic coefficient is often taken as a fraction of the peak ground acceleration which is expressed as a fraction of acceleration due to gravity. Being a linear static procedure (LSP) this method does not account for the dynamic characteristics of the foundation-structure-reservoir system and that of the ground motion. Therefore, this method may be applicable for preliminary design purpose.

b) LSP - Equivalent Lateral Force (ELF) Method
The equivalent lateral force method is also a linear static procedure of analysis and is commonly used for seismic design of buildings. A similar method was also developed for concrete gravity dams assuming that the response of the dam to earthquake ground motions is basically in the first mode of vibration which accounts to at least 80 percent to the total vibration response of the structure (Fenves and Chopra 1986). Hence the general deflected shape and the time period are sufficient for the estimation of the equivalent lateral loads needed for the seismic evaluation. The steps involved in the ELF method are as follows:

- The first step is to estimate the period of vibration of the first mode. The time period is given by

\[ T = \frac{2\pi}{\sqrt{\frac{M}{K}}} \]  

(6)

where,

- \( M \) is the total system mass
- (i.e. structure + added hydrodynamic mass* + backfill)
- \( K \) is the lateral stiffness of the structure foundation system

*Added hydrodynamic mass is the inertia of the water adjacent to the dam body
The second step is to determine the spectral acceleration for an equivalent single degree of freedom system. It can be obtained from the site specific or standard acceleration response spectrum for the corresponding time period obtained in the first step. The response spectrum may be code specified or could be computed by a seismic hazard estimation procedure.

Once the spectral acceleration has been determined, the total inertial force or the base shear on the structure due to the design ground motions can be estimated using the following equation.

$$ V_{\text{base}} = \alpha (S_A) \frac{W}{g} $$

where,

$\alpha$ is the base shear participation (obtained from the deflected shape of the structure and the mass distribution)

$S_A$ is the spectral acceleration obtained from the previous step

The analytical model of the structure is represented by a series of lumped masses as shown in the Figure 22 below. The total inertial force (base shear) is then distributed along the height of the structure at the location of each lumped mass. The magnitude of each inertial force is obtained as follows:
\[ F_z = PF \left( \frac{w_z}{g} \right) S_A(j_z) \]  \hspace{1cm} (8)

where,

PF is the mass participation factor for the fundamental mode

\( \frac{w_z}{g} \) is the respective lumped mass

\( \phi_z \) is the value of the mode shape at the respective lumped mass location

Figure 22 Equivalent Lateral Force Method – Step 4 (USACE 2007)

After all the inertial forces are determined the analysis can be carried out in the same way as other static procedures.

c) **LDP - Modal Analysis Response Spectrum Procedure**

This method is very similar to the ELF method but includes higher mode effects and is therefore a linear dynamic procedure (LDP). The peak responses of the linear elastic structures to earthquake ground motions are determined as response spectra. The number of modes required for each analysis varies. All modes with significant contribution to the total response of the structure should be included. The number of modes is adequate if the total mass participation of the modes involved in the analysis is at least 90 percent of the total mass of the structure. Modal analysis is performed with softwares and most software packages are equipped with this facility. This method is more preferable than the ELF method as it uses multiple numbers of modes and the ELF method is based on a single mode shape.

d) **LDP - Modal Analysis Time History Procedure**

This method is similar to the response spectrum modal analysis procedure except that the earthquake ground motion demands are in the form of time histories and the results are in the form of displacement and stress histories. This procedure provides valuable time history information which is not available in the response spectrum modal analysis procedure. This is basically a linear elastic response analysis.
e) **NDP – Nonlinear Direct Integration Time History Technique**

This process involves nonlinear structural response and the direct integration of the equations of motion. A step by step numerical integration is performed to determine forces and displacements for a series of short time increments from the first application of load to any desired time. The motion of the system is evaluated on the basis of assumed response mechanism for each time increment. This process can be equally applicable for linear as well as nonlinear analyses. However the application of nonlinear analysis to concrete hydraulic structures is limited to cases for which experimental or observational evidence of nonlinear behaviour is available and that validity of numerical models have been demonstrated. This method is the most powerful method of analysis but also requires a very good understanding of the structural behaviour and representative ground motion time histories.

f) **NSP – Direct Displacement Based Approach**

All the above procedures described consider earthquake waves in the form of some force. However, recently an earthquake displacement based design approach has been developed. According to (Priestley M.J.N. and Mervyn J. Kowalsky 2002), the fundamental goal of the displacement based design is to obtain a structure which will reach a predetermined displacement when the structure is subjected to an earthquake consistent with the design level event. The structure is designed based on its behaviour at maximum response.

The basic idea behind this approach as suggested by (Priestley M. J. N. 1998) is as follows. Seismic demand is specified as a displacement spectrum. For an expected ductility demand the corresponding damping is estimated. Then with the predetermined design displacement, the effective period \( T_e \) for the maximum displacement response is obtained from appropriate design displacement spectra. The effective stiffness is obtained from the following equation.

\[
T_e = 2\pi \frac{M_e}{K_e}
\]  

(9)

where, \( M_e \) is the effective mass of the structure.

Then the design base shear for maximum displacement response is given by

\[
V_b = K_e \Delta_d
\]  

(10)

Thus the approach aims at designing the structure for the targeted level of performance which is specified as the maximum displacement response.

**g) NSP – Capacity Spectrum Method (CSM)**

The Capacity Spectrum Method compares the capacity of a structure to resist the lateral forces to the demands of earthquake response spectra in a graphical way which facilitates a visual
evaluation of the structural performance when subjected to earthquake ground motion (Freeman S. 2004). In other words, the procedure compares the capacity of the structure which is in the form of a push over curve with the demand on the structure in the form of response spectra (ADRS). The response of the structure is evaluated from the graphical intersection of the two curves. A typical capacity spectrum diagram is shown below.

![Capacity Spectrum Diagram](image)

**Figure 23 Capacity Spectrum Diagram (Freeman S. 2004)**

### 3.8.2 Physical modelling

Physical modelling of the structure mainly is carried out for experimental purposes. The model size is designed based on the numerical models. This topic is however outside the scope of the study.

### 3.9 Previous researches

The prediction of performance of dams during earthquakes is challenging for the following reasons (National Research Council - Panel on earthquake engineering for concrete dams 1990):

- Dams are complicated in shape as determined by their topography of the respective site.
- The response of the dam structure is influenced to a significant extent by the interaction of the dam wall with the reservoir water and the foundation rock. This implies that the foundation rock should not be considered to be 100% rigid and the earthquake induced response of the reservoir water should also be kept under consideration.
- The dam’s response may also be influenced by the variation in intensity and frequency characteristics of the earthquake ground motions over the width and height of the canyon. However this may be a challenge due to the lack of instrumental data to govern the spatial variation of ground motion.

It can be assumed that for low to moderate intensity earthquakes the response of the dam structure will be linear i.e., the resulting deformations of the dam will be directly proportional to the amplitude of the applied ground motion. The performance may not deviate beyond the expected range. However in the case of a major earthquake, the assumption of linear behaviour may result in the calculation of strain values that exceed the tensile strength of the dam’s
concrete. This indicates possible damage and in this case a non-linear analysis would be necessary. However, linear analysis would still be helpful in understanding the nature of dynamic performance of the structure and to determine the onset and location of initial cracking where the tensile capacity of the concrete has been exceeded.

Before the development of modern computing techniques, (i.e. before 1960s) dams were designed in a very simple method which expressed the effect of earthquake ground motions by introducing a seismic coefficient (National Research Council - Panel on earthquake engineering for concrete dams 1990). The compressibility effect of water was not considered and the dam body was considered to be rigid. Dam motion and foundation rock interaction was not considered in evaluating earthquake forces. In the design of gravity dams stress was not considered as a controlling factor, so the design was primarily concerned with satisfying criteria for stability against overturning and sliding (National Research Council - Panel on earthquake engineering for concrete dams 1990). Later as there were modern methods of computing and with the advent of modern computers it became possible to use sophisticated finite element analysis to analyse complicated structures. The study on different aspects was possible for the dam structures. The studies can be categorised roughly into two types which are discussed below.

### 3.9.1 Analytical studies

Many researchers have previously carried out seismic analytical studies for different kinds of dams using finite element procedure. The procedure is developing gradually and is currently an important part of dam design. Some of the research previously carried out is discussed below from various literatures.

Matheu, Hall and Kala (2004) performed a series of dynamic stress analyses on representative critical overflow and non-overflow monoliths of the Folsom Dam in California using the computer code EAGD-84 written and developed by Chopra et al at the University of California at Berkeley. The analyses were performed with due consideration to dam-foundation interaction and water compressibility effects. For the analyses two seismic loading scenarios were considered a) Only the horizontal component of the ground motion and b) both horizontal and vertical components of the ground motion. The ground motions used were the maximum credible earthquake events.

The results were compared with results from response spectrum based method analyses which indicated three critical regions in the non-overflow monolith which encountered very high levels of tensile stresses (peak value of 760 psi or 5.2 MPa). Those critical regions were at the heel of the dam and near the maximum reservoir level on both upstream and downstream faces of the dam. The compressive stresses were however well below the corresponding strength limits.
When dealing with massive structures like dams, the material behaviour is also a matter of concern. During extreme earthquake events, tensile stresses in the concrete may exceed the tensile strength capacity of concrete. Oskouei and Askogan (2007) studied the effect of non-linear behaviour of concrete on the seismic response of concrete gravity dams and as a special case, the Pine Flat dam was chosen. The failure mechanism was addressed with the Drucker-Prager criterion which is a modified representation of the Mohr- Coulomb failure criterion according to Aksoy and Suleyman (2010).

Also the interaction between the dam and the compressible water was included as linear interaction and a rigid foundation was assumed for the finite element modelling. The effect of the sediments and alluvium in the reservoir bottom were not considered. The study concluded emphasizing the tensile cracking as an important factor in nonlinear behaviour of dams which mark the onset of differences in the linear and nonlinear responses of dams. This causes reduction of structural stiffness and hence the increase in displacement. Also as other researchers, they concluded that the critical section of the dam is at the level of discontinuity where cracks originate from both upstream and downstream directions and tend to join thereby resulting into an imminent separation of the upper section of the dam.

As Ghanaat (2004) has stated, when assessing concrete hydraulic structures considering their seismic performance, the widely practiced methodology is checking the state of stress. The acceptance criterion for compressive stresses is that they should not exceed the compressive strength capacity of the concrete by a certain factor. As per American practice, the factor for new dams in design phase is 1.5 whereas for existing dams it is 1.1 (USACE 2003). Generally tensile stresses should not exceed the tensile strength capacity of the concrete however if tensile stresses get set up beyond the strength capacity of the concrete then up to five excursions beyond the limit are accepted. However, the limit to the extent of stress excursion across the section of the concrete or its maximum allowable magnitude is left to the designer to judge as there are no provisions regarding this. This might lead to differing views from different designers. To address this issue, in recent years, the acceptance criteria for dams for performance against seismic loading has been updated relating the extent of damage with the total duration of stress excursion beyond the tensile strength capacity and the demand-capacity ratio. The concept of these acceptance criteria is discussed in detail in Chapter 7.

Ghanaat (2004) has performed an extensive level of time history dynamic analysis for various kinds of concrete hydraulic structures including concrete gravity dams. He has developed and proposed special damage criteria. Engineer Manual EM 1110-2-6051 from USACE (2003) has also proposed the procedures as outlined in Ghanaat’s work.
Ghanaat (2004) investigated the safety evaluation of dams through a failure mode approach which is based on the probable modes of failure, followed by the current practice of stress checks. The target of seismic safety evaluation is to conduct appropriate analyses and evaluations that demonstrate whether or not certain failure modes can develop. Ghanaat investigated overstressing, sliding, joint opening and other failure modes potentially damaging to the structure along with the foundation. He has then proposed performance evaluation criteria demonstrating what extent of overstressing leads to cracking and/or joint opening or ultimate failure itself. The criteria are proposed based on both linear-elastic and nonlinear analyses including the dam-water and dam-foundation interaction effects. The proposed acceptance criterion is not based on stress checks alone; but also it examines stress demand-capacity ratios, accumulated duration of overstress excursions, spatial distribution of stresses, and other factors to determine whether or not nonlinear response in the form of cracking and joint opening could lead to failure mechanisms.

Ghanaat and Rasmussen (2005) performed an evaluation of seismic stability of Claytor dam in Ohio, using both linear and nonlinear time history analysis techniques. The study also included post-earthquake static analysis. When performing nonlinear analysis, the nonlinearity was confined to the cracked base while the rest of the structure was assumed to behave linearly. The non-linearity was simulated by using special interface gap-friction elements. The study results showed that the spillway piers and towers will suffer damage under the MCE (Ms 6.8, PGA 0.22g) but will remain safe with little impact on the safety and operation of the dam.

Arabshahi and Lotfi (2008) investigated the nonlinearities of the dam and foundation interface. They studied various possibilities of natural isolation occurring along the dam–foundation interface during an earthquake. The investigation was based on a plasticity-based formulation in the local stress space of interface elements to model sliding as well as partial opening along the dam base. The study also included effects of base deformation, sliding parameters and foundation flexibility. The results show that sliding generally reduces the maximum amount of tensile principal stresses in the dam body; however, the amount of reduction is generally not large enough to prevent cracking, especially at the upper parts of the dam.

The Koyna Dam of India is probably the most studied dam relating to seismic analysis. This dam is one among the very few dams that suffered an earthquake event at a relatively short distance and incurred substantial level of damage which was beyond the serviceability limit. Lee and Fenves (1998) proposed a plastic damage concrete model for the earthquake analysis of dams which was also applied to analyse the Koyna dam. The proposed model is rate dependent which includes the effects of strain softening. It is based on the assumption that the stiffness of the section reduced with the opening of the cracks but the deformation recovers upon crack closing during the reverse cycles of the ground motion. The behaviour is simulated by a scalar
degradation model. Appropriate assumptions for limits of inelastic strain are made for simulating the displacements owing to large crack openings. The model was used to evaluate the response of Koyna dam in the 1967 Koyna earthquake. The results agree with the formation of cracks at approximately the same location as the real cracks after the earthquake that occurred thus validating the model.

