Development of Response Spectrum for Indian context and its correlation with Design Spectrum

BY

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Major Project

Submitted in partial fulfillment of the requirements

For the degree of

Master of Technology in Civil Engineering

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May 2013

Declaration

This is to certify that

a. The thesis comprises my original work towards the Degree of Master of Technology in Civil Engineering (Computer Aided Structural Analysis and Design) at Nirma University and has not been submitted elsewhere for a degree.

b. Due acknowledgement has been made in the text to all other material used.

Tejan N. Patel

Certificate

This is to certify that the Major Project entitled **"Development of Response Spectrum for Indian context and its correlation with Design Spectrum"** submitted by **Mr. Tejan N. Patel (11MCLC20)**, towards the partial fulfillment of the requirements for the degree of Master of Technology in Civil Engineering (Computer Aided Structural Analysis and Design) of Nirma University, Ahmedabad, is the record of work carried out by him under my supervision and guidance. In my opinion, the submitted work has reached a level required for being accepted for examination. The results embodied in this major project, to the best of my knowledge, haven't been submitted to any other university or institution for award of any degree or diploma.

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Abstract

Earthquake is a natural hazard that cause damage or sometimes complete collapse of man-made structures. Every earthquake event does not have potential to cause damage to the structure only strong ground motion does it. Therefore, it is important to understand the response of structures subjected to such strong ground motions, i.e. seismic excitations. Maximum response shown by structure to strong ground motion is an important design input for earthquake resistant design.

Present study focuses on generating Response Spectrum for Indian subcontinent. It also aims toward developing Design Spectrum from response spectrum and compare them with the design spectrum of IS: 1893 - 2002 (Part-1). Response Spectrum is developed through solving equation of motion for Single Degree of Freedom System (SDOF) using numerical algorithm Newmark-Beta method. About 184 earthquake ground motions recorded at 23 recording stations in India are considered. Various Strong Ground Motion parameters like Peak Ground Acceleration (PGA), Root Mean Square (RMS) acceleration and Duration are considered to qualify earthquake ground motion to strong ground motion. Strong Ground Motion (67) quantified out of 184 earthquake excitation records are divided into four regions like North, South, East and West. Family of Displacement, Pseudo-velocity and Pseudo-acceleration response spectrum are generated. Statistical approach is employed to derive single representative Response Spectrum for each response quantity for each region.

In order to understand conservativeness or deficiency of such response spectrum as compared to code based design spectrum, four storey R.C.C. building is considered. Peak acceleration and Peak Base Shear is calculated from response spectrum as well as for code based design spectrum. An attempt is made to generate response spectrum for Multi Degree of Freedom (MDOF) system subjected to earthquake ground excitation. It is found that, single response spectrum is enough to estimate lateral force on the building system since response spectrum generated for each mass level of MDOF system is related to their dynamic mode shapes.

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Abbreviation Notation and Nomenclature

SDOF .... Single Degree of Freedom

MDOF .... Multi Degree of Freedom

EQ .... Earthquake

PGA .... Peak Ground Acceleration

PGV .... Peak Ground Velocity

PGD .... Peak Ground Displacement

RMS .... Root Mean Square

m .... Mass of Building

k .... Stiffness of Building

c .... Damping of Building

σ .... Standard deviation

ζ .... Damping Ratio

ω_n .... Natural Frequency (Hz)

u_g .... Ground Displacements

D .... Relative displacement

V .... Pseudo-velocity

A .... Pseudo-acceleration

Sa .... Spectral acceleration

Tn .... Natural time period

SRSS .... Square root of sum of square

CQC .... Complete quadratic combination

f_{ck} .... Characteristic Strength of Concrete

f_y .... Characteristic Strength of Steel

Contents

List of Tables

List of Figures

Chapter 1

Introduction

1.1 General

Earthquake is a disastrous natural force that cause damages to almost all manmade structures. Hundreds of small earthquakes occurs around the world regularly. It is important that earthquakes should be understood fundamentally so as structures can be protected. Many earthquakes that occurs are so weak that they can be detected by measuring instruments only and never be felt. Therefore it is important to know the characteristics of an earthquake. Strong Ground Motion i.e. ground motion which has sufficient strength to affect people and damage structures are of prime importance. Strong ground motions produced by earthquakes are random in nature and contain energy of different magnitudes. Strong ground motions are prescribed by parameters like Peak Ground Acceleration, Root Mean Square acceleration, Duration etc.

Structures when subjected to strong ground motion i.e. seismic excitation respond differently depending upon its in-built dynamic properties. Structure modelled as Single Degree of Freedom (SDOF) system when subjected to seismic excitation, equation of motion is described as

$$m\ddot{u}(t) + c\dot{u}(t) + ku(t) = -m\ddot{u}_g(t) \tag{1.1}$$

where m, c and k are mass, damping and stiffness of the system respectively.
u(t), $\dot{u}(t)$ and $\ddot{u}(t)$ are relative displacement, relative velocity and relative acceleration of the mass w.r.t ground.
$\ddot{u}_g(t)$ is ground acceleration.
Closed form solution is not possible for the Equation 1.1 because of right hand side term $\ddot{u}_g$ is not a continuous time function but is a discrete time function. Numerical algorithm is required to solve Equation 1.1 and thus solution of Equation 1.1 is time intensive. In order to save computational effort to estimate seismic force, concept of Response spectrum was developed. It is now widely accepted among researchers and is a central part of design code of various countries. Response spectrum provides maximum response of SDOF system under seismic excitation.

1.2 History of Response Spectrum Method [22]

The mathematical formulation of the Response Spectrum Method (RSM) first appeared in the doctoral dissertation of M.A. Biot in 1932. The Response Spectrum Method remained in the academic sphere of research for about 40 years and did not gain wide engineering acceptance until the early 1970s mainly because of two reasons, firstly, difficulty in computing the response of structures to earthquake ground motion and, second, there were only a few well-recorded accelerograms that could be used for that purpose. This started to change in 1960s with the arrival of digital computers and the commercial availability of strong-motion accelerographs. Before the digital computer age, the computation of structural response was time consuming, and the results were unreliable. By the late 1960s and early 1970s, the digitization of analog accelerograph records and the digital computation of ground motion and of the response spectra were developed completely and tested for accuracy. Then, in 1971, with the occurrence of the San Fernando, California, earthquake, the modern era of RSM was launched.

1.3 Earthquake Excitation [1]

The most useful way of defining the ground shaking during an earthquake is the time variation of ground acceleration. Thus, for given ground acceleration the solution of the problem is defined completely for a SDOF system with known mass, stiffness, and damping properties. Ground motion during an earthquake is measured by strong motion instruments which record the acceleration of the ground. Three orthogonal components of ground acceleration, two in the horizontal direction and one in the vertical, are recorded by strong-motion accelerograph. It does not record continuously but it is triggered into motion by the first wave of the earthquake to arrive. After triggering, the recording continues for some minutes or until the ground shaking falls again to imperceptible levels. The instruments must be regularly maintained and serviced so that they produce a record when shaking occurs.

The basic element of an accelerograph is a transducer element which is characterized by its natural frequency fn and viscous damping ratio ζ Instrumental records of strong ground shaking were scarce for many years and very few records were obtained from a destructive earthquake in some parts of the world. For example, no strong-motion records were obtained from two earthquakes during 1993 that caused much destruction: Killari, Maharashtra, India, September 30, 1993; and Guam, a U.S. territory, August 8, 1993. Now when a strong earthquake occurs it is desirable to have many stations instrumented to record the ground motions. But as it is difficult to predict future earthquakes and having limited budgets for installation and maintenance of instruments, recordings of ground motion in the region of strongest shaking is only occasionally possible. Many records have been obtained in regions where moderate ground shaking has occurred. The first strong-motion accelerogram was recorded during the Long Beach earth-quake of 1933, and since that time several hundred records have been obtained.

1.4 Objective of Study

The main objective of the study is to develop response spectrum for Indian subcontinent based on recorded earthquake excitation. Focus of the study is to understand response spectrum developed for the country are comparable with the design spectrum given in code. Study also aimed at deriving design spectrum from generated response spectrum for various regions of the country and compare them with the design spectrum given in IS:1893-2002 (Part-1) code. Overall present study is an attempt to look at conservativeness or deficiency of the design spectrum based estimation of lateral force on the structure.

1.5 Scope of Work

To achieve above mentioned objectives, following scope of work is proposed.