3.9.2 Experimental

**Shake table tests**

Mathieu and Leger (2009) performed shake table tests on sliding response of a gravity dam incorporating the uplift pressure from reservoir water in the University of Montreal in 2008 and have interesting findings. The test was performed on a 1.5m high concrete gravity dam model with a smooth concrete-concrete joint simulating a cold lift joint. The sliding tests were performed with dry, wet and pressurised conditions with varying values of peak water pressures from 15 kPa to 45 kPa. The findings indicated that the uplift pressures reduce the vertical force resultant thereby promoting sliding and the friction coefficient and the related residual displacements are sensitive to the sliding interface conditions. The presence of water in the crack further reduces the sliding stability of the section. The researchers emphasize that this aspect of the sliding mechanism which is usually not discussed in the design phase of the dams must be accounted for so as to develop a high confidence level on the prediction of seismic residual sliding displacements.

A seismic investigation on the famous Three Gorges dam was carried out by Li et al (2005) The study aimed at assessing the performance of the structure against design earthquake events as specified by the seismic design code of China. The study involved a shaking table experiment along with finite element analysis which confirmed the structure’s capability to resist seismic action as desired by the seismic code of China.

Ghaemmaghami and Ghaemian (2007) conducted a shake table test on the Sefid-rud buttress dam relating to the damage caused by the Manjil earthquake in 1990. The researchers tested a 1:30 geometric scaled model of the tallest monolith of the dam on a shaking table with high frequency capacity to investigate the nonlinear seismic response. To simulate the as-built features of the structure construction joints were included in the model. After comparing the experimental results with finite element analysis results, the study concluded that the construction joint effects are important factors in nonlinear seismic analysis of dams.

Earlier Javanmardi, Leger and Tinawi (2005) studied the seismic structural stability of concrete gravity dams considering transient uplift pressures in cracks. The study was aimed at developing a theoretical model for transient water pressure variations along tensile seismic concrete crack with known crack wall motion history. The proposed model was validated by
experimental tests which showed that water can penetrate into seismically induced cracks making them partially saturated. The saturation length is small during the opening of the crack and increases with the closing in the next cycle of ground motion. Water pressure decreases along this length from maximum at the mouth to zero at the end of the crack. The saturation length and the pressure increase with the number of cycles and these factors govern the seismic uplift. The researchers conclude that due to the gravity action of the dam body, the instability from the uplift has very little effect. However, the researchers have emphasized the need of further study on the critical sliding safety factor against downstream sliding owing to seismic tensile opening at the heel.

**Real scale testing**

An experimental study of dam-water-foundation interaction was conducted on Dongjiang Arch dam in China. According to Ghanaat et al (1992) tests were performed in two stages. In the preliminary tests, the dam including the reservoir was subjected to excitations by detonating explosive charges buried in the foundation rock. The systems responses to the explosions were recorded by accelerometers, pressure transducers as well as by three-component seismographs. In the secondary test series, the system was excited by shock waves propagated in the lake from small suspended charges. The experimental analysis concluded that contained explosive detonations were the best means for exciting the dam-foundation-reservoir system for dynamic studies and is in perfect agreement with the analytical procedures.

**3.10 Conclusion**

Following the above studies the seismic analysis of Arun III dam will be carried out in a step by step manner starting from the simplest technique i.e. linear static analysis using the seismic coefficient method. The analysis will be advanced further into linear elastic dynamic analysis using response spectrum modal method of analysis and finally using time history analysis as the most complex method of analysis. However, for the performance assessment against seismic loads, the seismic hazard parameter of the region of the dam-location is necessary to be defined. Since the history of seismic instrumentation in the region is relatively new, very little is known of the seismic hazard of the region. Therefore an attempt was made to establish the seismic hazard for the eastern Nepal Himalayas which is discussed in Chapter 4.
Chapter 4

Hazard Assessment and Representative Design Earthquake Ground Motions

4.1 Introduction

The seismic hazard analysis refers to the estimation of a measure of the strong earthquake ground motion expected to occur at the concerned site. Seismic hazard analysis is carried out for the design of new structures or for the seismic safety assessment of important existing mega-structures like dams, nuclear power plants, high-rise buildings, bridges of long spans and so on. Seismic hazard should not be confused with seismic risk. According to Trifunac and Anderson (1977), hazard relates to the severity of ground motions at a site, regardless to the consequences, whereas the risk refers to the consequences as per Jordanovski et al (1993).

In this chapter the seismic hazard assessment of the Arun river valley is discussed. The seismic hazard assessment was carried out using the Probabilistic seismic hazard analysis approach. The Component Attenuation Model (CAM) approach was used as a comparative tool for the analysis procedure.

4.2 Seismic Hazard Assessment

4.2.1 Probabilistic Seismic Hazard Analysis (PSHA)

According to McGuire (2007) Probabilistic Seismic Hazard Analysis (PSHA) is the evaluation of annual frequencies of exceedance of ground motion levels (typically designated by peak ground acceleration or by spectral accelerations) at a site the result of which is a seismic hazard curve (annual frequency of exceedance plotted against ground motion amplitude) or a uniform hazard spectrum (spectral amplitude plotted against structural period, for a fixed annual frequency of exceedance).

A site may have a large number of possible earthquake magnitudes each of which have the possibility of occurring at a number of locations at varying distances. PSHA incorporates the frequency of occurrence of earthquakes of different magnitudes from the seismic sources, the variability of the earthquake locations, and ground motion attenuation relationships. This approach has been adopted for the considered site located in the Arun river valley in eastern Nepal. An explanation on the procedure as per EM 1110-2-6050 (USACE 1999) is presented below.

a) Procedure

The probability distribution of a complex dependent variable can be derived provided that its relationship with other independent variables is known along with their probability distributions. The above concept on derived distributions was proposed by Cornell (1968).
Based on this and subsequent studies the methodology evolved to the current practice which is described below as explained in EM 1110-2-6050 (USACE 1999).

The ultimate target of a PSHA is the computation of the frequency of exceedance of a certain level of ground motion which can be calculated using the following equation:

\[
\nu(z) = \sum_{N} \left[ \sum_{M} \lambda(m_i) \cdot P(R=r_j|m_i) \cdot P(Z>z|m_i,r_j) \right]_n
\]  

(11)

Where,

\( \lambda(m_i) \) is the frequency of occurrence of events of magnitude \( m_i \) on source \( n \)

\( P(R=r_j|m_i) \) is the probability of an earthquake of magnitude \( m_i \) on source occurring at a certain distance \( r_j \) from the site

\( P(Z>z|m_i,r_j) \) is the probability that an event of magnitude \( m_i \) will exceed the specified level of ground motion \( z \), for each source to site distance

Therefore, the annual frequency of exceeding a certain ground motion level at the site is obtained by summing over all magnitudes and source-to-site distances for that source. Then, the total rate of ground motion exceedance at the site \( \nu(z) \) is obtained by adding the rates for all the sources. An outline of the procedure adopted is illustrated in Figure 25. The components of the above equation are briefly explained below.

- **Rate of occurrence of earthquakes, \( \lambda(m_i) \)**

  The rate of occurrence of earthquakes \( \lambda(m_i) \) is obtained based on earthquake recurrence assessments. This is the first step carried out in PSHA. Recurrence curves express the rate of occurrence of earthquakes equal to or greater than a certain magnitude. The most widely used model for this is the Gutenberg-Richter recurrence relationship given as follows:

  \[
  \log_{10} N(M) = a - bM
  \]  

(12)

where, \( N \) is the number of earthquakes of magnitude \( M \) or greater occurring per year. The constants ‘\( a \)’ and ‘\( b \)’ vary according to the area studied. The cumulative earthquake recurrence relationship is given by the truncated exponential of the Gutenberg-Richter recurrence law which is as follows:

\[
N(M>m) = \alpha(m^0) \cdot \frac{10^{b(m-m^0)} - 10^{-b(m^0-m^0)}}{1.0 - 10^{-b(m^0-m^0)}}
\]  

(13)

where

\( m^0 \) = Lower bound magnitude of interest to the calculation
m" = Maximum magnitude event that can occur on the source
b = Slope or b-value of the recurrence curve
\( \alpha (m^0) \) = Frequency of occurrence of events of magnitude \( m_0 \) and larger
\( \lambda (m_i) \) is obtained by discretizing the recurrence curve into small intervals of magnitude.
\[
\lambda (m_i) = N(m > m_i - \Delta m / 2) - N(m > m_i + \Delta m / 2)
\] (14)

- **Distance probability distribution**

The distance probability distribution, \( P(R = r_j | m_i) \), depends on the geometry of the earthquake sources and their distance from the site; an assumption is made that earthquake occurrence is equally likely on all parts of the source. The function \( P(R = r_j | m_i) \) should also incorporate the magnitude dependence of the earthquake rupture size; larger magnitude earthquakes have larger rupture areas, and thus have the higher probability of releasing energy closer to a site than smaller magnitude earthquakes on the same source.

The probability distribution for distance from the site to the earthquake rupture on the source is computed conditionally on the earthquake magnitude because it is affected by the rupture size.

The equations for the cumulative probability distribution for a linear rupture segment along a linear fault adopted in EM 1110-2-6050 (USACE 1999) given by Der Kiureghian and Ang A. (1977) are as follows:

\[
P(R<r) = 0 \quad \text{for} \quad R<(d^2+L_0^2)^{1/2}
\] (15)

\[
P(R<r) = \frac{(r^2-d^2)^{1/2}-L_0}{L-X(m_i)} \quad \text{for} \quad (d^2+L_0^2)^{1/2} \leq R < (d^2+[L+L_0-X(m_i)]^2)^{1/2}
\] (16)

\[
P(R<r) = 1 \quad \text{for} \quad R > (d^2+[L+L_0-X(m_i)]^2)^{1/2}
\] (17)

where

\( X(m_i) \) is the rupture length in km for magnitude computed by the following equation

\[
X (m_i) = MIN[\exp(-4.654 + 1.189m_i), \text{fault length}]
\] (18)

The variables “d”, “\( L_0 \)” and \( L \) are respectively the perpendicular distance from the site to the fault, the offset from the perpendicular (if any) to the nearest end of the fault and the length of the fault. The variables are illustrated in Figure 24.
The conditional probability function for distance $P(R = r_j | m_i)$ is obtained by discretizing the cumulative distance probability relationship using an appropriate step size.

- Conditional probability of ground motion exceedance

The conditional probability of exceeding a ground motion level for a certain earthquake magnitude and distance is determined from the ground motion attenuation relationships selected for the site. These relationships incorporate the uncertainty in ground motion estimation given ‘$m$’ and ‘$r$’. The function $P(Z > z | m_i, r_j)$ is usually evaluated assuming that the ground motion values are log normally distributed about the median value.

$$P(Z > z | m_i, r_j) = 1.0 - F' \left( \frac{\ln(z) - E[\ln(Z)]}{S[\ln(Z)]} \right)$$  \hspace{1cm} (19)$$

where $E[\ln(Z)]$ is the mean logarithmic ground motion level given by the attenuation relationship and $S[\ln(Z)]$ is the standard error of the log normal ground motion level. The attenuation relationships used are explained in the subsequent section below.
b) PSHA for Arun III dam site

Arun III is a hydropower project planned to be built in the Arun River in eastern Nepal. The proposed dam site is located in the Num village in Sankhuwasabha district, Koshi zone, Nepal. The seismic hazard estimation for the region and the steps followed are explained below.

- **Characterising seismic sources for PSHA**

A seismic source represents a region of the earth’s crust where the characteristics of earthquake activity are recognized to be different from those of the adjacent crust. These sources are
identified on the basis of geological, seismological and geophysical data. A map issued by the Ministry of Physical Planning and Works, GoN was overlayed on the Google earth image of the region which is shown below.

The site in interest has six identified faults within a circle of 200 km radius as shown in Figure 26. The faults and their seismological features are listed in Table 7.

**Table 7 Seismic sources around Arun III dam site**

<table>
<thead>
<tr>
<th>S.N.</th>
<th>Fault ID</th>
<th>Name of Fault</th>
<th>Type</th>
<th>Length (km)</th>
<th>Ms</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>HFT-1.17</td>
<td>Dharan - Madhumalla</td>
<td>R</td>
<td>112</td>
<td>7.2</td>
</tr>
<tr>
<td>2</td>
<td>MBT-2.7</td>
<td>Saptakoshi - Deomai</td>
<td>R</td>
<td>62</td>
<td>6.7</td>
</tr>
<tr>
<td>3</td>
<td>LH-4.11</td>
<td>Taplejung</td>
<td>DS</td>
<td>25</td>
<td>6.7</td>
</tr>
<tr>
<td>4</td>
<td>MBT-2.6</td>
<td>Udaipur – Sunkoshi</td>
<td>R</td>
<td>95</td>
<td>8</td>
</tr>
<tr>
<td>5</td>
<td>HTH-5.10</td>
<td>Xuru Co</td>
<td>N</td>
<td>100</td>
<td>7</td>
</tr>
<tr>
<td>6</td>
<td>HTH-5.11</td>
<td>Pum Qu – Dinggya</td>
<td>N</td>
<td>90</td>
<td>7</td>
</tr>
</tbody>
</table>


**Magnitude frequency relationship**

Earthquake recurrence rates are estimated from seismic, geological and paleoseismic data. In an analysis of the earthquake catalogue of historical seismicity, it is important to translate the data into a common scale of magnitude as the scale used in the ground motion attenuation...
relationship and to account for the completeness in earthquake reporting as a function of time and location. Seismological data were collected from various sources (National Seismological Centre Nepal; National Society for Earthquake Technology Nepal - NSET 2009; Parajuli et al. 2009) which is presented in Table 8.