- Understand Response Spectrum concept for elastic systems.
- Collect Earthquake Ground Motion records from authentic sources for Indian subcontinent.
- Define parameters for strong ground motion.
- Develop and validate Displacement, Velocity and Acceleration Response Spectrum for El Centro ground excitation using Newmark Beta numerical algorithm through MATLAB.
- Develop Displacement, Velocity and Acceleration- Response Spectrum for various regions of Indian subcontinent for available earthquake records.
- Develop smooth design spectrum based on statistical analysis from developed Response Spectrum.
- Compare code based design spectrum with representative response spectrum for various parts of Indian subcontinent.

- Consider G+3 storey RCC building and estimate lateral force using developed response spectrum and code based design spectrum.
- Summary and conclusion of the study.

1.6 Organization of Report

The Major Project is divided into eight chapters. They are as follows:

Chapter 2 comprises of literature review covering various research papers, report etc. It focuses on various studies carried out to define strong ground motion parameters and their characteristics. It also includes papers discussing concept of response spectrum, its development and usefulness in Earthquake engineering. Chapter also covers literature related to generation of design spectrum from the derived response spectrum.

Chapter 3 gives concepts of response spectrum. It includes equation of motion of Single Degree of Freedom (SDOF) system and its solution subjected to earthquake excitations. It deals with the generation of response spectrum for SDOF system under El Centro (1940) earthquake excitation and validation of the same with reported result. Lastly, chapter covers important characteristics of Response Spectrum.

Chapter 4 covers study of various strong ground motion parameters and their characteristics. It includes compilation of ground motions for Indian subcontinent obtained from various sources. It also covers qualification of recorded earthquake excitation to Strong Ground Motion.

Chapter 5 includes generation of response spectrum for various regions of India using Newmark-Beta method through MATLAB. Statistical analysis of Response Spectrum is carried out to develop Design Spectrum for various regions of India. It also includes comparison among developed response spectrum, design spectrum and code based design spectrum.

Chapter 6 includes the estimation of lateral load on four storey RCC building using developed Response Spectrum for various regions of India. Comparison of lateral load and acceleration is carried out for RCC building from various developed generated response spectrum, design spectrum and code based design spectrum.

Chapter 7 includes equation of motion and its solution for MDOF system subjected to El Centro earthquake excitations. It also deals with the generation of Response Spectrum for MDOF system.

Chapter 8 includes the summary of the study, conclusions and future scope of work.

Chapter 2

Literature Survey

2.1 Introduction

The behaviour of building is of fundamental importance in modern structural design in order to ensure safety and serviceability of the structure. Thus, it is important to measure the response of the structure subjected to ground motion. For the said purpose and the objectives of major project enlist in Section 1.4 of chapter 1, an extensive Literature review related to strong ground motion parameters, strong ground motion characteristics and response spectrum development is carried out. Various research papers and technical reports have been referred to understand the basic concept of response spectrum, it's use to reveal significant characteristics of ground motions and development of smoothed design spectrum.

2.2 Literature Review

2.2.1 Characteristics of Ground motion

Various papers have been referred for basic understanding of ground motion parameters and their characteristics. Some of the important and relevant literatures are summarized below.

Vanmarcke and Lai [**6**] proposed a simple procedure for estimating the strong motion duration and the RMS strong-motion acceleration of earthquake ground motion records. Two simple measures of duration had been mentioned. The first defines duration as the time interval between the first and last peaks equal to or greater than a given level. The second definition was based on the concept of cumulative energy obtained by integrating squared accelerations. The first proposed procedure was used to obtain durations for time histories of ground acceleration where RMS acceleration was used as the basis. The definitions of duration was explained by considering S.ROCCO Friuli Earthquake excitation as shown in Figure 2.1.

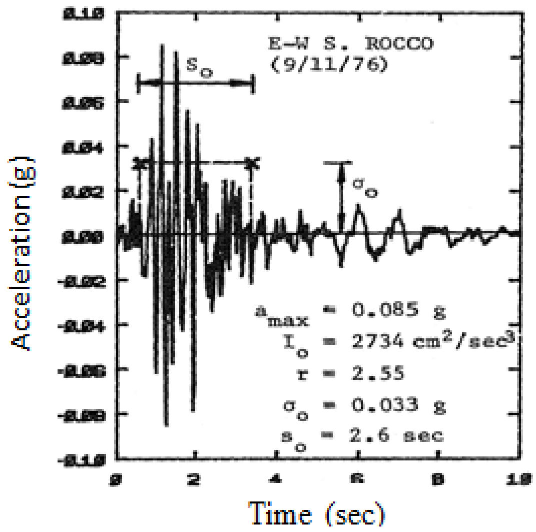


Figure 2.1: Time history of S. Rocco Friuli earthquake

McCann [7] looked into the use of the RMS acceleration (RMSa) and duration as a means of characterizing strong ground motion. A method was developed for identifying the strong motion part of the ground shaking based on the rate of change of the RMS acceleration. For a discrete accelerogram containing m points, the RMS was determined for each point n, where n varies from 0 to m.

RMS acceleration was expressed as:

$$rms_n = \sqrt{\frac{1}{n}\int_0^n [a(t)]^2 dt} \tag{2.1}$$

where rms = root mean square acceleration

a(t) = acceleration time history

The strong motion duration for a ground excitation was calculated using RMS acceleration as a threshold level.

Bolt [**8**] defined the "bracketed duration" of a record, as the time elapsed between the first and last acceleration excursions greater than a given level. This definition required that the absolute values of the acceleration of a record exceed some level. The paper dealt with the use of 0.05 or 0. 1g acceleration value as a threshold level. Therefore, records having a peak ground acceleration smaller than 0.05g had zero duration.

Bommer and Pereira [**9**] stated that the destructive capacity of ground motion increases with its duration, though the duration gives no indication at all of the damage potential. For two accelerograms with similar values of peak ground acceleration (PGA), the one with longer duration was likely to be more destructive, if the frequency contents were similar. For the motion to be damaging to engineered structures, the value of PGA, peak ground velocity (PGV) and Arias intensity was decided to be atleast 0.2g, 20cm/s and 0.8m/s respectively. Many accelerograms with PGA greater than 0.2g were not damaging, because intensity of shaking was as low as V or VI on MMI scale. So it was not possible to measure damage potential of ground motion by any single parameter. It would depend on all three ground motion parameters duration, frequency content and amplitude.

Maniyar and Khare [**10**] selected 20 ground motion time histories from all available recorded Indian earthquake events. Time histories selected were based on a detailed statistical study performed on various ground motion parameters like peak ground acceleration, peak ground velocity, peak ground displacement, RMS acceleration, RMs velocity, RMS displacement, Arias intensity, characteristic intensity, spectral acceleration, acceleration spectrum intensity and significant duration. Statistical analysis was performed by scaling the time histories to uniform values of various parameters considered. Based on the work, it was concluded that:

- Due to random nature of earthquake, it required more than one ground motion record to show variation in response.
- PGA parameter alone does not gave true information about damage potential of an earthquake. Selected parameters must be capable of capturing all information of ground motion parameters (amplitude, frequency content and duration) that affects response of structures.
- Spectral accelerations (Sa) varies for different fundamental periods for a given time history. At the same time, Sa also varies for different time histories at a given fundamental period.

Shoji et al. [11] dealt with the duration, RMS amplitude and PGA. It was observed that these parameters were affected by the hypocentral distance and local site conditions. The correlation between the duration, PGA and the range of shear-wave velocity in the upper layer were conducted. In addition, the duration and maximum amplitude were examined with emphasis on site amplification due to the local site conditions. The concluding remarks were summarized as follows:

- The duration had a general trend to become larger as near surface layers (upper 30-m) get softer, also PGA had the same tendency.
- With increasing hypocentral distance, the scatter of data for the duration would be larger and larger. The variation of duration at 'soft' sites was larger than those at hard sites.
- PGA and the duration had reciprocal tendency against hypocentral distance, namely PGA inversely proportional to the duration.

- The duration was not less important than the maximum amplitude and frequency content in earthquake engineering.

2.2.2 Response Spectrum

Various research papers old to recent suggests use of response spectrum, its characteristics and generation of design spectrum from them. Few papers summarized have with are classic work carried out by Newmark, Housner and Hudson.

Hudson [12] discussed several types of response spectrum of use in engineering seismology and the relationships between these spectra and other basic quantities such as energy inputs and seismic coefficients were given. The use of the response spectrum to reveal significant characteristics of ground motion was discussed, and the role of the response spectrum in establishing seismic coefficients for structural behaviour was illustrated by experimental data. Various methods for determining response spectra were compared and an electric spectrum analyzer was briefly described.

From the study of the response spectrum, general conclusions concerning the relative importance of various factors in the earthquake problem were deduced as follows:

- A typical response spectrum for ζ = 0, 10, 20 % was examined as shown in Figure 2.2. It was noticed that there were many irregular peaks and an amount of damping effectively removes most of the peaks.
- An evaluation of seismic coefficients or lateral force co-efficients could not be obtained without the use of the response spectrum. The maximum accelerations expected in a structure were not those which were recorded by the ground motion accelerometer, since dynamic amplification effects occur which made the structural accelerations considerably larger than the ground accelerations. From the response spectrum, the maximum value of the total shear force was directly obtained.

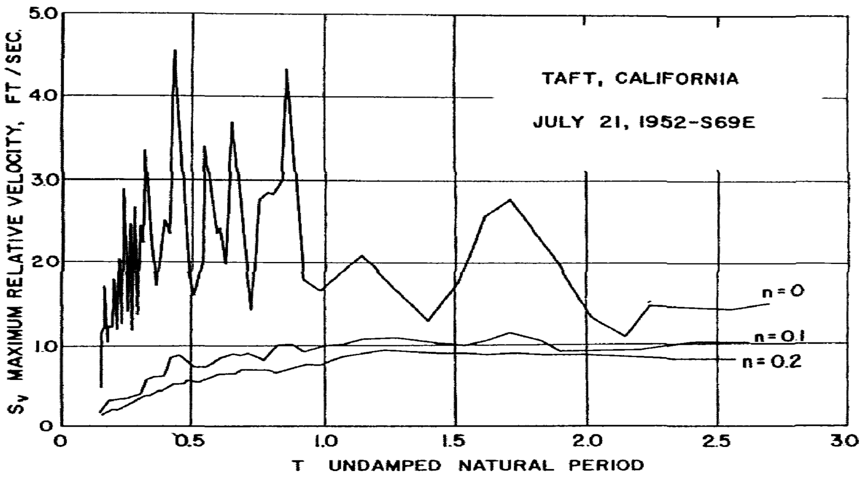


Figure 2.2: Typical earthquake response spectrum

Alford and Housner [**13**] inspected several strong motion earthquake records. They showed that acceleration for various earthquake ground motion are extremely irregular as shown in Figure 2.3 and Figure 2.4. So it was suggested that earthquakes must be analyzed as completely random phenomena. It was also found that damping is a very important parameter as relatively small amount of damping reduces structural response sharply.

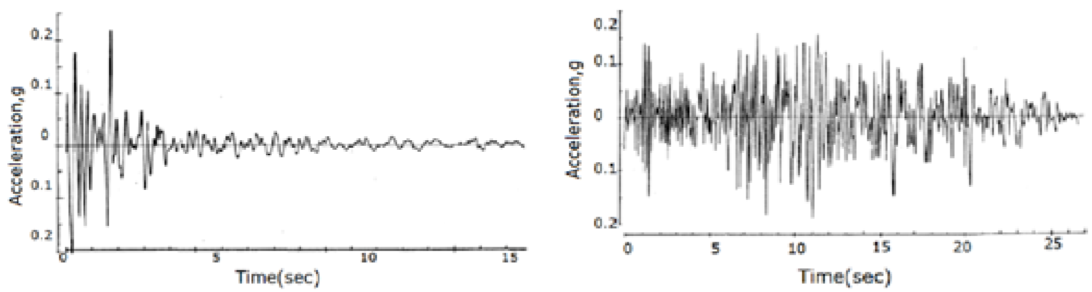


Figure 2.3: Accelerogram for Santa Barbara, California and Olympia, Washington, earthquake

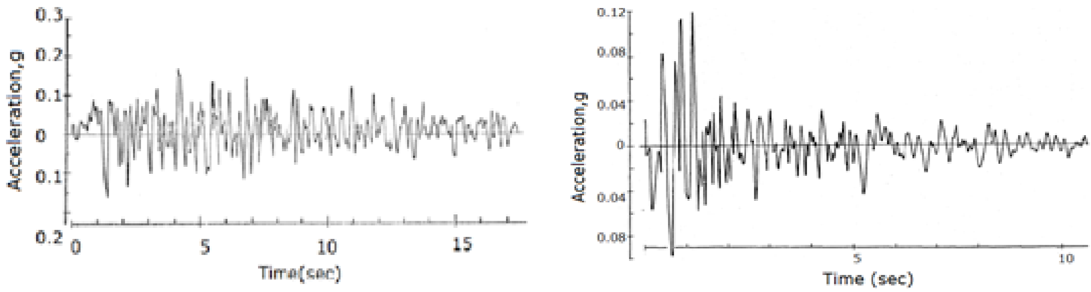


Figure 2.4: Accelerogram for Taft and Vernon, California, earthquake

Jenschke et al.[**14**] used three methods namely Probabilistic, Fourier Spectra and Response Spectra to investigate characteristics of strong ground motions. A description was given of the results obtained with response spectrum method. The relations and properties of five different response spectrum, absolute acceleration (AA), pseudo-absolute acceleration (PSAA), relative velocity, pseudo-relative velocity, relative displacement response spectrum were studied. The conclusions drawn from the investigations were as follows:

- Using frequency as abscissa avoids the accumulation of oscillations near the origin that occur when spectra was plotted versus period and thus usually gives smoother curves.

- AA and PSAA spectra were identical for zero damping and differ in a small amount for rest of damping curves as shown in Figure 2.5.

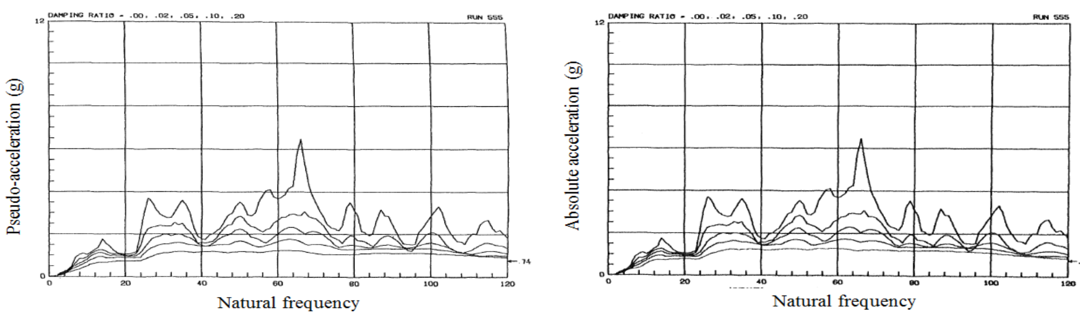


Figure 2.5: Pseudo Acceleration and Absolute Acceleration Vs Natural frequency plot for Transverse component of Earthquake

- The range of variability of acceleration was greatest, so it was selected for classifying ground motion events.

Ahmadizadeh et al.[14] studied the effect of ground motion parameters on pseudo-acceleration response spectrum. A total of 620 Iran's earthquake time histories were selected for the study. The average response spectrum from Iranian earthquakes showed high spectral accelerations for short period structures (about 0.2 to 0.4 sec) as shown in Figure 2.6. It resulted in severe damage to low rise structures. It was also observed that longer strong motion duration resulted in larger spectral values for periods longer than about 0.2 sec. For shorter strong motion duration, it showed larger spectral values in short periods.

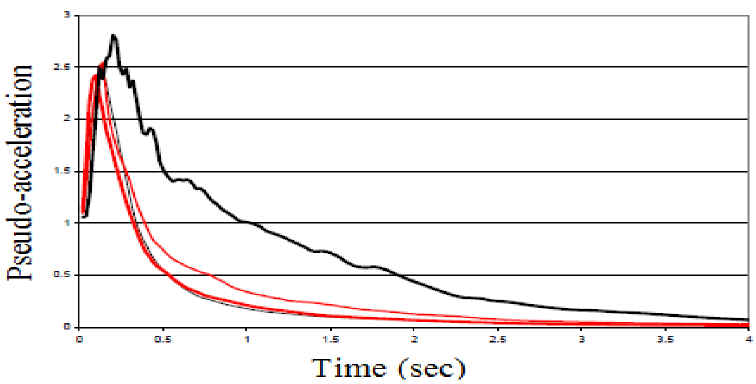


Figure 2.6: Average Pseudo-Acceleration Response Spectrum (Damping 5%)

Freeman [15] reviewed the concept of response spectrum by representing the ground motion record of Northridge earthquake in California. It was observed that response spectrum showed jaggedness with sharp peaks as shown in Figure 2.7. The peaks and valleys showed the sensitivity to response of structures to a slight variation in the natural period of vibration. So, a method of constructing smooth response spectrum was developed to overcome peaks and valleys from actual response spectrum curve.