Table 8 Earthquake events for Magnitude-Frequency study

<table>
<thead>
<tr>
<th>Year</th>
<th>Magntitude</th>
<th>Total Events</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>4.0-4.9</td>
<td>5.0-5.9</td>
</tr>
<tr>
<td>1100</td>
<td>1931</td>
<td>1</td>
</tr>
<tr>
<td>1932</td>
<td>1936</td>
<td>1</td>
</tr>
<tr>
<td>1937</td>
<td>1941</td>
<td>2</td>
</tr>
<tr>
<td>1942</td>
<td>1946</td>
<td>1</td>
</tr>
<tr>
<td>1947</td>
<td>1951</td>
<td>1</td>
</tr>
<tr>
<td>1952</td>
<td>1956</td>
<td>4</td>
</tr>
<tr>
<td>1957</td>
<td>1961</td>
<td>2</td>
</tr>
<tr>
<td>1962</td>
<td>1966</td>
<td>7</td>
</tr>
<tr>
<td>1967</td>
<td>1971</td>
<td>11</td>
</tr>
<tr>
<td>1972</td>
<td>1976</td>
<td>13</td>
</tr>
<tr>
<td>1977</td>
<td>1981</td>
<td>12</td>
</tr>
<tr>
<td>1982</td>
<td>1986</td>
<td>14</td>
</tr>
<tr>
<td>1987</td>
<td>1991</td>
<td>29</td>
</tr>
<tr>
<td>1992</td>
<td>1996</td>
<td>19</td>
</tr>
<tr>
<td>1997</td>
<td>2001</td>
<td>23</td>
</tr>
<tr>
<td>2002</td>
<td>2008</td>
<td>111</td>
</tr>
<tr>
<td></td>
<td>Total Events</td>
<td>241</td>
</tr>
</tbody>
</table>

Since the number of events is not complete, the data was statistically analysed using the method derived from the work of Stepp (1972) assuming that the occurrence rates are constant over time. Earthquakes are grouped in ‘magnitude classes’ and each class is assumed as a point process in time. The sequence of earthquake events is assumed to be a Poisson process. If \( k_1, k_2, k_3, \ldots, k_n \) are the number of events per unit time interval, then an unbiased estimate of the mean rate per unit time interval is given by

\[
rate = \frac{1}{n} \sum_{i=1}^{n} k_i
\]  

(20)

Where \( n \) is the number of unit time intervals. The rate of earthquake exceeding each magnitude using equation (20) is computed and the recurrence relationship is presented in Figure 27.
Ground motion attenuation characterisation for PSHA

Recently developed attenuation relationship for subduction zones by Atkinson and Boore (2003) was used for this purpose. The database used by Atkinson and Boore is extensive with above 2000 records from subduction zones around the world thus making it preferable to other attenuation models for the PSHA. The equation is given as follows:

\[
\log Y = c_1 + c_2 M + c_3 h + c_4 R - g \log R + c_5 sI_{S_e} + c_6 sI_{S_d} + c_7 sI_{S_f}
\]

(21)

\[
R = \sqrt{D_{\text{fault}}^2 + \Delta^2}
\]

\[
\Delta = 0.00724 \times 10^{0.507M}
\]

\[
g = 10^{0.301 - 0.01M} \quad \text{for interface events}
\]

\[
g = 10^{1.2 - 0.18M} \quad \text{for in-slab events}
\]

Where, \(Y\) is the peak horizontal acceleration (cm/s/s), \(M\) is moment magnitude, \(h\) is focal depth (km) and \(D_{\text{fault}}\) is the closest distance to the fault surface (km), \(\Delta\) is near surface saturation factor which accounts for fault geometry, \(c_1-c_7\) are coefficients and \(sI\) is a factor for amplification that corresponds to soil type. Thrust mechanisms are assumed to represent in-slab events if the events occur at depths greater than 50 km. It is impossible to predict the depth of the hypocentre so the magnitude dependent value \(\Delta\) is treated as a measure of the hypocentral depth in the analysis.

c) Return Period for Design Events

For the purpose of design of structures to specified level of earthquake actions, the hazard is usually represented by the return period expressed in years. The Engineering manuals from USACE which are commonly used internationally follow the probabilistic measure which for
major hydraulic structures allows a 50% probability of exceedance in 100 years (which is 145 years) for OBE and a 10% chance of exceedance in 100 years (which is 950 years) for MDE.

\[ T_r = \frac{T_e}{\ln(1-P_e)} \]  

(22)

Where

\( T_e \) is the service life of the structure

\( P_e \) is the probability of exceedance level

For a service life of 50 and 100 years, the return period years for different levels of exceedance probabilities are as follows:

<table>
<thead>
<tr>
<th>Probability</th>
<th>Service life 50 years</th>
<th>Service life 100 years</th>
</tr>
</thead>
<tbody>
<tr>
<td>2%</td>
<td>2475</td>
<td>4949 ( \approx ) 4950</td>
</tr>
<tr>
<td>5%</td>
<td>975</td>
<td>1949 ( \approx ) 1950</td>
</tr>
<tr>
<td>10%</td>
<td>475</td>
<td>949 ( \approx ) 950</td>
</tr>
<tr>
<td>50%</td>
<td>72 ( \approx ) 70</td>
<td>144 ( \approx ) 145</td>
</tr>
</tbody>
</table>

The national building code of Nepal does not specify any provision on return period years for design events and leaves it to the designer to decide reasonably.

As per historical records, the maximum magnitude event experienced within the Nepal Himalayas is the magnitude 8.2 event of 1934. On the basis of archaeological evidences, researchers have agreed that an event of magnitude 8 to 8.5 occurs roughly every 500 years in the Himalayan region (Molnar and Pandey 1988; Jackson and Bilham 1994; Pandey et al 1995; Bilham et al 2001). Therefore the return periods chosen for this study for the operational basis earthquake event is assumed as 448 years and for the maximum design earthquake event as 949 years. For the sake of simplicity the design events will be rounded up here to the nearest tens as follows:

- OBE 450 years return period
- MDE 950 years return period

d) Development of Site Specific Response spectra from PSHA

For developing the site specific response spectra the more straight-forward approach in PSHA is to develop an equal hazard spectrum from the results of the PSHA as recommended by EM 1110-2-6050 (USACE 1999) which is explained below.

PSHAs are conducted for a range of time periods of interest for the structure. When the appropriate exceedance frequency or the return period to be used is specified the spectral
ordinates are read off each hazard curve and are plotted against the vibrational period. The hazard curves developed for 450 years and 950 years return period for eastern Nepal including the Arun river valley are shown in Figure 28.

ADRS plots provide a comprehensive picture regarding the displacement demand of the ground motion against the respective spectral acceleration. Therefore the above UHRS plots were computed into ADRS format. Plots for the same in ADRS format are also shown below for 450 years and 950 years return period events. The plotted ADRS diagrams vary from the conventional ADRS plots that start with a constant acceleration plateau followed by a constant velocity hyperbolic form. Furthermore the ADRS plots do not have the constant displacement line which is a vertical cut-off after the constant velocity segment which is due to the attenuation equation model used. Seismological experts believe that this discord is not unusual in the seismic hazard attenuation models developed for the subduction zones around the world.
e) Disaggregation and significant earthquakes

PSHA provides an overall representation of earthquake hazard as it integrates all the possible earthquake occurrences and ground motions to calculate a combined probability of exceedance which incorporates relative frequencies of occurrence of different earthquakes. However, this representation does not give a picture of a design earthquake in terms of magnitude and distance which is necessary for the selection of representative ground motion time histories for dynamic analysis. Therefore the computed hazard has to be disaggregated into different magnitude and distance events with appropriate contributions. Researchers (Chapman M.C. 1995; Harmsen et al. 1999) have developed models for disaggregation of hazard and proposed mean and modal magnitudes and distances. The mean rate of exceedance corresponding to a particular value of response acceleration at specified probability of exceedances within a specified period of years is the sum of all contributions of mean rate of exceedances from various earthquakes with different magnitude-distance combinations.

From Figure 26 it is evident that there are two fault lines located at significantly close distances to the Arun III dam site. The Taplejung fault (LH - 4.11) and the Udaipur-Sunkoshi (MBT - 2.6) fault have sections as near as 6 km from the proposed dam site. The maximum magnitude potentials of these fault lines as assigned by MoPPW/GoN (2001) are M6.7 and M8 respectively for LH - 4.11 and MBT - 2.6. The hazard disaggregated in terms of magnitude and distance illustrated agreed with the fault locations and their magnitude potentials.

Figure 30 Disaggregation for PGA for 450 YRP event for Arun River valley
The disaggregated mean events with respect to magnitudes and distances contribution plotted for the Arun valley are shown in Figure 30 and Figure 31. As per the conventional practice which was initially proposed by (Chapman M.C. 1995; Harmsen et al. 1999) for finding out the most dominant event for the Arun river valley, a mean rate of exceedance corresponding to the peak ground acceleration for the required years return period hazard is disaggregated into units of magnitudes and distances. For return period events of 450 and 950 years from Figure 30 and Figure 31, it is observed that the most contribution is from magnitude range of M6.5-M8 in the distance ranges 10 km to 30 km. Therefore from the observation, the following recommendations are made in relation to the design earthquakes.

<table>
<thead>
<tr>
<th>Design event</th>
<th>Return period (Yrs)</th>
<th>Magnitude (L)</th>
<th>Distance (km)</th>
</tr>
</thead>
<tbody>
<tr>
<td>OBE</td>
<td>450</td>
<td>6.5 - 7</td>
<td>5 – 10</td>
</tr>
<tr>
<td>MDE</td>
<td>950</td>
<td>7.5 - 8</td>
<td>5 – 15</td>
</tr>
</tbody>
</table>

**4.2.2 Methodology using the Component Attenuation Model (CAM)**

For regions lacking sufficient strong motion data, seismic hazard analysis has usually been a real challenge. The challenge has been addressed by the development of the Component Attenuation Model (CAM) for relevant response spectral relationships for rock sites by Lam Wilson, Chandler and Hutchinson (2000).
The model is based on the fundamental relationship between earthquake ground motion properties and the seismic moment generated at the source of the earthquake. CAM provides a very simple methodology for the construction of the acceleration and displacement response spectra. According to Lam, Wilson, Chandler and Hutchinson. (2000) the methodology is reasonably accurate.

This approach has been included as a comparative tool for the considered site located in the Arun river valley in eastern Nepal. An explanation on the approach is presented below.

a) **Response spectral parameters using CAM**

The proposed response spectral attenuation function is given by the following equation.

\[ \Delta = 0.78 \Delta^* \alpha(M) G(R) \beta(R) \gamma(\text{crust}) \]  

(23)

\[ \alpha(M) = a_1 + a_2 \times (M - 5)^{a_3} \]

\[ \beta(R) = \left( \frac{30}{R} \right)^{C_R} \]

\[ \gamma(\text{crust}) = C_m C_u \]

\[ G(R) = \frac{30}{R} \]

Where,

\( \Delta \) Response spectral parameter (acceleration m/s^2, velocity mm/s or displacement mm)

\( \Delta^* \) Value of the parameter for reference event of magnitude 6 at a source 30 km away

\( \alpha(M) \) Source function

\( \beta(R) \) Function for anelastic whole path attenuation

\( \gamma(\text{crust}) \) Function for mid crust amplification and combined upper crust amplification and attenuation

\( G(R) \) Spherical attenuation factor

\( a_1, a_2 \) and \( a_3 \) Source function parameters

b) **Displacement and Acceleration response spectra modelling methodology**

The ultimate goal of seismic hazard analysis is to construct a response spectrum of the spectral parameters concerned. The Acceleration-Displacement response spectrum is the spectral displacement versus spectral acceleration plot for different vibrational period and it can be
constructed with a simple technique as described by Lam, Wilson, Chandler and Hutchinson. (2000).

A typical ADRS diagram is shown in Figure 32 along with the two corner periods, $T_1$ and $T_2$. The initial flat part of the spectrum ($T < T_1$) is defined by the maximum response spectral acceleration $RSA_{\text{max}}$. Attenuation relationship as given by Lam, Wilson, Chandler and Hutchinson. (2000), equation (23) is used for the determination of $RSA_{\text{max}}$.

![Figure 32: Response spectrum in ADRS format](image)

The corner period $T_1$ can be calculated using the following equation.

$$T_1 = \frac{2\pi}{RSA_{\text{max}} / RSV_{\text{max}}}$$  \hspace{1cm} (24)

The hyperbolic part of the spectrum ($T_1 < T < T_2$) is determined by the following expression:

$$RSA = 2\pi \frac{RSV_{\text{max}}}{T}$$  \hspace{1cm} (25)

The second corner period of the spectrum is determined by the following expression:

$$T_2 = 0.5 + 0.5(M - 5)$$  \hspace{1cm} (26)

Then $RSD_{\text{max}}$ is calculated using equation (27)

$$T_2 = 2\pi \frac{RSV_{\text{max}}}{RSD_{\text{max}}}$$  \hspace{1cm} (27)

The vertical part of the spectrum after the second corner period is defined by the maximum response spectral displacement $RSD_{\text{max}}$. The attenuation relationships developed by Lam, Wilson, Chandler and Hutchinson. (2000) are used for the determination of $RSD_{\text{max}}$. 
c) **Modelling for design applications**

Recurrence curves express the rate of occurrence of earthquakes equal to or greater than a certain magnitude. The most widely used model for this is the Gutenberg-Richter recurrence relationship as previously introduced in section (a).

\[
\log_{10} N(M) = a - bM
\]

The truncated form of the above equation for earthquakes above a certain level of magnitude (usually magnitude 5 below which the ground motions are not of much significance as to the response of structures) is given by the following equation:

\[
\log_{10} N(M) = a - b(M - 5)
\]  \hspace{1cm} (28)

Based on the above equation and assuming a uniform seismicity, Lam et. al. (2000) derived the following equation for an event of a given return period.

\[
M = 5 + \{\log_{10}(2\pi R^2 T_{rb}) - 7 + a\}/b
\]  \hspace{1cm} (29)

For a given design return period and with appropriate seismicity parameters ‘a’ and ‘b’, equation (29) can be used to determine a range of M-R combination sets.

Compared to the de-aggregation procedure carried out in the probabilistic seismic hazard analysis, this procedure of direct determination of the magnitude-distance combination for the associated hazard (which is represented in terms of return period event) is very simple and straight-forward. The M-R combinations listed in Table 10 and 11 are consistent with the disaggregation plots shown in Figures 30 and 31.

**4.3 Seismic Hazard Analysis for Arun river valley, eastern Nepal**

Arun III is a hydropower project planned to be built in the Arun River valley in the Himalayas in eastern Nepal. The proposed dam site is located in the Num village in Sankhuwasabha district, Koshi zone, Nepal. The development of the design response spectra and for the region is explained below.

**4.3.1 Design Response spectra**

Using equations (24), (25), (26) and (27) design response spectra were developed for the concerned region in ADRS (Acceleration Displacement Response Spectra) format. ADRS plots provide a comprehensive picture regarding the displacement demand of the ground motion against the respective spectral acceleration. ADRS format response spectra were developed for different magnitude events (M = 5.5 - 7.5) for a range of source-to-site distances (R = 10 - 70 km) which are shown below.
Figure 33 ADRS diagram for Magnitude 5.5 across different source-to-site distances

Figure 34 ADRS diagram for Magnitude 6 across different source-to-site distances

Figure 35 ADRS diagram for Magnitude 6.5 across different source-to-site distances
4.3.2 Magnitude frequency relationship

Earthquake recurrence equation developed for the PSHA is equally applicable in this case. The earthquake recurrence relationship developed using the Guterberg-Richter model is shown in Figure 27 under a similar heading for the PSHA along with the data in Table 8.