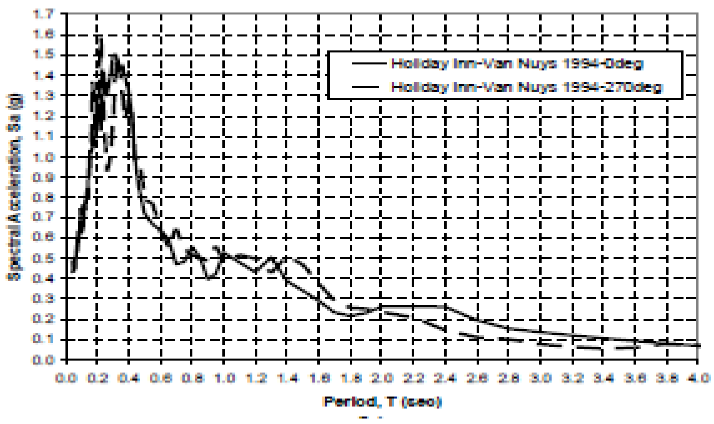


Figure 2.7: Northridge response spectrum

2.2.3 Design Spectrum

Good amount of literature is available about generating design spectrum from response spectrum. Few important work related to design spectrum development is given here with.

Newmark [16] developed vertical and horizontal (two components) response spectra for a series of 14 strong motion earthquake records for 0.5, 2, 5 and 10 % of critical damping. It was decided that the ground motion data were generally valid only in the frequency range of 0.05Hz to 30Hz and accordingly the response spectra were plotted only for this range. The mean and mean plus one standard deviation response spectrum for both horizontal and vertical components were computed.

Response Spectra:

The response computations was carried out for 38 frequencies and the shape of the spectra was influenced by the interval in the frequency range as shown in Figure 2.8. It was found that influence had not been large if small intervals of frequency was used.

Response Amplifications:

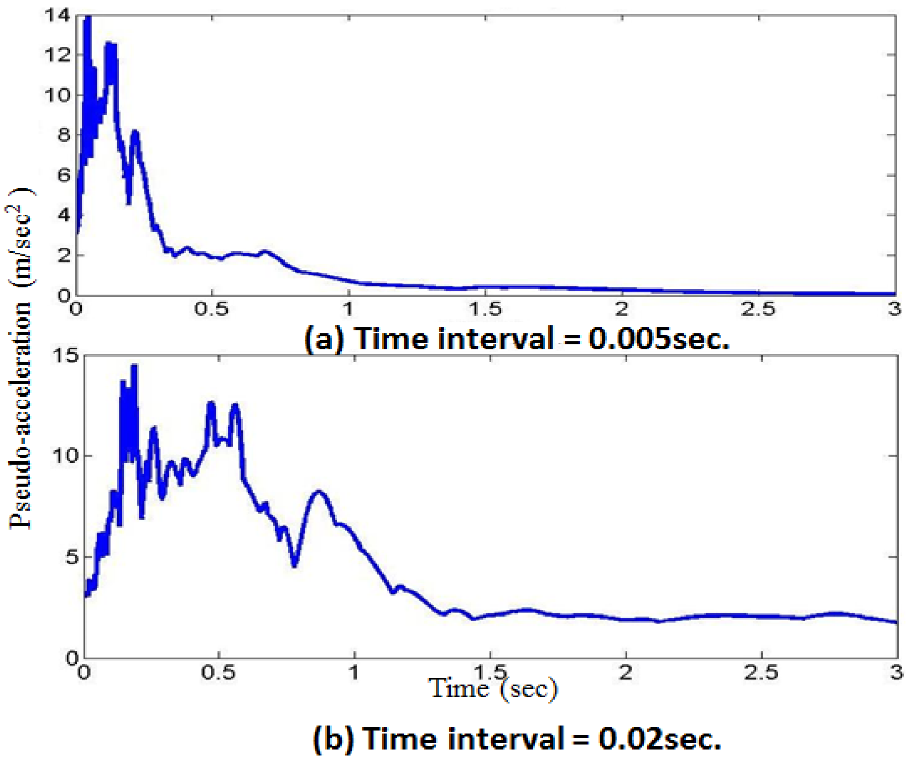


Figure 2.8: Response spectra with different interval in frequency range

It was found necessary to carry out the statistical computation at each frequency in order to account accurately for variation of amplification factors. Studies of the response amplification in the various ranges of frequencies was made by studying plots on the four-way logarithmic plot by normalizing the ground motion to PGD, PGV, PGA.

Amplification Factors:

The amplification factors was used to develop design response spectra. To compute the amplification factor, the ratio of the computed response to the maximum ground motion was carried out for displacement, velocity and acceleration at each frequency for a particular range of interest. From the study of plotted response spectra, amplification factors were presented over a frequency range. The displacement, velocity and acceleration amplifications for a number of frequencies were averaged and results were presented as shown in Table 2.1.

A value of 75 percentile means that 75 percent of the values fall at or below that

Table 2.1: Amplification Factors

Damping ζ %	**50 percentile**			**75 percentile**			**90 percentile**		
	α_A	α_V	α_D	α_A	α_V	α_D	α_A	α_V	α_D
0.5	4.00	2.86	1.98	5.02	3.81	2.66	5.95	4.67	3.27
2	2.91	2.23	1.69	3.52	2.89	2.23	4.06	3.48	2.72
5	2.20	1.74	1.39	2.59	2.19	1.80	2.93	2.60	2.17
10	1.72	1.38	1.13	1.97	1.69	1.43	2.20	1.98	1.71

particular amplification value.

The frequency range used for averaging the amplifications were as follows :

Horizontal displacement .... 0.2 to 0.4 Hz

Horizontal velocity .... 0.4 to 2.0 Hz

Horizontal acceleration .... 2.0 to 6.0 Hz

Mohraz [17] presented a study of 54 earthquake records (three components from each record) from 46 stations in 16 seismic events. Response spectrum was generated for each of this records. Three regions of amplifications were determined in a typical response spectrum; the low-frequency or displacement region, the intermediate-frequency or velocity region, and the high-frequency or acceleration region.

Procedure for constructing a design response spectrum was discussed. The three amplifications (displacement, velocity, and acceleration) and the three ground-motion parameters at the site were estimated, and then spectrum ordinates in each region were obtained from the product of the ground motion and its amplification in that region. Since the peak ground motions, Acceleration (A), Velocity (V), Displacement (D), for various earthquake records differ, response spectrum was normalized to determine design spectrum. At each frequency, the ratio of the computed response to the maximum ground motion for acceleration velocity, and displacement was obtained. These ratios were called as the amplification factors (acceleration amplification, velocity amplification, etc.). It was found that corresponding amplifications were nearly constant in each region so they were averaged within the region to obtain design amplifications. A design spectrum was constructed by computing the spectral bounds in each frequency region as product of the ground motion and response amplification.

Ghasemi et al.[18] proposed a practical procedure for constructing smooth response spectrum from the peak values of ground motion. The dynamic amplification factors were calculated for horizontal and vertical components by considering a selection of Iran's strong-motion records. They were compared with amplification factors reported by Mohraz [17], Newmark and Hall [16]. Different response spectrum were developed from different strong motion records and were compared by normalizing to the same scale. A method to construct tripartite graph (all spectral quantities like displacement, velocity and acceleration displayed in a single graph) was shown. Procedure for the construction of smooth design spectrum using peak ground motion parameters was presented by using an example.

Seed et al.[19] presented the results of a statistical analysis of the spectral shapes of 104 ground motion records obtained from 23 earthquakes for four different site conditions: rock, stiff soils less than about 150 ft deep, deep cohesionless soils with depths greater than about 250 ft, soil deposits consisting of soft to medium stiff clays with associated strata of sands or gravels. Normalized acceleration response spectra was first determined and analyzed statistically to obtain the mean spectrum shape and the mean-plus-one standard deviation spectrum shape (84 percentile approximately). It was found that there was wide difference in spectral shapes depending on the site conditions.

2.3 Summary

In this chapter, review of relevant literature is carried out. The review of literature includes study of strong ground motion parameters, concept of response spectrum along with its characteristics and method to develop design spectrum.

Chapter 3

Response Spectrum Concept

3.1 Introduction

The most important applications of the theory of structural dynamics is in analyzing the response of structures to ground shaking caused by an earthquake. As a structural engineer, mostly we are concerned with the deformation of the structural system or displacement u(t) of the mass relative to the moving ground. Once the deformation response history u(t) is evaluated by dynamic analysis of the structure, the internal forces are determined by static analysis of the structure at each time instant. At any instant of time t the equivalent static force f_s is the external force that will produce the deformation u determined by dynamic analysis.

The chapter deals with the basic understanding of the response quantities like displacement, velocity and acceleration. In subsequent section, equation of motion for SDOF system subjected to ground excitation is derived and is solved by using numerical method (Newmark-Beta) through MATLAB. Also, other sections discuss in detail about generation of Response spectrum along with its characteristics by considering El Centro ground excitation.

Response spectrum is an important tool in the seismic analysis and design of structures. It provides a convenient means to summarize the peak response of all possible linear SDOF systems to a particular component of ground motion. It also provides a practical approach to apply the knowledge of structural dynamics to the design of structures and development of lateral force requirements in building codes. The response spectrum is a plot of the peak values of a response quantity as a function of the natural vibration period Tn of the system. Each such plot is for a SDOF systems having a fixed damping ratio ζ, and several such plots for different values of ζ are included to cover the range of damping values encountered in actual structures. A variety of response spectra are defined depending on the response quantity that is plotted.

Deformation Response Spectrum

For designing a structure, lateral force is most important, which is evaluated if the maximum relative displacement u is known. A displacement response spectrum is the plot of maximum displacement of a SDOF to a particular ground motion as a function of the natural frequency and damping ratio of the SDOF as shown in Figure 3.1. It provides necessary information to compute the peak values of deformation and internal forces. It is denoted by D. Mathematical form is given as

$$D = max|u(t)| \tag{3.1}$$

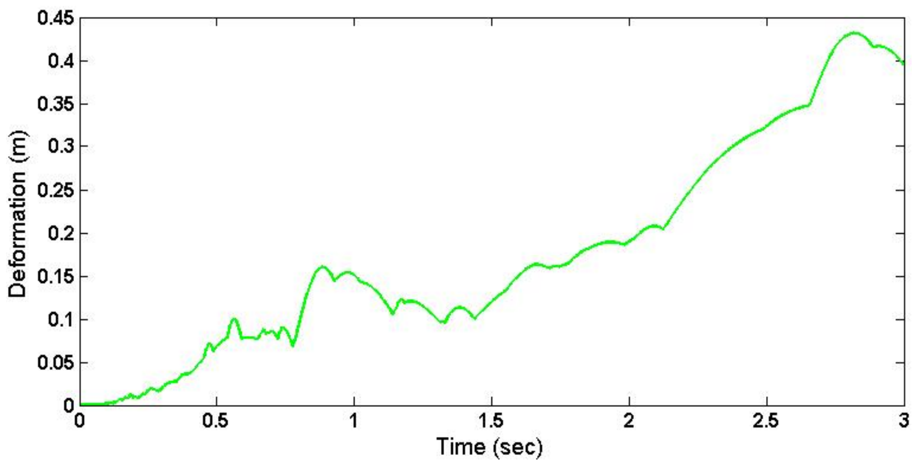


Figure 3.1: Deformation Response Spectrum for El Centro Ground Motion ($\zeta = 0.02$)

Pseudo-velocity Response Spectrum

Let a pseudo-velocity be V. Now the kinetic energy associated with it is equal to the maximum strain energy of the spring, $\frac{1}{2}mV^2 = \frac{1}{2}kD^2$. From the relation $k = m\omega_n^2$, it gives

$$V = \omega_n D \tag{3.2}$$

The pseudo-velocity response spectrum is a plot of pseudo-velocity (V) as a function of the natural vibration period T_n or structural frequency of the system as shown in Figure 3.2

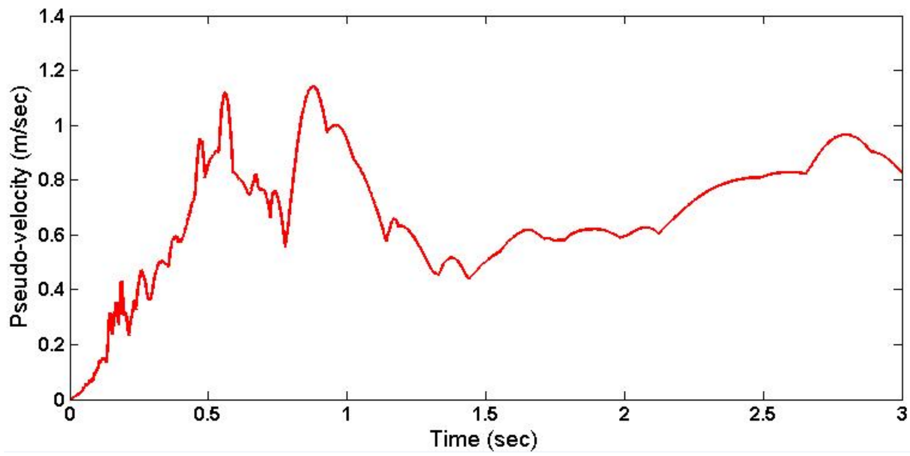


Figure 3.2: Pseudo-velocity Response Spectrum

Pseudo-acceleration Response Spectrum

Consider the spring force-displacement relationship $f_s = ku$. If relative displacement u is known, spring force f_s is obtained. Now consider f_s as a pseudo-inertia force, which can be written in terms of the pseudo acceleration 'a' as ma. As we have the relationship, $ma = f_s = ku$, it gives $a = \frac{k}{m}u = \omega_n^2 u$. It is written as

$$A = \omega_n^2 D \tag{3.3}$$

The pseudo-acceleration response spectrum is a plot of 'A' as a function of the natural vibration period T_n or structural frequency of the system as shown in Figure 3.3.

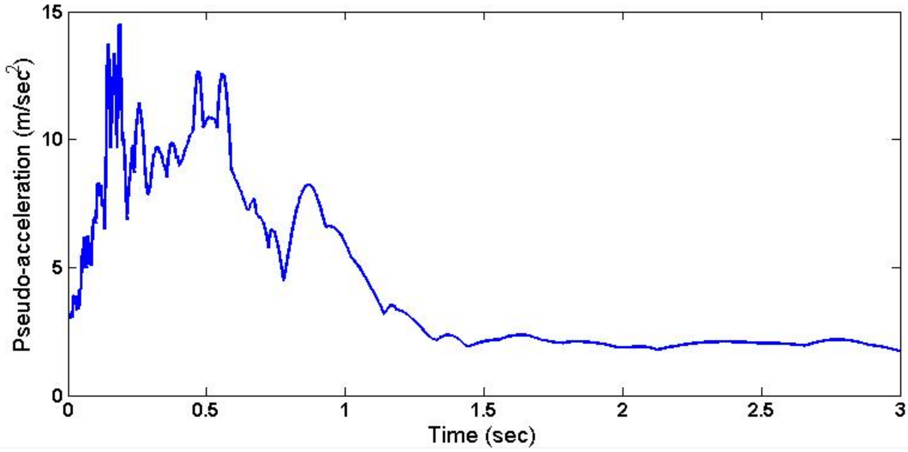


Figure 3.3: Pseudo-acceleration Response Spectrum

3.2 Equation of Motion

Consider a one storey structural model that has only one degree of freedom i.e. the lateral displacement of the girder as shown in Figure 3.4 (a) and Figure 3.4 (b). Under the action of the earthquake ground motion, $\ddot{u}_g$, the structure deforms. From Figure 3.4(b), f_I denotes the inertia force, f_S the spring force and f_D denotes the damping force.