4.3.3 Return periods for design events

As per recorded evidence, the maximum magnitude event experienced within the Nepal Himalayas is the magnitude 8.2 event of 1934. Researchers have agreed that an event of magnitude 8.0 - 8.5 occurs roughly every 500 years in the Himalayan region (Molnar and Pandey 1988; Jackson and Bilham 1994; Pandey et al 1995; Bilham et al 2001). Therefore the
return periods chosen for this study for the operational basis earthquake event is assumed as 450 years and for the maximum design earthquake event as 950 years.

4.3.4 M-R combinations

For the adopted return periods the events with various M-R combinations were determined using equation (29). The set of M-R combinations are listed in Tables 10 and 11 for both the Operational Basis and the Maximum Design events. The Operational Basis event corresponds to 20% chance of exceedance (i.e. 450 years return period) and the Maximum Design event corresponds to 10% probability of exceedance (i.e. 950 years return period) in a service life of 100 years. SD\(_{\text{max}}\), SV\(_{\text{max}}\) and SA\(_{\text{max}}\) were obtained using equation (23) with appropriate values of the parameters as proposed by Lam, Wilson, Chandler and Hutchinson. (2000). PGA of the selected event (M-R combination) was calculated using the following formula by Lam, Wilson, Chandler and Hutchinson. (2000)

\[
\text{EPGA} = \frac{\text{SA}_{\text{max}}}{3}
\]  

(30)

<table>
<thead>
<tr>
<th>Magnitude (L)</th>
<th>R(km)</th>
<th>SD(_{\text{max}}) (mm)</th>
<th>T2 (s)</th>
<th>SV(_{\text{max}}) (mm/s)</th>
<th>T1 (s)</th>
<th>SA(_{\text{max}}) (m/s(^2))</th>
<th>PGA(g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>6.7</td>
<td>8</td>
<td>158</td>
<td>1.35</td>
<td>879</td>
<td>0.22</td>
<td>25</td>
<td>0.86</td>
</tr>
<tr>
<td>7.2</td>
<td>13</td>
<td>170</td>
<td>1.6</td>
<td>807</td>
<td>0.24</td>
<td>21</td>
<td>0.7</td>
</tr>
<tr>
<td>7.3</td>
<td>14</td>
<td>174</td>
<td>1.65</td>
<td>804</td>
<td>0.24</td>
<td>21</td>
<td>0.7</td>
</tr>
<tr>
<td>7.4</td>
<td>15</td>
<td>179</td>
<td>1.7</td>
<td>802</td>
<td>0.25</td>
<td>21</td>
<td>0.7</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Magnitude (L)</th>
<th>R(km)</th>
<th>SD(_{\text{max}}) (mm)</th>
<th>T2 (s)</th>
<th>SV(_{\text{max}}) (mm/s)</th>
<th>T1 (s)</th>
<th>SA(_{\text{max}}) (m/s(^2))</th>
<th>PGA(g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>8.2</td>
<td>12</td>
<td>427</td>
<td>2.1</td>
<td>1620</td>
<td>0.27</td>
<td>38</td>
<td>1.2</td>
</tr>
<tr>
<td>8.5</td>
<td>15</td>
<td>418</td>
<td>2.25</td>
<td>1504</td>
<td>0.28</td>
<td>34</td>
<td>1.1</td>
</tr>
<tr>
<td>8.6</td>
<td>17</td>
<td>392</td>
<td>2.3</td>
<td>1387</td>
<td>0.29</td>
<td>31</td>
<td>1.0</td>
</tr>
</tbody>
</table>

4.4 Comparison with previous studies

For the validation of the computed hazard the peak ground acceleration values for three different return periods were computed and compared with available values from other researches in the Himalayan region.

The earliest attempt on seismic hazard study in the Himalayan region was made by Khattri et al. in 1984 and for most of the Himalayan region the study results show a hazard value of 0.7g for a 475 years return period event. In 1993 BECA Group Ltd carried out a seismic hazard study for the Government of Nepal in order to develop the national building code of Nepal. Due to paucity of any reliable seismic data the outcome from the study was mentioned only as
guidelines. That was the first attempt of any seismicity related study in Nepal. Bhatia et al under GSHAP investigated the seismic hazard for the Himalayan region using the probabilistic approach which depicts the second lowest value for a 475 years return period event among the six different values presented in Table 12. Also, Nath et al (2005) carried out a study in the neighbouring Sikkim Himalayas using strong motion seismometry and calculated the PGA for 10% probability of exceedance for 50 years (i.e. a return period of 475 years) which for various locations around Sikkim which is also shown in Table 12. The seismicity in the Northeast Indian Himalayas was studied by Das and co-workers (2006) preparing uniform hazard seismic maps for an exposure time of 100 years and a 50% probability of exceedance. For comparison with the other available values from previous researches the hazard values for return periods of 950 years, 475 years, 100 years and 50 years were calculated by both CAM and PSHA approaches.

Table 12 PGA values comparison for selected return period events

<table>
<thead>
<tr>
<th>Methodology</th>
<th>PGA values (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>950</td>
</tr>
<tr>
<td>This study</td>
<td></td>
</tr>
<tr>
<td>CAM</td>
<td>1.1</td>
</tr>
<tr>
<td>PSHA</td>
<td>0.77</td>
</tr>
<tr>
<td>BECA 1993 (Nepal)</td>
<td>0.6</td>
</tr>
<tr>
<td>Nath et al. 2005 (Sikkim)</td>
<td></td>
</tr>
<tr>
<td>GSHAP (Bhatia et al) 1999</td>
<td></td>
</tr>
<tr>
<td>Mahajan et al 2009</td>
<td></td>
</tr>
<tr>
<td>Khattri et al 1984</td>
<td></td>
</tr>
<tr>
<td>Das et al 2006 (Assam)</td>
<td></td>
</tr>
<tr>
<td>From other researchers</td>
<td></td>
</tr>
</tbody>
</table>

The two methods used were compared against each other for lower magnitude ranges and the peak ground velocity values were very much in agreement. The values from CAM were on the higher side for higher return period events. CAM is a good approach for magnitudes up to M7 as mentioned by Lam, Wilson, Chandler and Hutchinson (2000) period events but was not developed for M >7.0 events.

4.5 The Maximum Credible Earthquake (MCE) Event

As discussed in section 3.6 (a), the MCE is the greatest reasonably sized event expected from a specific source which is a determined on the basis of seismological and geological evidence. Therefore the MCE is necessarily a deterministic event. Since the Arun valley may be affected by earthquakes from various sources in the vicinity (Refer Figure 26) each with their respective fault mechanism, maximum magnitudes and distance from the site. According to researchers (Molnar and Pandey 1988; Bilham R. 1995; Pandey et al 1995; Upreti 1999; Bilham et al 2001;
Dahal R. K. (2006), the Himalayan region experiences an event of magnitude of 8.0-8.5 roughly every 500 years. Also according to seismological and archaeological evidences magnitudes as high as 8.5 have occurred in the past (National Society for Earthquake Technology Nepal - NSET 2009).

To calculate MCE a return period of 1950 years would be suitable (5% probability of exceedance in 100 years). However due to scarcity of appropriate data the prediction of lower probability values was difficult by the probabilistic approach and the help of historical records and geological data was sought for estimating MCE. The maximum credible earthquake in the vicinity of the site was specified based on the faults data from the GoN, Ministry of Physical Planning and Works. An area of approximately 200km radius around the proposed dam site was considered for the seismic hazard study. The Department of Development and Building Construction under the Ministry of Physical Planning and Works, GoN has published a map of Nepal demarking the recognized faults within the Nepal Himalayas (Refer Figure 26). The map shows that the Arun valley consists of faults with a maximum magnitude potential as high as M8 (Refer Table 7) within a distance as near as 20km from the site (Udaipur-Sunkoshi fault). Therefore the MCE is adopted as an event of the following order which can be expected to have a PGA at least equal to that of the MDE level.

Magnitude       M 8 – M 8.5
Distance         10 – 15 km

4.6 Discussion and Conclusion

The uniform hazard spectra for three different probabilities of exceedance were calculated using the attenuation equations developed by Atkinson and Boore (2003). The same curves were also plotted in the ADRS format and a further disaggregation of the hazard curves was carried out to point out the events of the hazard level in terms of magnitude and distance from site.

For a comparison the hazard was assessed with the methodology using CAM. It is a straightforward method based on the concept of uniform distribution of seismicity from a point source which gives fairly concordant results in the low to moderate seismicity range but for high magnitude ranges (above M7) the results show a higher deviation as mentioned by Lam, Wilson, Chandler and Hutchinson. (2000). The CAM approach was not developed for regions of high seismicity with known sources and therefore the M-R combinations obtained may be on the higher side thereby overestimating the hazard for magnitudes above 7.

Although the CAM approach is simpler and easier to follow than the PSHA approach the hazard as estimated from CAM for region of high seismicity seems to be on the higher side and therefore for the rest of the research, the seismic hazard as estimated by PSHA will be adopted. The final values are tabulated below.
Table 13 Design earthquake events for the Arun river valley

<table>
<thead>
<tr>
<th>Design event</th>
<th>Return period (Yrs)</th>
<th>Magnitude (M)</th>
<th>Distance (km)</th>
<th>PGA (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>OBE</td>
<td>450</td>
<td>6.5 – 7</td>
<td>5 – 10</td>
<td>0.5</td>
</tr>
<tr>
<td>MDE</td>
<td>950</td>
<td>7.5 – 8</td>
<td>5 – 15</td>
<td>0.77</td>
</tr>
<tr>
<td>MCE</td>
<td></td>
<td>8 – 8.5</td>
<td>10 – 15</td>
<td>≥0.77</td>
</tr>
</tbody>
</table>
Chapter 5
Selection of Representative Ground Motions

For the selection of ground motions a number of guidelines have been published by different organisations. The methodology presented in EM 1110-2-6051 (USACE 2003) is a widely adopted method. Bulletin 72 published by (ICOLD 1989) also provides guidelines specifically for the design of large dams. However, recommendations in the methodology have been criticised on statistical/mathematical grounds by seismologists (Shrikhande M. and Basu S. 2005) and therefore the USACE method has been adopted here.

5.1 Maximum Credible Earthquake and Maximum Design Earthquake

According to the definition provided in ER 1110-2-1806 (USACE 1995; USACE 1995) the MCE is the greatest earthquake that can reasonably be expected to be generated by a specific source on the basis of seismological and geological evidence. For critical structures, the MCE event must satisfy the damage control performance limit state to ensure that there is no risk of collapse. Critical structures are defined in ER 1110-2-1806 (USACE 1995) as structures that are part of a high hazard project whose failure will result in loss of life. The Maximum Design Earthquake is the maximum level of ground motion for which a structure is designed or evaluated. As a minimum for other than critical structures, the MDE ground motion has a 10 percent chance of being exceeded in a 100 year period, (or a 950 year return period). It can either be a probabilistic or a deterministic event.

The peak horizontal ground acceleration value corresponding to 10% probability of exceedance in 100 years was determined as 0.77 g which is adopted as the MDE event. The resulting 5-percent-damping response spectrum is shown in the following figure. As discussed in section 4.5, the MCE event is estimated to be with a minimum level of 0.77g PGA (M8-M8.5, R15-R20). Therefore the MCE and the MDE events seem to converge regarding their M-R combination measures (Refer Table 13). Therefore the MDE may be assumed to be equivalent to the MCE. The response spectrum obtained in the previous chapter (MDE – 950 years return period in Figure 29) is reproduced here in Figure 38.
5.2 Selection of Ground Motion Time Histories

5.2.1 Use of recorded and synthetic time-histories

As outlined by EM 1110-2-6051 (USACE 2003), when available for the parameters and conditions of a design earthquake, real earthquake time-history records are preferable when subject to further scaling or modification for use in seismic analyses. However, there may be cases when there are very few or no time histories recorded during earthquakes similar to the design events. Specifically in the Himalayan context where the seismic instrumentation started only from the 1980s, the lack of real earthquake ground motion records that match the design event conditions has always been a challenge. The lack of earthquake records for large-magnitude, near-source events, is yet to be addressed throughout the seismic regions of the world.

In cases where suitable records are lacking, it's customary to use synthetic time histories developed using numerical modelling techniques that simulate the design event conditions (earthquake rupture, source-to-site seismic wave propagation and path properties). Various computer codes have been developed to modify real records as per the design requirement (Spectrum matching in time/frequency domains) or to generate artificial records simulating the target design event conditions and parameters including: RSPMATCH (Abrahamson N.A. 1993), TARSCTH (Papageoegiou et al. 2002), RASCAL (Silva and Lee 1987), SIMQKE (Vanmarcke E.H. and Gasparini D.A. 1976) and GENQKE (Lam et al. 2000).

5.2.2 Criteria for selection

The design or site specific response spectra (in the form of Uniform Hazard Response Spectrum, UHRS) provide us the measure of intensity corresponding to a certain hazard level. This should reflect the magnitude, distance, site condition and other parameters that control the ground motion characteristics (Stewart et al. 2001). Candidate time histories should have been recorded in a tectonic environment similar to that for the design earthquake (USACE 2003).
This is desirable because the geology of the region and mechanism of faulting affects the frequency content of the strong ground motion records. Also the earthquake magnitude has to match the magnitude of the target design earthquake. When selecting the earthquake records, it is desirable to use earthquake magnitudes within 0.25 magnitude units of the target magnitude (Stewart et al. 2001). Source-to-site distance is also an important parameter to be matched when selecting the earthquake time-histories because the distance from the source is also a deciding factor in the level of shaking. The intensity of shaking decreases with the increase in distance from the source.

The subsurface conditions for the candidate strong motion records should also match with the conditions for the design earthquake (e.g. rock, soil, etc.). Records from firm soil sites can substitute for those recorded on rock and vice versa, if a sufficient number of recordings for the same site condition are not available, as per EM 1110-2-6051.

5.3 Selection and Scaling approach

5.3.1 Number of time-histories

The European code for earthquake design, Eurocode 8 (BSI 1998) has mentioned that at least seven sets of records should be used for time history analyses. According to the American practice, there are different approaches for linear and nonlinear analyses. The procedural guidelines from the National Institute of Standards and Technology (Idriss I.M. 1993) suggests that at least three time histories and as many as seven time histories should be used in the analysis. EM 1110-2-6051 (USACE 2003) states that for linear dynamic analyses, at least three time histories should be used for each design earthquake and for nonlinear analyses at least five time histories should be used for each component of motion. Fewer time histories are required for linear analysis because the dynamic response of a linear structure is determined largely by the response spectral content of the motion, whereas the response of a nonlinear structure may be importantly influenced by the time domain character of the time history (e.g. shape, sequence, and number of pulses) in addition to the response spectrum characteristics. Since these time domain characteristics may vary greatly for time histories having similar spectral content, more time histories are required for nonlinear.