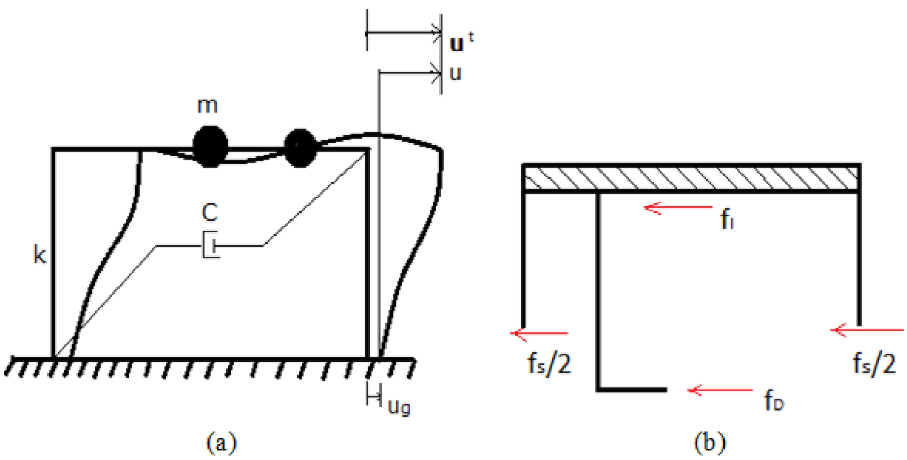


Figure 3.4: (a) SDOF system subjected to ground motion (b) Free Body Diagram

According to Newton's second law of motion, a dynamic system is in equilibrium at each time instant. The displacement of ground is denoted by u_g, the total or absolute displacement of mass by u^t and the relative displacement between the mass and ground by u at each instant of time. These displacements are related by,

$$u^t(t) = u(t) + u_g(t) \tag{3.4}$$

The equation of motion for the SDOF system subjected to earthquake excitation can be derived by concept of dynamic equilibrium from the free body diagram. The equation of dynamic equilibrium is,

$$f_I + f_D + f_S = 0 \tag{3.5}$$

As the structure is linearly elastic, therefore elastic resisting force is,

$$f_S = ku \tag{3.6}$$

The viscous damping force f_D is assumed to vary linearly with relative velocity $c\dot{u}$, so for a linear system the damping force is,

$$f_D = c\dot{u} \tag{3.7}$$

The inertia force is equal to the product of mass times its acceleration and acts opposite to the direction of acceleration. It is related to the total acceleration $\ddot{u}^t$ at the mass by,

$$f_I = m\ddot{u}^t \tag{3.8}$$

Substituting Equation 3.6, 3.7 and 3.8 in Equation 3.5 and using Equation 3.4

$$m\ddot{u}^t + c\dot{u} + k(u) = 0 \tag{3.9}$$

$$m\ddot{u}(t) + c\dot{u}(t) + ku(t) = -m\ddot{u}_g(t) \tag{3.10}$$

It is the equation of motion governing the relative displacement or deformation u(t) of the linear structure subjected to ground acceleration $\ddot{u}_g(t)$ where m,c, and k are the mass, damping and stiffness respectively of the system. Thus the relative displacement u(t) of the structure due to ground acceleration $\ddot{u}_g(t)$ will be identical to the displacement u(t) of the structure if its base were stationary and if it were subjected to an external force $-m\ddot{u}_g(t)$. Dividing the above equation by m and using basic relationships of structural dynamics; $k = m\omega_n^2$ and $c = 2m\omega_n\zeta$, we get

$$\ddot{u}(t) + 2\zeta\omega_n\dot{u}(t) + \omega_n^2 u(t) = -\ddot{u}_g(t) \tag{3.11}$$

For a given ground motion $\ddot{u}_g(t)$, the deformation response u(t) of SDF system depends only on the natural vibration period of the system and its damping ratio. Thus any two systems having the same values of T_n and ζ will have the same deformation response u(t) even though one system may be more massive than the other or one may be stiffer than the other. Ground acceleration during earthquakes varies irregularly therefore analytical solution of the equation of motion is carried out by using numerical methods.

3.2.1 Solution of Equation of Motion using Numerical Method

Analytical solution of the equation of motion for SDOF system given by Equation 3.10 is usually not possible if the excitation-applied force or ground acceleration varies arbitrarily with time. Such problem can be solved by the numerical time-stepping methods for integration of differential equations. There are two basic approaches to numerically evaluate the dynamic response. The first approach is numerical interpolation of the excitation and the second is numerical integration of the equation of motion. Both approaches are applicable to linear systems but the second approach is related to non-linear systems.

Many numerical integration methods are available for the solution of equation of motion specified in previous section. All the numerical integration method have two basic characteristics. First, they do not satisfy differential equation at all time t, but only at discrete time intervals, say Δ(t) apart. Secondly, within each time interval Δ(t), a specific type of variation of the displacement u, velocity $\dot{u}$,and acceleration $\ddot{u}$ is assumed. Thus, several numerical integration methods are available depending on the type of variation assumed for u, $\dot{u}$ and $\ddot{u}$ within each time interval Δt.

Time stepping Methods

Equation of motion in the case of base excitation due to earthquake is given as,

$$m\ddot{u}(t) + c\dot{u}(t) + ku(t) = -m\ddot{u}_g(t) \tag{3.12}$$

Now, subject to initial conditions u_0=$u(0)$; and $\dot{u_0}$=$\dot{u}(0)$ usually the system is assumed to have a linear damping. The applied force at discrete time intervals and the time increment Δt_i = t_{i+1} - t_i is usually taken to be constant, although this is not necessary. The response is determine at discrete time instants t_i, denoted as time i; the displacement, velocity, and acceleration at the i^{th} step are denoted by u_i, $\dot{u}_i$ and $\ddot{u}_i$ respectively. These values are assumed to satisfy Equation 3.12 at time $i:$ as,

$$m\ddot{u}_i + c\dot{u}_i + ku_i = p_i \tag{3.13}$$

Where ku_i is the resisting force at time i; $(f_S)i$=ku_i for a linearly elastic system but will depend on the prior history of displacement and velocity at time i if the system is inelastic. In subsequent section numerical procedure is presented, which enable us to determine the response quantities u_{i+1}, $\dot{u}_{i+1}$ and $\ddot{u}_{i+1}$ at time (i+1) step that satisfy Equation 3.12 at time i+1:

$$m\ddot{u}_{i+1} + c\dot{u}_{i+1} + ku_{i+1} = p_{i+1} \tag{3.14}$$

If the numerical procedure is applied successively with $i = 0$, 1, 2, 3,....the time stepping procedure gives the desired response at all time instants with the known initial conditions u_0 and $\dot{u}_0$.

Types of Time Stepping Methods

Three types of time stepping procedures are as follows:

1) Method based on the interpolation of the excitation function.

2) Method based on finite difference expressions for the velocity and acceleration.

3) Method based on the assumed variation of acceleration.

In a direct integration method, the system of equation of motion is integrated successively by using step by step numerical method. No transformation of equation of motion is needed prior to integration and using difference formulas that involve one or more increments of time usually approximates time derivatives. Basically two principle approaches used in the direct integration method: Explicit and implicit schemes. In an explicit scheme, the response quantity are expressed in terms of previously determined value of displacement, velocity, and acceleration. In an implicit scheme the difference equations are combine with the equation of motion, and the displacements are calculated directly by the solving the equation.

Newmark Beta Method [1]

The well known Newmark direct integration method is quite often used to compute the structural response, and hence in this section a procedure that incorporates the Newmark type numerical scheme in solving the equation of motion under the earthquake excitations is given in brief.

This method is based on the assumption that the acceleration varies linearly between two instants of time. Two parameters α and β are used in this method, which can suit the requirement of the particular problem. In order to illustrate the use of this numerical integration method, consider the solution of linear dynamic equilibrium equations of motion as given in Equation 3.14. Newmark developed a family of time-stepping methods based on the following equations:

$$\dot{u}_{i+1} = \dot{u}_i + [(1-\gamma)\Delta t]\ddot{u}_i + (\gamma\Delta t)\ddot{u}_{i+1} \tag{3.15}$$

$$u_{i+1} = u_i + (\Delta t)\dot{u}_i + [(0.5-\beta)(\Delta t)^2]\ddot{u}_i + [\beta(\Delta t)^2]\ddot{u}_{i+1} \tag{3.16}$$

Newmark used Equations 3.14, 3.15 and 3.16 iteratively for each time step to obtain displacement of the structural system. The parameter β and γ define the variation of acceleration over a time step and determine the stability and accuracy characteristics of the method. Typical selection for γ is 1/2 and $1/6 \leq \beta \leq 1/4$ is satisfactory from all point of view, including that of accuracy. These two equations, combined with the equilibrium Equation 3.14 at the end of the time step, provide the basis for computing u_{i+1}, $\dot{u}_{i+1}$ and $\ddot{u}_{i+1}$ at time (i+1) from the known u_i, $\dot{u}_i$ and $\ddot{u}_i$ at time i. Iteration is required to implement these computations because the unknown $\ddot{u}_{i+1}$ appears in the right side of Equation 3.15 and 3.16. When $\gamma = 1/2$ and $\beta = 1/6$, Equations 3.15 and 3.16 correspond to the linear acceleration method. When $\gamma = 1/2$ and $\beta = 1/4$, this correspond to the assumption that the acceleration remain constant. The complete algorithm using the Newmark Beta integration method is given in Table 3.1.

Table 3.1 Newmark's Direct Integration Method[18]

---

1) Initial calculation

(1.1) Form stiffness matrix [k], mass matrix [m] and damping matrix [c]

(1.2) Specify integration parameter γ and β

(1.3) Specify initial conditions u_0, $\dot{u}_0$, $\ddot{u}_0$

(1.4) $\ddot{u}_0 = \frac{p_0 - c\dot{u}_0 - ku_0}{m}$

(1.5) Select Δt time interval

(1.6) Calculate modified stiffness, $\widehat{k} = \mathrm{k} + \frac{\gamma}{\beta\Delta t}\mathrm{c} + \frac{1}{\beta(\Delta t)^2}\mathrm{m}$

(1.7) Calculate constants, a = $\frac{1}{\beta \Delta t}$m + $\frac{\gamma}{\beta}$c; and b = $\frac{1}{2\beta}$m + Δt($\frac{\gamma}{2\beta}$-1)c.

2) Calculation for each time step, *i*

(2.1) $\Delta\ \widehat{p_i} = \Delta p_i$ + a$\dot{u}_i$ + b$\ddot{u}_i$

(2.2) $\Delta u_i = \frac{\Delta \widehat{p}_i}{\widehat{k}}$

(2.3) $\Delta\ \dot{u}_i = \frac{\gamma}{\beta \Delta t}\Delta u_i$ - $\frac{\gamma}{\beta}\ \dot{u}_i$ + Δt(1-$\frac{\gamma}{2\beta}$)$\ddot{u}_i$.

(2.4) $\Delta\ \ddot{u}_i = \frac{1}{\beta(\Delta t)^2}\Delta u_i$ - $\frac{1}{\beta \Delta t}\ \dot{u}_i$ - $\frac{1}{2\beta}\ddot{u}_i$

(2.5) $u_{i+1} = u_i + \Delta u_i$, $\dot{u}_{i+1} = \dot{u}_i + \Delta \dot{u}_i$ and $\ddot{u}_{i+1} = \ddot{u}_i + \Delta \ddot{u}_i$

3) Repetition for the next time step. Replace *i* by *i* + 1 and implement steps 2.1 to 2.5 for the next time step.

For the ground acceleration excitation $\ddot{u}_g$(t), replace p_i by -m$\ddot{u}_{gi}$ in Table 3.1. The computed u_i, $\dot{u}_i$, and $\ddot{u}_i$ gives response value like displacement, velocity and acceleration relative to the ground. Total velocity and total acceleration can be computed from $\dot{u}_i^t = \dot{u}_i + \dot{u}_{gi}$ and $\ddot{u}_i^t = \ddot{u}_i + \ddot{u}_{gi}$, respectively.

3.3 Generation of Response Spectrum

In this section, method to construct Response Spectrum by obtaining response quantities is shown under El Centro earthquake excitation. In order to obtain response quantity, equation of motion given by Equation 3.12 is solved using Newmark-Beta method discussed in Section 3.2 through MATLAB.

The response spectrum for El Centro ground motion component $\ddot{u}_g(t)$ is developed by implementing following steps :

1. Collect the ground motion data of an El Centro earthquake. Define the ground acceleration $\ddot{u}_g(t)$ numerically. This ground motion ordinates are defined at time interval of 0.02 second.

2. Select the natural vibration period T_n and damping ratio ζ of a SDOF system.

3. Compute the deformation response u(t) of this SDOF system due to the ground motion $\ddot{u}_g(t)$ by any of numerical methods such as Newmark-Beta method, Runge-Kutta method etc.

4. Determine maximum deformation (u_o) which is the peak value of relative deformation u(t).

5. Determine the spectral ordinates using relation $V = \omega_n D$ and $A = \omega_n^2 D$.

6. Repeat steps 2 to 5 for a different range of T_n and ζ values which covers all possible systems of engineering interest.

7. Present the results of steps 2 to 6 graphically to produce three separate spectra.

In order to validate solution technique adopted to determine response spectrum, an example of SDOF system subjected to N-S Component of El Centro ground motion available in Chopra [1] is consider. The response spectrum is generated following procedure mentioned above and is compared with Chopra [1].

1. Ground motions are usually measured by accelerographs and expressed in the form of accelerograms. The ground acceleration is defined by numerical values at discrete time instants. These time instants should be closely spaced to describe accurately the highly irregular variation of acceleration with time. Typically, the time interval is chosen to be 1/100 to 1/50 of a second, which contains 1500 to 3000 ordinates to describe the ground motion.

   Figure 3.5shows the time history plot of an El Centro earthquake excitation. Here ground motion ordinates are defined at 0.02sec. Peak ground acceleration is $\ddot{u}_{go} = 3.1276$ (m/sec^2).

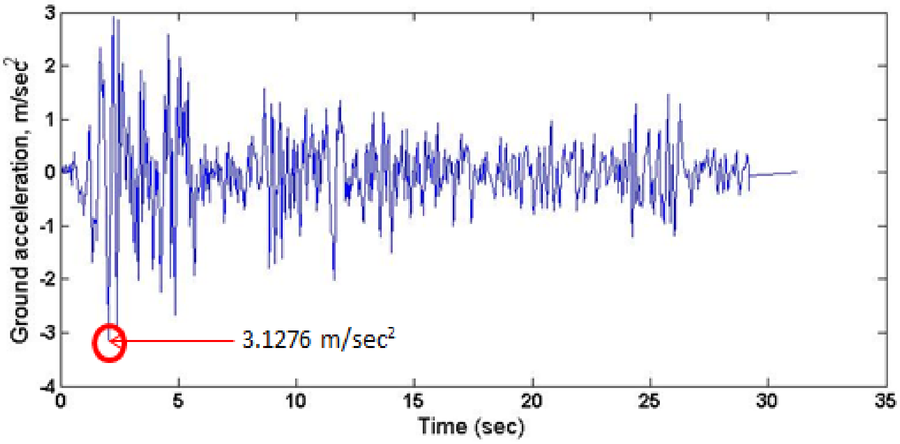


Figure 3.5: North-South component of ground acceleration recorded at Imperial Valley District, El Centro, 1940

2. Consider various SDOF systems with different Tn, but the same ζ subjected to El Centro ground excitation. It includes 3000 possible SDOF systems for damping of 2%. Here three SDOF systems considered for a damping value of 2%, Tn = 0.5s, 1s, 2s is shown in Figure 3.6.

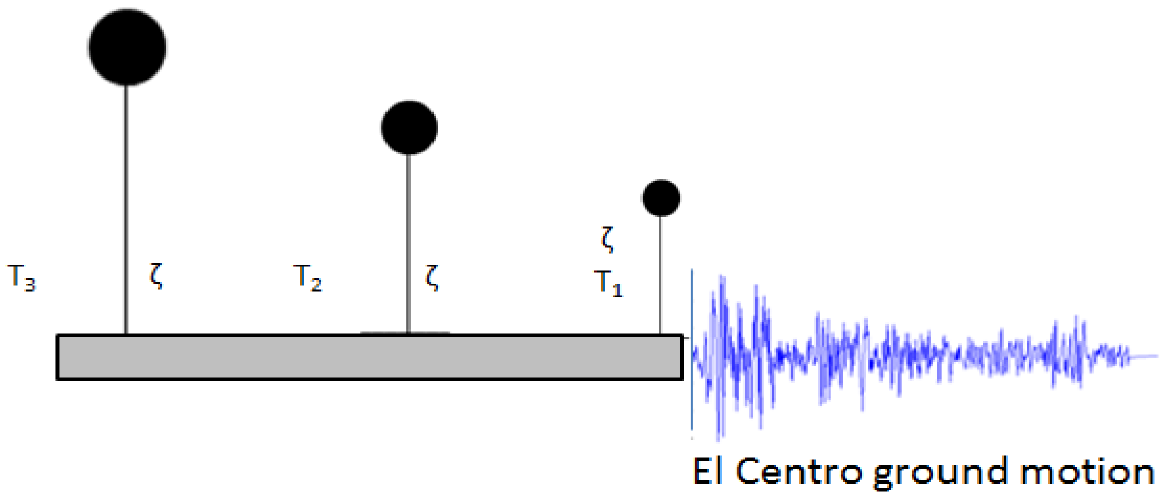


Figure 3.6: Different SDOF systems under earthquake ground excitation

3. The deformation response u(t) of this three SDF system due to the ground motion $\ddot{u}_g(t)$ is computed by using Newmark-Beta method.