5.3.2 Scaling approach

The selected ground motions should be scaled to the approximate level of the design response spectrum. The EM 1110-2-6051 states that the scaling factor should be chosen such that the sum of the differences of the logarithms of the spectral accelerations of the scaled time-history and the logarithm of the design response spectrum is approximately equal to zero over the period range of significance to structural response. Apart from this numerical fit, a visual fit is also desired between the plot of the response spectrum of the scaled time-history along with the
design response spectrum. Furthermore, over the defined period range of significance, the mean of the selected records’ spectra are desired to be not more than 15 percent lower than the design response spectrum at any period. Also, the average of the ratios of the mean spectrum to the design spectrum should be equal to or greater than unity (USACE 2003) in the defined period range of significance to structural response.

5.4 Selected ground motions

5.4.1 Design earthquake from seismic hazard analysis

The Maximum Design Earthquake as obtained from disaggregation of the seismic hazard analysis result is an event of magnitude 7.5 to 8 at a distance of 5 to 15 km which corresponds to a return period of 950 years or a hazard level of 0.77g.

<table>
<thead>
<tr>
<th>Design event</th>
<th>Return period (Yrs)</th>
<th>Magnitude</th>
<th>Distance (km)</th>
<th>PGA (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>MDE</td>
<td>950</td>
<td>7.5 - 8</td>
<td>5 – 15</td>
<td>0.77</td>
</tr>
</tbody>
</table>

5.4.2 Tectonic environment and site condition

The concerned site lies in the Himalayan fold belt. The region is a convergent plate boundary of two continental crusts and therefore the representative accelerograms were selected from the same region or a similar tectonic environment. The Uttarkashi earthquake record is from a Himalayan earthquake and the Manjil earthquake is from a similar region in Iran whilst the Landers earthquake and the Chi-Chi earthquake are from high seismicity regions. The concerned site is a rock site and therefore the records were chosen specifically from rock sites from the strong motion data sources.

5.4.3 Selected records and scaling

Five sets of time histories were finalised for the MDE. Four of the time histories were selected from the representative convergent boundary earthquakes from around the seismic regions of the world including the Himalayas. The fifth record was artificially generated using GENQKE, a FORTRAN based computer code for generating synthetic earthquake records, developed at the University of Melbourne by Lam, Wilson, Chandler and Hutchinson (2000).

Special care was taken to match the seismological parameters of the records with the design earthquake. Among the real earthquake accelerograms, the records from the Landers earthquake and Uttarkashi earthquake were from rock sites whereas the records from the Chi-Chi earthquake and the Manjil earthquake are from stiff soil sites (with Chi-Chi site being enlisted in the COSMOS database as site class C, which translates to a stiff soil site).
The accelerogram records were scaled to provide a desired degree of fit with the design spectrum or the target spectrum (i.e. Figure 38), over the significant period range to the structural response. The scaling multiplier values along with the various selected earthquakes records' seismological parameters are shown in Table 14. The real earthquake time history records were downloaded from the virtual data centre of COSMOS (http://www.cosmos-eq.org/). The five selected accelerograms are plotted in Figure 39 (a)-(e). The accelerograms were processed using Seismosignal v3.3.0, a signal processing software from (SeismoSoft Ltd 2010).

Table 14 Details of the selected representative ground motions

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Unit</th>
<th>Landers*</th>
<th>Chi-Chi*</th>
<th>Uttarkashi*</th>
<th>Manjil*</th>
<th>GENQKE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Record/Station</td>
<td></td>
<td>Joshua</td>
<td>Taichung</td>
<td>Bhatwari</td>
<td>Abbar</td>
<td>Synthetic</td>
</tr>
<tr>
<td>Date</td>
<td></td>
<td>28/06/1992</td>
<td>20/09/1999</td>
<td>20/10/1991</td>
<td>20/06/1990</td>
<td>-</td>
</tr>
<tr>
<td>Magnitude</td>
<td>Mw</td>
<td>7.3</td>
<td>7.6</td>
<td>7</td>
<td>7.5</td>
<td>7.5</td>
</tr>
<tr>
<td>R</td>
<td>Km</td>
<td>10</td>
<td>11.4</td>
<td>21.7</td>
<td>12.6</td>
<td>10</td>
</tr>
<tr>
<td>Soil type</td>
<td></td>
<td>Rock</td>
<td>Stiff</td>
<td>Rock</td>
<td>Stiff</td>
<td>Rock</td>
</tr>
<tr>
<td>Duration</td>
<td>seconds</td>
<td>80</td>
<td>160</td>
<td>36</td>
<td>53</td>
<td>21</td>
</tr>
<tr>
<td>Scale multiplier</td>
<td></td>
<td>1.8</td>
<td>0.7</td>
<td>2.25</td>
<td>1.48</td>
<td>-</td>
</tr>
</tbody>
</table>

*http://www.cosmos-eq.org/

(a) Uttarkashi earthquake record after scaling

(b) Manjil earthquake record after scaling
Gravity dams are usually massive structures and therefore exhibit high structural stiffness characteristics. The non-overflow section for the Arun III dam has a height of 65m and is 65.7m wide at the base which gives the structure a massive size. A modal analysis of the section was carried out in ANSYS which showed that the first natural period of vibration of the unit section
(width of 1m into the plane of the paper) was 0.132 sec. Therefore the period range of significance to the structure has been assumed as 0 s - 0.14 s.

The spectra of the selected scaled accelerograms were checked for their fit with the target design spectrum (Figure 38) in the significant period range of interest for the structure. The spectra are plotted in the traditional response spectral acceleration versus period format in Figure 40 (a) and (b), whilst in ADRS format in Figure 40 (c). The spectra show a good fit over the significant period range for the structural response of the Arun III dam which is shown in Figure 41 (b) although the Chi-Chi and the Landers spectra are clearly conservative at the higher periods.
A plot of the mean of all the scaled spectra with the target spectrum was also plotted and is shown in Figure 41 and is within ±15% of the target spectrum.

To illustrate the procedure the Landers earthquake record is shown in Figure 42 with the unscaled and the scaled records’ response spectra compared to the target spectrum in the period range of significance. The procedure was simply to multiply the time history acceleration values with a multiplier such that the PGA value of the record is equal to the PGA value of the target spectrum over the period range of 0.01 to 0.14 seconds (as shown in Figure 42).
Figure 42 Scaled and unscaled records of Landers earthquake in the form of response spectra

5.5 Conclusion

This chapter presents the selection and comparison of the four real and one synthetic accelerograms selected to be representative of the earthquake ground motions for the dynamic analysis of the Arun III dam structure. The ground motions were selected from similar tectonic environment and within comparable range of magnitude and distance as defined by the seismic hazard assessment. The real records were scaled satisfying all necessary criteria for scaling. The next step in the study was to prepare the finite element model of the dam and to perform a preliminary performance assessment which is described in Chapter 6.


Chapter 6

Finite Element Modelling and Preliminary Seismic Analysis

6.1 Introduction

Finite element method is a very powerful method to deal with engineering problems for which exact solutions cannot be obtained. It is a numerical procedure in which the structure is discretized into a finite number of sub-domains called elements. Each of the elements is assigned to an element matrix and all the element matrices are assembled to obtain the global matrix. Then the boundary conditions are imposed and the equations are solved. The structural models need to be developed with careful consideration to the geometry, stiffness and mass distribution. All these attributes are related to the dynamic characteristics of the structure. As always, engineering experience and judgement plays an important role in the preparation of a satisfactory model that reflects simplicity as well as the proper representation of the dynamic behaviour of the structure.

When carrying out a finite element analysis the behaviour of the structure is an important parameter to be satisfied by the model. Also if the structure can be represented by a simplified two-dimensional model then the expense of time with a three-dimensional model can be avoided. The following section discusses the behavioural aspect of the structure when subjected to various boundary conditions suggesting the finite element model suitable for the analysis.

6.2 Numerical modelling

6.2.1 Expected Structural Behaviour

A gravity dam is a massive structure and therefore its construction is staged into a step by step procedure and therefore joints are inevitable. Depending upon the extent of the dimensions of the structure, there may be joints along the axis of the dam, across the dam axis as well as along the height of the dam. However, for long conventional concrete dams with transverse contraction joints and without keyed joints, a two-dimensional model is considered to correctly represent the structural behaviour (USACE 1995). The Arun III dam as per its design is a 155 m long structure of more or less a consistent cross section for which the assumption of plane strain is valid. Therefore, it may be represented by a planar 2-D model with its deformations restricted in a single plane.

However nonlinear behaviour is expected at the base of the dam-foundation interface when the seismic load is high. Also, cracks are expected to develop near the base of the dam as well as near the maximum reservoir level as depicted earlier in Figure 17.
6.2.2 Development of the Finite Element Model

If one dimension is very large compared to the other directions with all the major forces in the direction normal to it and the principal strain in the direction of the longest dimension can be assumed as zero, then the state of strain is called a plane strain condition. In this case, though all principal stresses are non-zero, the principal stress in the direction of the longest dimension can be disregarded for calculations. Thus, allowing a two dimensional stress analysis. With the assumption that the dam is restrained along its axis (fixed at the abutments) it can be assumed that every slice taken from the dam section behaves in the same manner under the action of the reservoir water load which acts in the direction normal to the longest axis. Thus the problem can be idealized as a plane strain problem. Throughout the study a unit length (1m in the longitudinal direction) of the dam was considered for the calculations.

The dam cross section was idealized as a series of four-node solid elements with plane strain idealization. The idealized model for excitation along the upstream-downstream direction and vice versa is shown in Figure 43. At each cross-section discontinuity, area partitioning was introduced to generate elements having uniform cross-section properties. In addition, midpoint nodes and a node at the water pool elevation were also provided for better accuracy. Except for the nodes at the dam base, which are fixed, all other nodes include one translation and one rotational degree of freedom. The model consists of 709 elements and 769 nodal points with a total of 1538 degrees of freedom.

Figure 43 Arun III dam finite element model
6.2.3 Assumptions

a) Structural mass and section parameters

In the finite element analysis, the element stiffness properties and lumped masses are computed from the cross-section area, mass, and moments of inertia. The cross-section properties at each level of discontinuity are computed using the dimensions provided in Figure 44.

![Figure 44 Cross section of the nonoverflow monolith of Arun III dam](image)

For important structures in Nepal the commonly used grade of concrete is above 30 MPa. Therefore, a characteristic compressive strength capacity of 35 MPa was assumed for the analysis purpose with a static modulus of elasticity of 30 GPa and a Poisson’s ratio of 0.2. The dynamic modulus was calculated using the equation from the British Standard BS 8110-2:1985 (BSI 2001) which is as follows

\[ E_s = 1.25E_D - 19 \]  

Rearranging which we obtain the following equation

\[ E_D = \frac{E_s + 19}{1.25} \]  

Where \( E_s \) is the static modulus of elasticity and \( E_D \) is the dynamic modulus of elasticity.

The static modulus of elasticity as suggested by the ACI Committee 318 (2010) is a function of the compressive strength of the concrete as represented by the following equation.

\[ E_s = 0.043w^{1.5}\sqrt{f_{c}'} \text{MPa} \]  

Where, \( w \) is the density of the concrete in kg/m\(^3\)

The tensile strength capacity of the concrete was computed using the following equation which is based on a rule of thumb:

\[ f_t = 0.08f_{c}' \]  

Where \( f_{c}' \) is the compressive strength of concrete.
The value obtained from (34) is close to the values obtained from other formulae commonly in use viz. (ACI : 3 MPa). The CEB-FIP Model Code for Concrete Structures (1990) specifies upper and lower bounds for the characteristic tensile strength according to which the range is 2.2MPa and 4.2MPa.

The Poisson’s ratio was adopted from EM 1110-2-6051 (USACE 2003). The input parameters used in the analysis for defining the material of the dam model are as follows:

For concrete gravity dams it is a usual practice to adopt 5% damping in structural analysis, hence 5% damping is adopted for all the analyses.

- Material: Concrete
- Characteristic compressive strength of the concrete: 35 MPa
- Tensile strength capacity of concrete: 2.8 MPa
- Static Young’s Modulus of Elasticity of concrete: 30 GPa
- Dynamic Young’s Modulus of Elasticity of concrete: 40 GPa
- Static Poisson’s ratio of concrete: 0.2
- Dynamic Poisson’s ratio of concrete: 0.192
- Mass density of concrete: 2400 kg/m$^3$
- Mass density of water: 1000 kg/m$^3$
- Damping: 0.05

b) Hydrostatic Pressure

The water pressure always acts normal to the surface. In the case of inclined face of the dam, the pressure is computed as vertical and horizontal components. The equation as referred from section 3.3 is given below

$$P_{ii} = \frac{1}{2} \gamma w H^2$$

c) Added hydrodynamic mass

Due to the horizontal acceleration of the foundation and the dam there is an instantaneous hydrodynamic pressure exerted against the dam in addition to hydrostatic forces. In the analysis the added hydrodynamic mass model developed by Westergaard (1933) was used. The equation as referred from section 3.3 is given below

$$p = \frac{7}{8} \rho \sqrt{h(h-z)\ddot{u}}$$

d) Seismic load

For the linear static analysis which is adopted as the preliminary level of analysis the seismic load is primarily accounted for as a coefficient multiplied by the weight of the dam. However for subsequent analyses in the dynamic level the seismic load is accounted for by the application
of a suitable response spectrum or time history record of a response parameter (displacement, velocity or acceleration). These methods are used later in subsequent sections. Apart from the seismic action on the dam body itself there is a seismic inertia force exerted by the mass of water which is adjacent to the dam which is called the hydrodynamic pressure is discussed above.

6.3 Preliminary Analysis

As the preliminary step in seismic analysis the model was first analysed with the linear static approach using the seismic coefficient method. The first study was to find the optimum mesh density for the structural model.

In the next stage the model was analysed with three different boundary conditions separately. The sensitivity analysis with the mesh density and the various loading conditions are discussed below.

6.3.1 Model validation and sensitivity analysis with mesh density

To get a comprehensive idea of the effect of fineness of mesh, four different meshes were prepared. The values of stresses on the same horizontal plane were compared for all four. The meshes were obtained after dividing the area into different sections and meshing them separately as per the required fineness. The different mesh resolution with the number of elements is presented below:

<table>
<thead>
<tr>
<th>Number of elements</th>
</tr>
</thead>
<tbody>
<tr>
<td>264</td>
</tr>
<tr>
<td>759</td>
</tr>
<tr>
<td>2,490</td>
</tr>
<tr>
<td>27,373</td>
</tr>
</tbody>
</table>

Load and Boundary Conditions

The loads and constraints defined for the model are as follows:

1) Acceleration due to gravity = 9.81 m/s²

2) Hydrostatic Load = Linear variation from zero at the maximum water level to 618 kN/m at the bottom of the reservoir.

3) Base of the dam is fixed.

4) For all the calculations the model of the dam was assumed to be 1m thick in the direction perpendicular to the plane of the page.
Model validation

For verification of the results obtained, a horizontal plane was considered at a depth of 31 m from the top of the dam. The plane is shown in Figure 45.

\[ \sigma = \frac{\sum N}{A} \pm \frac{\sum M' y}{I} \]  \hspace{1cm} (35)

Where,

\( \sigma \) = Normal stress on the horizontal plane under consideration

\( \sum N \) = resultant vertical force from forces above the horizontal plane

\( A \) = area of horizontal plane considered

\( \sum M' \) = summation of moments about the centre of gravity of the horizontal plane

\( y \) = distance from the neutral axis of the horizontal plane to where \( \sigma \) is desired

\( I \) = moment of inertia of the horizontal plane about its centre of gravity

For a point in the upstream and downstream edges, the equation reduces to

\[ \sigma = \frac{\sum N}{T} \left(1 \pm \frac{6e}{T} \right) \]  \hspace{1cm} (36)

where,

\( T \) = Length of the plane considered

\( e \) = eccentricity of the resultant = \( \frac{\sum M'}{\sum N} \)
For the considered horizontal plane, the loads acting on the plane are as follows:

- Hydrostatic load upto the level of the plane
- Weight of the concrete above the plane

The load is calculated in the following table (Refer Figure 45 for the loads)

**Table 15 Loads acting on the plane**

<table>
<thead>
<tr>
<th>Load</th>
<th>N +</th>
<th>H +</th>
</tr>
</thead>
<tbody>
<tr>
<td>Units</td>
<td>kN</td>
<td>kN</td>
</tr>
<tr>
<td>Water (H)</td>
<td>..</td>
<td>-4125</td>
</tr>
<tr>
<td>N1</td>
<td>+9225.6</td>
<td>..</td>
</tr>
<tr>
<td>N2</td>
<td>+6048</td>
<td>..</td>
</tr>
<tr>
<td>N3</td>
<td>+2262</td>
<td>..</td>
</tr>
<tr>
<td>ΣN</td>
<td>+17535.6</td>
<td></td>
</tr>
</tbody>
</table>

**Table 16 Moment relative to the centroid of the section**

<table>
<thead>
<tr>
<th>Load element</th>
<th>Moment Arm</th>
<th>Moment +</th>
</tr>
</thead>
<tbody>
<tr>
<td>Units</td>
<td>m</td>
<td>kNm</td>
</tr>
<tr>
<td>H</td>
<td>9.7</td>
<td>-39876</td>
</tr>
<tr>
<td>N1</td>
<td>12.5</td>
<td>115320</td>
</tr>
<tr>
<td>N2</td>
<td>0.3</td>
<td>1814</td>
</tr>
<tr>
<td>N3</td>
<td>-10.0</td>
<td>-22695</td>
</tr>
<tr>
<td>ΣM’</td>
<td></td>
<td>54562.99</td>
</tr>
</tbody>
</table>

∴ \( e = \frac{54563}{17536} = 3.1 \text{ m} \) i.e. eccentricity is towards the upstream of the centroid

Now for the upstream and downstream edges the normal stress is given by

\[
\sigma_z = \frac{\Sigma N}{T} \left(1 \pm \frac{6e}{T}\right)
\]

Substituting the values we get,

∴ \( \sigma_{zu} = 0.70\text{MPa} \)

and \( \sigma_{zd} = 0.23\text{MPa} \)

Now from the Finite Element Analysis we have the following results:
Table 17 Comparison of FEA results with manual calculation

<table>
<thead>
<tr>
<th>Normal Stresses*</th>
<th>Manual Calculation</th>
<th>FEA Result (264 Elements)</th>
<th>FEA Result (759 Elements)</th>
<th>FEA Result (2490 Elements)</th>
<th>FEA Result (27373 Elements)</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \sigma_{ZU} )</td>
<td>0.70</td>
<td>0.66</td>
<td>0.68</td>
<td>0.69</td>
<td>0.70</td>
</tr>
<tr>
<td>( \sigma_{ZD} )</td>
<td>0.23</td>
<td>0.25</td>
<td>0.22</td>
<td>0.21</td>
<td>0.20</td>
</tr>
</tbody>
</table>

* All values of stresses are in MPa.

A comparison of the values obtained from the two methods is illustrated in Figure 46. The values obtained from different mesh resolutions seem to be in agreement with the manually obtained results.

![Distance from upstream face (m)](image)

**Figure 46** Comparison of the FEA and Manual calculation results

**Conclusion**

Therefore the values are comparable and the model is validated. For further analysis of the structure the model with 759 elements is used as the results are in good agreement with the manual calculations and it is anticipated that this model will provide the results with an optimum expense of time during the finite element calculations for the linear dynamic analyses which will be carried out in the later stages of this study.

**6.3.2 Sensitivity with foundation rigidity**

In the absence of any foundation material it can be assumed that the structure has an absolutely rigid foundation. However this cannot be the case in reality since rigidity is a relative term. Therefore the analysis of the structural behaviour in the presence of foundation flexibility is desirable.
Assumptions

The degrees of freedom along the foundation and dam interface were removed when assuming a flexible foundation. The extent of the foundation extends to a depth of 1.5 times the height of the dam beneath the river bed level and lengths equal to the dam height in both the upstream and the downstream directions. The assumption for the extent of depth was made on the basis of usual practice (Matheu et al. 2004) for foundation consideration.

![Figure 47 Extent of the foundation for FEA](image)

Foundation material

According to the feasibility study report prepared for the Department of Electricity Development, GoN the foundation comprises of Gneiss rock. The physical properties were selected based on (Harvey B., Pezard and Petrov, 2005) which are listed below:

- Young’s modulus of elasticity = 56.2 MPa.
- Density = 2700 kg/m³
- Poisson’s ratio = 0.277

The values of the Young’s modulus and the Poisson’s ratio are both for static analysis.

- Normal Stresses

The results from the finite element analysis after introducing the foundation material is discussed in terms of normal stress at different points where the profile changes. The differences
in the values of normal stresses at different nodes on the dam profile are listed below (for the location of the nodes, refer Figure 48.)

The values of the normal stresses along the foundation nodes were compared for the two cases (Refer Table 18). The values of normal stresses at the base of the dam seem to be affected by the change in foundation property (Note points X and Y in Table 18).

Table 18 Normal stresses on kinks with and without foundation material

<table>
<thead>
<tr>
<th>Points</th>
<th>Node number</th>
<th>Normal stresses with rigid foundation (MPa)</th>
<th>Normal stresses with flexible foundation (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>2221</td>
<td>-0.0022</td>
<td>-0.0022</td>
</tr>
<tr>
<td>B</td>
<td>2231</td>
<td>-0.0027</td>
<td>-0.0027</td>
</tr>
<tr>
<td>C</td>
<td>24</td>
<td>-0.6756</td>
<td>-0.6682</td>
</tr>
<tr>
<td>D</td>
<td>71</td>
<td>-0.0036</td>
<td>-0.0036</td>
</tr>
<tr>
<td>E</td>
<td>118</td>
<td>-0.4888</td>
<td>-0.4703</td>
</tr>
<tr>
<td>F</td>
<td>4406</td>
<td>-0.2134</td>
<td>-0.2968</td>
</tr>
<tr>
<td>G</td>
<td>10096</td>
<td>-0.0041</td>
<td>-0.0041</td>
</tr>
<tr>
<td>H</td>
<td>86227</td>
<td>-2.1348</td>
<td>-3.5185</td>
</tr>
<tr>
<td>I</td>
<td>10046</td>
<td>-1.0604</td>
<td>-1.613</td>
</tr>
<tr>
<td>X</td>
<td>24367</td>
<td>0.3829</td>
<td>-4.9585</td>
</tr>
<tr>
<td>Y</td>
<td>24372</td>
<td>-0.3004</td>
<td>-2.91</td>
</tr>
</tbody>
</table>

However, the normal stresses of the nodes along the dam foundation interface do not differ much for the two cases except at the edges. The plot of the normal stresses of the nodes along the dam-foundation interface is shown in Figure 49 below:
Maximum Displacement

The maximum displacement in both the cases was at the crest of the dam at point B and the values are as follows:

- With rigid foundation: 0.001179 m
- With flexible foundation: 0.002413 m

Foundation material properties

To assess the effect of the change in property of the foundation material, a sensitivity analysis was carried out with changing values of the Modulus of elasticity and the change in the peak displacement and the peak normal stress were noted. A plot of the same is shown in below.
Conclusion

The toe and the heel of the dam are sensitive to the change in the properties of the foundation. But the effects do not seem significant enough as to alter the peak displacement of the dam by a significant figure. The primary matter of concern is the tensile stresses developing in the model but with the assumption of a flexible foundation, small compressive stresses are developed. Also, for Gneiss rock as a foundation material, the lowest value of modulus of elasticity reported is 40 MPa. This provides us with a foundation material which can be assumed as rigid for the analyses henceforth.

6.3.3 Linear Static Analysis: Load cases

For the static analysis of the model of the dam three different conditions were analysed. In the first case, the dam was analysed with a full reservoir case without the application of seismic load. In the second case the seismic load was applied alone without considering the weight of the dam and the effect of the hydrostatic pressure from the reservoir water. In the third condition all three forces were assumed to act on the structure. The results are discussed below.

6.4 Results and Discussion

The results show that the upstream face experiences the tensile stress which is maximum at the discontinuity (Node I, refer Figure 48). However when the model was analysed under the gravity and the hydrostatic pressure then the maximum tensile stress occurred at the base of the dam (Node X, refer Figure 48). The values of the peak normal tensile stresses in all three cases are illustrated in Table 19. The contour plots of the normal stress for all three cases are shown in Figure 52 (a), (b) and (c).
### Table 19 Peak Normal (SY) tensile stresses

<table>
<thead>
<tr>
<th>Load Case</th>
<th>Peak tensile stress (MPa)</th>
<th>Nodal location</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gravity + Hydrostatic (G+P)</td>
<td>0.2</td>
<td>X</td>
</tr>
<tr>
<td>Seismic (E)</td>
<td>5.2</td>
<td>I</td>
</tr>
<tr>
<td>Gravity + Hydrostatic + Seismic (G+P+E)</td>
<td>4.2</td>
<td>I</td>
</tr>
</tbody>
</table>

*For brevity, the forces are expressed as G (Gravity), P (Hydrostatic pressure) and E (Seismic).*

![Normal Stress contour plot for case G+P](image)

*a) Normal Stress contour plot for case G+P*
b) Normal Stress contour plot for case E

c) Normal Stress contour plot for case G+P+E

Figure 52 Normal stress contour plots for different load cases
The contour plot for the major principal stress is shown in Figure 53 for the third case (G+P+E). It can be noted that the stresses exceed the limiting tensile strength value of concrete (2.8 MPa). The nodes exceeding this value are marked and shown in Figure 54.

Figure 53 Major Principal Stress contour plot for static analysis (G+P+E)

Figure 54 Nodes exceeding the tensile strength value of concrete (G+P+E)
6.5 Conclusion

The sensitivity analyses indicated the optimum mesh density of the model was 759 elements whilst the foundation consisting of Gneiss rock could be assumed rigid for the analysis. A static analysis was carried out as the preliminary step which provided an idea about the behaviour of the structure when subjected to different combinations of load. The response of the structure was as expected with the tensile stresses developing on the upstream face and the compressive stresses dominating the downstream face. To obtain a more detailed investigation into the behaviour of the structure, a further analysis of comparatively more sophistication is necessary. Therefore the next step will be to conduct a linear dynamic analysis which is discussed in the following chapter.
Chapter 7

Dynamic Analysis of Arun III dam

7.1 Introduction

This chapter illustrates the dynamic analyses (response spectrum modal analysis, linear and nonlinear time history analyses) procedures carried out for the earthquake response computation of the tallest monolith section of the Arun III dam. The structural model including the added hydrodynamic mass of the water on the upstream face of the dam is considered. For the modal analysis the natural periods and mode shapes are determined and then used to compute the displacements and stresses induced by a set of design ground motions obtained from seismic hazard study for the concerned site. The seismic responses are computed and illustrated in contour plots and compared with the results from linear static analysis.

7.2 Response Spectrum Analysis

A response spectrum represents the response of single degree of freedom systems to a time history loading function. It is a plot of response (acceleration, velocity or displacement) versus frequency or the vibrational period. It is an analysis in which the results of a modal analysis are used with a known spectrum to calculate displacements and stresses in the model. It is used as an initial step in dynamic analysis mainly in place of time history analysis to determine the response of structures to random or time-dependent loading conditions such as earthquakes, wind loads, ocean wave loads, etc. (ANSYS Inc. 2010).

In general, due to their massive form the deformation behaviour of concrete gravity dams can be represented primarily by the first mode of vibration. However the contributions from the subsequent modes also come into consideration when dealing with excessively tall structures. Therefore, in this analysis, the effects of first ten modes of vibration are considered for evaluating the response of the dam section. The analyses were carried out with the finite element analysis software ANSYS 12.

7.2.1 Assumptions

For the response spectrum modal analysis a total of ten modes of vibration were considered in an attempt to provide the most accurate response with the most optimum expense of time. The stiffness characteristics of the material are as assumed in section 6.2.3. As the analyses in this stage are pertaining to dynamic state the appropriate dynamic material properties were adopted from those listed in section 6.2.3. The model was again a 2-dimensional plane strain model with 4 node quadratic element as in the static analysis.
7.2.2 Computation of earthquake response

Computation of earthquake response of the dam cross section involved the superposition of the static gravity and hydrodynamic loads together with the earthquake using the Response Spectrum Modal Analysis procedure. The response spectrum analysis consisted of the following:

- The natural periods and mode shapes along the upstream-downstream direction.
- The maximum deflections and stresses due to the excitation.

The earthquake response of the dam model was computed with the Response Spectrum Modal analysis procedure using the finite element analysis software ANSYS 12 including 10 modes of vibration. In ANSYS the response spectrum modal analysis is carried out using the Displacement – Frequency response spectrum. Displacement – Frequency response spectra of the selected representative ground motions were employed for the analysis as seismic input.

**Frequencies/natural periods and mode shapes**

The natural periods and mode shapes of the dam section were determined using the finite element software ANSYS. The results for the first ten natural periods of vibration along the transverse axis of the dam as obtained from the modal analysis results are summarized in Table 20. The respective mode shapes are shown in Figure 55 a) through j).

<table>
<thead>
<tr>
<th>Mode</th>
<th>Frequency (Hz)</th>
<th>Time period (s)</th>
<th>Modal Masses* (tons)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>8.6</td>
<td>0.1166</td>
<td>2945</td>
</tr>
<tr>
<td>2</td>
<td>18.5</td>
<td>0.0542</td>
<td>1512</td>
</tr>
<tr>
<td>3</td>
<td>20.9</td>
<td>0.0480</td>
<td>46</td>
</tr>
<tr>
<td>4</td>
<td>31.8</td>
<td>0.0314</td>
<td>394</td>
</tr>
<tr>
<td>5</td>
<td>40.8</td>
<td>0.0245</td>
<td>47</td>
</tr>
<tr>
<td>6</td>
<td>43.1</td>
<td>0.0232</td>
<td>190</td>
</tr>
<tr>
<td>7</td>
<td>47.6</td>
<td>0.0210</td>
<td>18</td>
</tr>
<tr>
<td>8</td>
<td>55.6</td>
<td>0.0180</td>
<td>81</td>
</tr>
<tr>
<td>9</td>
<td>56.7</td>
<td>0.0176</td>
<td>63</td>
</tr>
<tr>
<td>10</td>
<td>61.7</td>
<td>0.0162</td>
<td>58</td>
</tr>
<tr>
<td>Sum</td>
<td></td>
<td></td>
<td>5354</td>
</tr>
</tbody>
</table>

*The modal masses correspond to the X-direction

Total mass of the structure = 5973.4 Tons

The sum of the modal masses constitutes approximately 90% of the total mass of the structure and therefore a modal analysis with 10 modes sufficiently represents the behaviour of the structure under dynamic earthquake load.
Figure 55 Ten Mode Shapes for Arun III Dam

a) Mode No 1
b) Mode No 2
c) Mode No 3
d) Mode No 4
e) Mode No 5
f) Mode No 6
g) Mode No 7
h) Mode No 8
i) Mode No 9
j) Mode No 10
• **Structural response**

The responses of the structure analysed were the deformation (displacement) and the stresses induced in the structure. For all the five cases of seismic loading, the maximum displacement observed was at the crest of the dam on the upstream edge (node A). The maximum displacement response among the five seismic analyses results was from the Manjil earthquake with 15 mm of displacement. For a comparison, the design response spectrum was also used in the analysis. The displacement of the upstream edge for the different seismic input response spectra is listed in Table 21 along with the maximum normal and the maximum principal stresses. The location of the nodes is shown in the Figure 56. The maximum tensile stress values for both the cases were obtained in the lowermost discontinuity in the downstream face (node H).

<table>
<thead>
<tr>
<th>Response</th>
<th>Node</th>
<th>Uttarkashi Spectrum</th>
<th>Manjil Spectrum</th>
<th>ChiChi Spectrum</th>
<th>Landers Spectrum</th>
<th>Synthetic Spectrum</th>
<th>Target Spectrum</th>
<th>Static Analysis</th>
</tr>
</thead>
<tbody>
<tr>
<td>Displacement</td>
<td>(A)*</td>
<td>9</td>
<td>15</td>
<td>6</td>
<td>6</td>
<td>8</td>
<td>8</td>
<td>25</td>
</tr>
<tr>
<td>SY (MPa)</td>
<td>(H)*</td>
<td>4.4</td>
<td>7.0</td>
<td>2.7</td>
<td>2.9</td>
<td>4.0</td>
<td>4.0</td>
<td>7.7</td>
</tr>
<tr>
<td>S1 (MPa)</td>
<td>(H)*</td>
<td>5.0</td>
<td>8.0</td>
<td>3.0</td>
<td>3.3</td>
<td>4.6</td>
<td>4.5</td>
<td>8.6</td>
</tr>
</tbody>
</table>

* The location of the nodes is illustrated in the figure below.

**Figure 56 Location of critical nodes in the structure model**
For stress check purpose in seismic analysis of concrete gravity dams, it is customary to compare the tensile strength capacity of the concrete with the maximum tensile values of the major principal stress. From the observation, it is evident that for each case of seismic loading, the principal stress has a deciding value for the stress level. Therefore, for this analytical study, the first principal stress was taken into consideration to form a basis for conclusion. The contour plots of the major principal stress for the various cases of response spectrum loading were obtained from the finite element analysis software ANSYS 12. Also the regions with the stress values exceeding the tensile strength capacity of concrete were checked. The contour plots from the response spectrum modal analyses carried out for the response spectra for five different earthquake records and the design spectrum are presented in Figures 57 to 62 together with an illustration of the nodes in the model of the dam where the tensile stress values exceed the tensile strength capacity of the assumed grade of concrete. These plots for the major principal stress and the critical tensile stress region from the response spectrum analysis can be compared with the static analysis results previously plotted in Figure 53 and Figure 54.

It is notable that the five records selected as the design events differ from each other regarding their spectral responses (refer Figure 40). The records for the Landers and the ChiChi earthquakes have higher acceleration and displacement ordinates in the higher vibrational period ranges whereas, the records of the Manjil, the Uttarkashi and the Synthetic earthquakes have higher acceleration ordinates for the lower vibrational periods. The modelled dam has a fundamental period of vibration equal to 0.132 sec. The results obtained from the analysis are therefore pronounced for the Manjil, Uttarkashi and the Synthetic records compared to the ChiChi and the Landers records as expected.

On the other hand, the linear static method appears very conservative. The critical tensile region as depicted by the results from the static analysis is overly estimated and covers a vast region in the upstream face only. In contrast, the results from the dynamic response spectrum analyses have shown the tensile stress hot spots on both the upstream and the downstream sides. The dynamic analysis results are more realistic than the static analysis results since the cyclic reversed nature of the earthquake loading is better represented.
Figure 57 Response Spectrum Modal Analysis results: Uttarkashi

a) Major Principal Stress plot

b) Nodes exceeding tensile strength capacity
Figure 58 Response Spectrum Modal Analysis results: Manjil

a) Major Principal Stress plot: Manjil

b) Nodes exceeding tensile strength capacity
Figure 59 Response Spectrum Modal Analysis results: Chi-Chi

a) Major Principal Stress plot: Chi-Chi

b) Nodes exceeding tensile strength capacity
a) Major Principal Stress plot: Landers

b) Nodes exceeding tensile strength capacity

Figure 60 Response Spectrum Modal Analysis results: Landers
Figure 61 Response Spectrum Modal Analysis results: Synthetic

a) Major Principal Stress plot: Synthetic

b) Nodes exceeding tensile strength capacity
**Figure 62 Response Spectrum Modal Analysis results: Design Spectrum**

- **a)** Major Principal stress contours: Design Spectrum
- **b)** Nodes exceeding tensile strength capacity
7.3 Linear Time History Analysis

Time history analysis is carried out to calculate the states of stresses, deformations and section forces according to elastic or inelastic (as per linear or nonlinear analysis) stiffness characteristics of various components of the structure. The seismic input for such an analysis is ground motion time histories. From the analysis both the magnitudes and the time-varying characteristics of the seismic response are calculated which is followed by an appropriate interpretation and evaluation in terms of demand-capacity ratios, cumulative inelastic duration, extent of overstressed region and probable modes of failure are the basis for judgement on the estimation for probable level of damage.

7.3.1 Acceptance Criteria

It is customary to assess concrete hydraulic structures on the basis of stress checks when considering their seismic performance. A very commonly accepted criterion is the number of stress excursions (usually acceptable within 5 excursions). This methodology has been widely practiced around the world. However stress checks alone can not correctly evaluate the performance of dams against seismic loads as the extent of stress excursion and the duration of stress excursion are not addressed by simple stress checks. Ghanaat (2005) carried out extensive study in the seismic analysis of dams and established the following acceptance which is also recommended by USACE (2003). The acceptance criteria are based on the demand capacity stress ratio and the cumulative inelastic duration of the structure under the earthquake time history loading. It is proposed that the structure’s level of damage will be under acceptable limits of damage control if the performance curve falls below the limit as in Figure 63. If the curve falls outside the limit then nonlinear analysis will be required to assure that collapse will not occur.

Figure 63 Performance curve for Concrete Gravity Dams (USACE 2007)
The demand capacity ratio is defined as the ratio of computed tensile stress to tensile strength of the concrete. For gravity dams the DCR is computed using the principal stress demands and the capacity is computed using equation (34) and the maximum permitted DCR for dams is 2. In contrast, the cumulative inelastic duration is obtained by multiplying the number of stress point excursions beyond a certain level by the analysis time step (Ghanaat Y. 2004).

As per Ghanaat, for arch dams, the allowable cumulative duration is taken equal to the duration of five harmonic stress cycles having a magnitude twice the tensile strength and an oscillation period equal to 0.2 seconds, which makes up a cumulative duration of 0.4 seconds for a DCR of 1. The cumulative duration for a DCR of 2 is assumed zero. Arch dams resist loads by cantilever action along with arch action unlike gravity dams which rely solely on cantilever mechanism and therefore a lower cumulative duration of 0.3s is assumed for gravity dams (Ghanaat Y. 2004).

7.3.2 Assumptions

The stiffness characteristic of the material was as assumed in section 6.2.3. As the analyses in this stage are pertaining to dynamic state the appropriate dynamic material properties were adopted from those listed in section 6.2.3. The model was again a 2-dimensional plane strain model with 4 node quadratic element as in the static analysis.

7.3.3 Procedure

This section illustrates the transient analysis usually known as time history analysis procedure applied to the tallest section of the Arun III dam for the computation of earthquake response. The structure is assumed to be linear and the seismic load in the form of displacement time history is applied as the base excitation. The set of design ground motions were obtained from seismic hazard study. The seismic responses were computed and illustrated as displacement response time histories and stress response time histories for the critical regions in the cross section. The critical regions were identified on the basis of analysis results from response spectral modal analyses (RSMA). The stress and displacement hot-spots indicated by the contour plots from RSMA were adopted as the critical regions for the linear time history analysis.

Computation of earthquake response of the dam cross section using the Time history Dynamic Analysis procedure consists of the following steps:

a. Applying the static loads

b. Applying the transient load

The earthquake response of the dam was computed with the Time history Dynamic Analysis procedure using the finite element analysis software ANSYS 12 using two load-steps. In
ANSYS the full transient analysis is carried out as a combination of load-steps of desired duration. The first load step was the static load step involving the application of the static loads - gravity load, hydrostatic pressure and added hydrodynamic mass. The second load step was the dynamic load step involving the application of the displacement time history in the form of base excitation.

a) Application of static load
The static load acting on the dam section are the self weight of the dam body, the hydrostatic pressure from the reservoir water and the added hydrodynamic mass which represents the pounding effect of the water column adjacent to the dam body which accelerates with the dam as it moves during the earthquake. In the first load step of the analysis, these loads are applied in the similar manner as in static analysis. The boundary condition along the base of the dam body is that all degrees of freedom are fixed as in a static analysis. The first load step is recorded and then the second load step is prepared. Since the procedure selected is a full transient analysis, there is no necessity of considering master degrees of freedom or any mode shapes and natural frequencies.

b) Application of dynamic load
The dynamic load acting on the dam is the earthquake load which is transferred to the dam body through the nodes which connect it to the ground (i.e. the base of the dam). Therefore the dynamic load has to be applied at the base nodes. In ANSYS the base excitation can be applied to the structure in two ways:

1) As acceleration time history using the large mass method and
2) As displacement time history using it as a boundary condition at the base of the structure

The first method is complicated compared to the second one. A mass element has to be created and assigned a high mass value with certain assumptions before applying the acceleration time history. The second method is simpler and needs the displacement time history of the earthquake record to be used in the transient analysis of the structure. The displacement time history is applied as the boundary condition after removing the fixity of the nodes in the corresponding direction along which the earthquake ground motion is applied. Base excitations are therefore best applied in terms of displacement histories rather than as acceleration histories. Therefore the displacement time histories were used as the boundary condition representing the seismic action. The displacement time histories for the five earthquake records (refer Figure 39 for the acceleration time histories) are plotted using Seismosignal v3.3.0 (SeismoSoft Ltd 2010) are shown in Figure 64 for the information.
Figure 64 Displacement time histories of the design earthquake events (SeismoSoft Ltd 2010)
7.3.4 Results

The responses of the structure analysed were the deformation (displacement) and the stresses induced in the structure. As observed in the response spectrum analysis carried out earlier, the maximum deflection was expected at the crest of the dam on the upstream edge (node A). Therefore the response history of the crest for all five cases of transient loading was observed. The maximum displacement response among the five seismic analyses results was from the Manjil earthquake record with 14 mm of displacement which corresponds to a very low drift ratio of 1/5000. The displacement of the upstream edge for the different seismic input response spectra is listed in Table 22 along with the maximum normal and the maximum principal stresses for various points of significance in the cross section of the dam. The results between the time history analyses and the response spectrum analyses (refer Table 21) were quite consistent as expected. The location of the points/nodes is shown in Figure 65. The upstream side was in tension and the downstream side was in compression.

Table 22 Response maxima for different time history seismic inputs (−ve Stresses - compression)

<table>
<thead>
<tr>
<th>Response</th>
<th>Node</th>
<th>Uttarkashi</th>
<th>Manjil</th>
<th>ChiChi</th>
<th>Landers</th>
<th>Synthetic</th>
<th>Static</th>
<th>RSMA_{max}</th>
</tr>
</thead>
<tbody>
<tr>
<td>Displacement</td>
<td>(A)*</td>
<td>8</td>
<td>14</td>
<td>9</td>
<td>8</td>
<td>10</td>
<td>25.0</td>
<td>15</td>
</tr>
<tr>
<td>(\sigma_y) (MPa)</td>
<td>(I)*</td>
<td>1.03</td>
<td>1.19</td>
<td>0.7</td>
<td>0.7</td>
<td>0.84</td>
<td>7.7</td>
<td>8.2</td>
</tr>
<tr>
<td>(\sigma_1) (MPa)</td>
<td>(I)*</td>
<td>3.03</td>
<td>4.22</td>
<td>0.9</td>
<td>0.8</td>
<td>2.99</td>
<td>8.0</td>
<td>6.5</td>
</tr>
<tr>
<td>(\sigma_y) (MPa)</td>
<td>(X)*</td>
<td>0.21</td>
<td>0.34</td>
<td>0.6</td>
<td>0.7</td>
<td>0.21</td>
<td>-11.2</td>
<td>6.5</td>
</tr>
<tr>
<td>(\sigma_1) (MPa)</td>
<td>(X)*</td>
<td>0.23</td>
<td>0.44</td>
<td>0.8</td>
<td>0.9</td>
<td>0.34</td>
<td>-2.0</td>
<td>5.2</td>
</tr>
<tr>
<td>(\sigma_y) (MPa)</td>
<td>(H)*</td>
<td>-2.31</td>
<td>-2.15</td>
<td>-1.97</td>
<td>-1.97</td>
<td>-0.78</td>
<td>6.8</td>
<td>7</td>
</tr>
<tr>
<td>(\sigma_1) (MPa)</td>
<td>(H)*</td>
<td>-0.4</td>
<td>-0.37</td>
<td>-0.33</td>
<td>-0.34</td>
<td>-0.35</td>
<td>8.6</td>
<td>8</td>
</tr>
</tbody>
</table>

**The location of the nodes is illustrated in Figure 65 below.**
7.3.5 Comparison with acceptance criteria

The results from the linear transient analysis were compared with the acceptance criteria as discussed in 7.3.1. The graph developed from the results of the principal stress response at node I, for the three most severe cases, Manjil, Uttarkashi and Synthetic records are shown in Figure 66 through 70. For example, Figure 67 indicates that the greatest demand/capacity ratio was 1.5 with the ratio being exceeded for a total time of 0.12 seconds as can be observed from the principal stress time history of the node plotted in Figure 66 (i.e. 0.12 seconds reflects 12 excursions with a stress demand greater than the tensile capacity and each of duration 0.01 seconds). For the Chi Chi and Landers records, the stresses remained below the tensile strength capacity of the concrete. For the load case of the Manjil earthquake though the performance curve lies within the acceptable region, the number of excursions is greater than 5, and a nonlinear analysis has been undertaken in section 7.4 to further check the performance.

Figure 66 Major principal stress history for the node at the upstream discontinuity (Node I) for the Manjil earthquake

Figure 67 Performance under Manjil earthquake
Figure 68 Major Principal Stress history for the node at the upstream discontinuity (Node I) for the Uttarkashi earthquake

Figure 69 Performance under Uttarkashi earthquake

Figure 70 Major Principal Stress history for the node at the upstream discontinuity (Node I) for the Synthetic record
7.4 Nonlinear Time History Analysis

From the linear time history analysis of the dam model it was found that the structure will suffer cracks but collapse would not occur. For the confirmation of such behaviour of the structure, a nonlinear analysis was performed with some modification in the finite element model. The structure was remodelled introducing some spring elements across a section in the critical tensile region in the dam.

7.4.1 Assumptions

The stiffness characteristic of the material was as assumed in section 6.2.3 except for the spring elements used. The dynamic material properties were as previously adopted in the linear time history analysis. The model was again a 2-dimensional plane strain model with 4 node quadratic elements as in the previous analyses. However, to account for the nonlinearity in the material in a certain region of the structure, nonlinear elements were introduced which represent the potential crack interface. This interface was strategically located at the nodal points representing the critical tensile region as obtained from the linear time history analysis results. The location chosen was at the four nodes at the discontinuity (on the upstream side) starting from the upstream face of the dam towards downstream (Refer Figure 72). In nonlinear analysis the model has to be checked against MCE event. For this study as we adopted MCE to be equivalent to the MDE level event (Refer Section 4.5), the most severe case of earthquake loading among the five events was chosen, i.e. the Manjil earthquake.

7.4.2 Procedure

Four spring elements were introduced on the first four nodes across the discontinuity. The model was then subjected to the most severe of the loading cases, i.e. the Manjil earthquake record. A picture of the remodelled structure is shown in Figure 72.
COMBIN39 is a unidirectional spring elements are basically nonlinear elements which will function according to a user-defined load-deflection curve facilitated as real constant input in ANSYS (ANSYS Inc. 2010). The element has a longitudinal or torsional capability in 1, 2 or 3 dimensional applications. The longitudinal option (that has been utilised in this analysis) is a uniaxial tension-compression element with a maximum of 3 degrees of freedom (nodal x, y and z) at each node. Bending and axial loads are not accounted for in the element.

This analysis was a case involving the nonlinearity of the material. To simulate this in the model, a 10mm thick strip of the concrete across the discontinuity was assumed to be represented by springs. For computation of the load deflection curve, the concrete was assumed
to follow a bilinear stress-deformation relationship as proposed by Hilsdorf and Brameshuber (1991) which is as shown in Figure 74.

\[
\sigma(\omega) = \sigma_{tm}(1-0.85 \frac{W}{w_1}) \quad \text{for} \quad 0.15 \sigma_{tm} < \sigma < \sigma_{tm} \quad (37)
\]

\[
\sigma(\omega) = 0.15 \sigma_{tm} (w_c - w_1) \quad \text{for} \quad 0 < \sigma < 0.15 \sigma_{tm} \quad (38)
\]

Where, \( \sigma_{tm} \) is the mean tensile strength and \( E_c \) is the modulus of elasticity both in N/mm\(^2\) given by the following equations.

\[
\sigma_{tm} = 0.3 f_{ck}^{2/3}
\]

\[
E_c = 10^4 \sigma_{cm}^{1/3}
\]

Each spring will replace a strip of concrete of volume equivalent to 1.3m x 1m x 0.01m. The load deformation curves computed for the springs from the bilinear relationship are shown in Figure 76 (a) and (b). The figures show that for tensile load the deflection limit for elastic behaviour is 0.0007mm after which the material will start to fail. With a dynamic modulus of elasticity of 40,000 MPa, this amounts to a stress limit of 2.8MPa. Similarly for the compressive load, the deflection limit is 0.0087mm which implies to a stress limit of 35MPa.
113

This analysis is different from the linear time history analysis only in the respect that it includes nonlinear elements. The results obtained were analysed and the information is presented in the following section.

7.4.3 Results

After completion of the analysis first the histories of the principal stresses for the nodes across the section with spring elements was checked. Then, the relative displacements of the two nodes of each of the springs were computed and compared with the respective section (tensile or compressive) of the input curve. The stress histories of the nodes exhibited tensile stresses beyond the tensile strength limit thereby confirming the nonlinearity in the behaviour of the material (Refer Figure 77). Also to be noted is that the number of peaks in the plot is equal to the number of stress excursions as obtained from linear analysis earlier.
The relative displacements of the nodes of each of the springs were then computed and compared with the element real constant input curve defined for modelling. The comparison revealed that the first three springs exceeded the displacement limits of 0.7mm for the tensile curve which implies that the springs will break and therefore cracks will appear across the broken springs (Refer Table 23). Therefore the cracks will occur and propagate from the upstream face (Spring No 1) of the dam towards downstream with its tip occurring in between the Springs No. 2 and 3, amounting to a crack length of approximately 2.6 m.

### Table 23 Spring elements states after nonlinear analysis

<table>
<thead>
<tr>
<th>Spring No.</th>
<th>Max relative displacement (mm)</th>
<th>Concluding Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.5</td>
<td>&gt;0.7mm Spring breaks</td>
</tr>
<tr>
<td>2</td>
<td>1.03</td>
<td>&gt;0.7mm Spring breaks</td>
</tr>
<tr>
<td>3</td>
<td>0.84</td>
<td>&gt;0.7mm Spring breaks</td>
</tr>
<tr>
<td>4</td>
<td>0.01</td>
<td>&lt;0.7mm Spring intact</td>
</tr>
</tbody>
</table>

**Crack forms across the section from Spring No. 1 to Spring No. 3**

### 7.5 Discussion and Conclusion

The response spectrum analysis demonstrated that the response of the structure was first mode dominant. The response spectra of the ground motions (refer Figure 40 (a) and (b)) indicated high ordinates in the time period corresponding to the first mode for the Manjil, Uttarkashi and the Synthetic records which has been clearly reflected by the response of the structure as shown in Figures 66, 68 and 74 where the model experiences tensile stresses exceeding the tensile strength capacity of concrete. For the spectra of the Landers, Chi Chi and the target design event, the ordinates were lower and hence the response spectrum analyses results (refer Figures 70, 72 and 76) have reflected the behaviour accordingly.
The time history analysis, demonstrated that the stresses in the model exceeded the tensile strength capacity of concrete for three of the loading cases which are listed in the order of their severity:

a) Manjil earthquake  
b) Uttarkashi earthquake  
c) Synthetically generated ground motion

The remaining two cases did not demonstrate any signs of nonlinearity therefore an acceptance check was carried out for only the three cases mentioned above. The three cases were checked for the acceptance criteria as mentioned in section 7.3.1. From the comparison of results with the acceptance criteria it was found that the cumulative elastic duration for all the cases remained below 0.3 seconds and the demand capacity ratio was below 2.

Albeit the fact that the stresses exceed the tensile strength capacity of the concrete the comparison of the results from the linear time history analysis shows that the stress excursions are within the acceptable limits with minor cracking. However, for the Manjil earthquake loading case, the total number of stress excursions was greater that 5 and a nonlinear analysis of the structure was performed using spring elements with special nonlinear behaviour (load-deflection curve) modelled along the potential crack interface to further check the performance. The model was analysed with the Manjil earthquake record and as expected, the model suffered a crack of approximately 2.6m length along the interface with the springs indicating minor damage.

Therefore from the finite element analysis it can be concluded that the dam structure is prone to minor damage from localised cracking when subjected to the design level events.
Chapter 8

Conclusions and Recommendations

8.1 Conclusions

The conclusions drawn from this study are summarised below:

a. Arun III dam is the proposed dam of the Arun III HEP which is under construction. It is located in the eastern Nepal Himalayas which is in a seismically active region. With concern to the grave consequences following a potential earthquake, Arun III dam should be assessed relating to its performance against major earthquakes which are likely in the Himalayan seismic environment. For this purpose, a deep understanding of the characteristics of a concrete gravity dam including the study of performance assessment methods is necessary.

b. The seismic design and analysis of structures is carried out with consideration to three different levels of desired performance. a) Serviceability Performance, b) Damage Control performance and c) Collapse Prevention Performance. The related design criteria are i) Operational Basis Earthquake (450 years RP), ii) Maximum Design Earthquake (950 years RP) and iii) Maximum Credible Earthquake (deterministic event ) respectively.

c. For the analysis of the structure, information about the seismic hazard of the respective site is necessary. For regions lacking the information about seismic hazard, the estimation of hazard can be carried out by various methods available. For regions of high seismicity with known seismic sources the PSHA procedure is more suitable than CAM which was developed for lower seismicity regions (M < 7).

d. The representative design ground motions are selected from similar tectonic environment and within comparable range of magnitude and distance as defined by the seismic hazard assessment. The real records can then be scaled satisfying all necessary criteria for scaling.

e. The seismic analysis of structures is carried out in a step by step manner starting from the simplest i.e. the static approach and slowly incorporating complexity. From the results of the linear static analysis, the upstream side of the dam model was observed to have tensile stresses higher than the tensile strength capacity of concrete.

f. To investigate further about the structural behaviour, dynamic analysis is necessary. According to the widely followed convention in linear dynamic analysis the dam models are checked against the acceptance criteria as outlined in EM 1110-2-6051 (USACE 2003). From the results of the dynamic analysis it was confirmed that the structure undergoes minor cracking across parts of the section but this does not result in collapse of the structure.
g. A further confirmation about the damage level was achieved by carrying out a nonlinear time history analysis which showed that the cracks will not propagate beyond the length of 2.6m from the upstream face of the dam. Arun III dam is capable of resisting the maximum design level earthquake event with only minor cracking.

### 8.2 Recommendations

This case study was aimed at establishing the seismic hazard level for Eastern Nepal and investigating the seismic performance of the proposed Arun III dam. With a limited scope, this study was not able to incorporate the following aspects which are recommended for future investigation:

a. Due to the lack of attenuation models from the Himalayan region, the Atkinson and Boore model was used which was developed for subduction zones around the world. The Himalayan region is however different from the regular subduction zones where a continental crust collides with a oceanic crust. Therefore seismic activity will actually be different from that in the subduction zones. Therefore for a better assessment of the Himalayan seismic hazard native attenuation models are required which ultimately means a strong motion instrumentation array for the region is necessary. This would enable a better estimate of all the levels of probability (OBE, MDE, MCE).

b. The Arun III dam is designed to be a massive structure with a huge volume of reservoir. Also there will be discontinuities, chambers and drainage holes inside the dam which will be the weak points in the case of a seismic event. This study did not incorporate the presence of these in the model which can be the subject of a further investigation.

c. The sharply shaped edges and abruptly varying discontinuities in the design document may be reconsidered for a smooth edged structure with gradually varying discontinuities which is a recommendation for the design aspect of the structure. In addition, a further study could be undertaken to better optimise the cross section shape of the dam.

d. In the case of nonlinearity in the dam, the behaviour of the dam along the third dimension may be of interest which has not been a part of this study. A further investigation on this aspect may be of interest.

e. The action of upthrust and silt pressure was not analysed in this study which might be of concern for further study.

f. In this study, only ground shaking effects have been investigated. The direct effects of the fault rupture on the concrete gravity dam can be the subject of further research.
References


Google Earth image Viewed on 10/10/2008.


Lam et al. (2000). GENQKE. Melbourne, VIC, Australia, University of Melbourne.


Teimuraz Matcharashvili, Tamaz Chelidze and Joachim Peinke (2007). "Increase of order in seismic process around large reservoir by water level periodic variation." Nonlinear Dynamics 51(399-407).


USACE (1999). Response Spectra and Seismic Analysis for Concrete Hydraulic Structures, Department of the Army USACE, Washington, DC.


USACE (2003). EM 1110-2-6051 Time History Dynamic Analysis of Concrete Hydraulic Structures, Department of the Army USACE, Washington, DC, HQUSACE.


Westergaard (1933). "Westergaard's Added Hydrodynamic mass model."