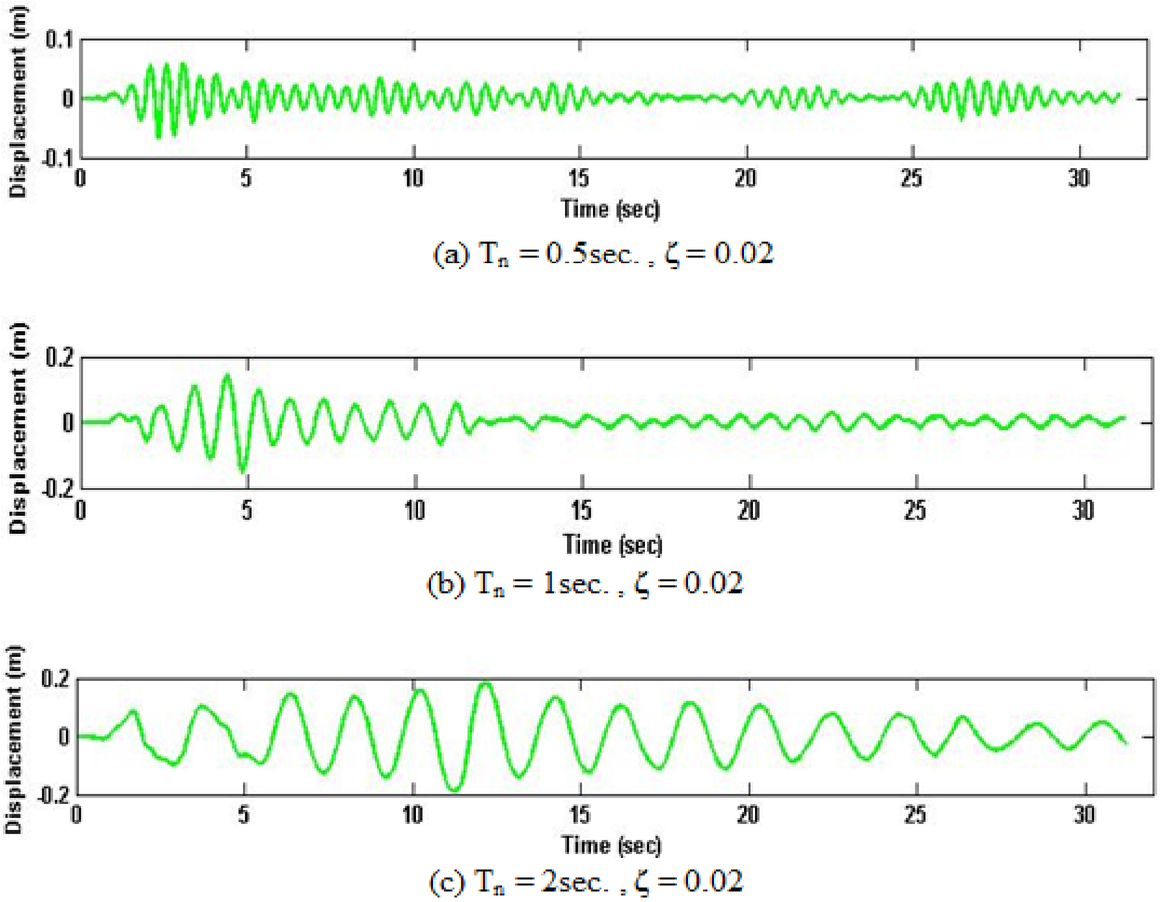


Figure 3.7: Deformation response of three SDOF systems with $\zeta = 2\%$ and Tn = 0.5,1 and 2sec. of El Centro ground motion

The time variation of the deformation induced by El Centro ground motion in three SDF systems is presented in Figure 3.7.

The damping ratio, $\zeta = 2$ %, is taken same for the three systems so that only the differences in their natural periods are responsible for the large differences in the deformation responses. It is observed that the time required for SDOF system to complete a cycle of vibration when subjected to this earthquake ground motion is very close to the natural period of the system.

4. For each system the peak value of deformation $D = u_o$ is determined from the deformation response history. Usually, the peak occurs during ground shak-

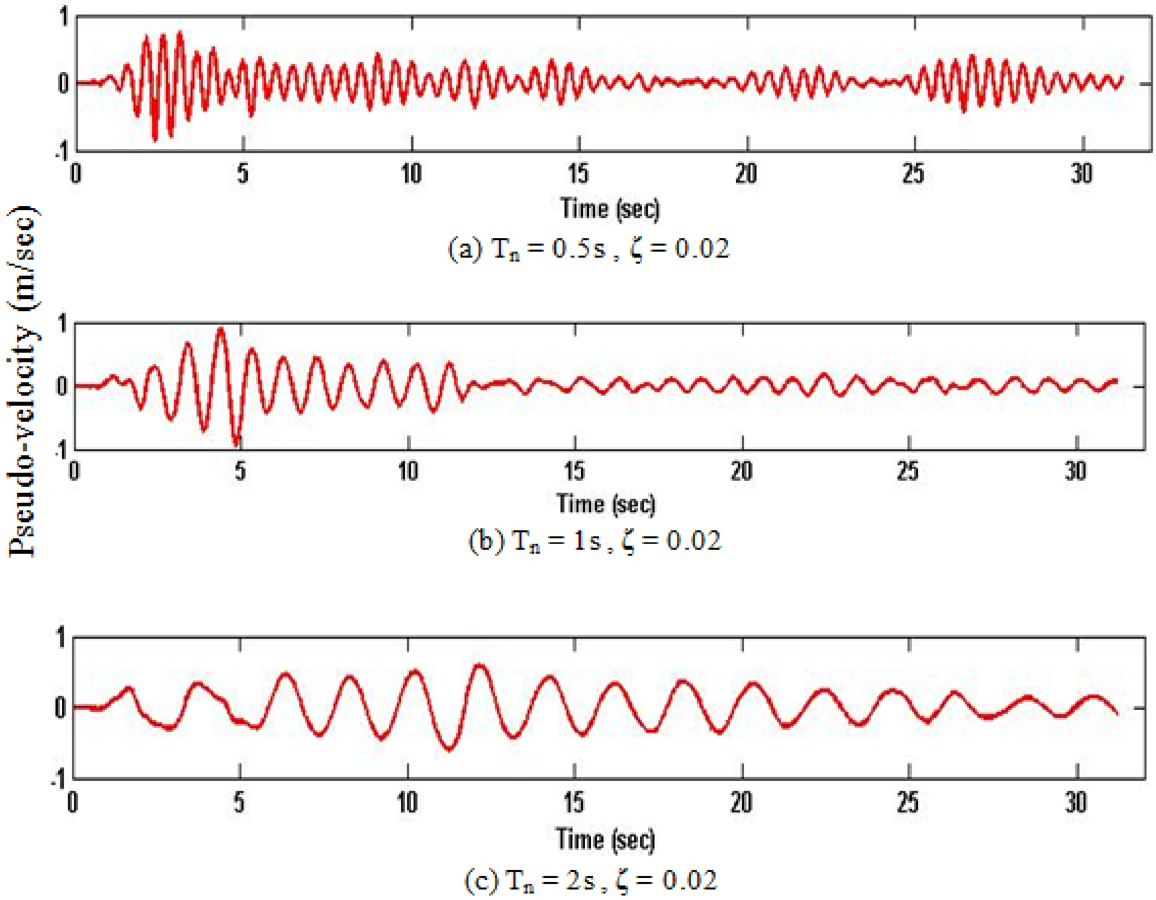


Figure 3.8: Pseudo-velocity response of three SDOF systems with $\zeta = 2\%$ and Tn = 0.5,1 and 2sec. of El Centro ground motion

ing, however for lightly damped systems with very long periods the peak response may occur during the free vibration phase after the ground shaking has stopped.Peak deformation response for three SDOF system is shown in Table 3.1.

5. Once the deformation response history u(t) has been evaluated by dynamic analysis of the structure, pseudo-velocity V(t) and pseudo-acceleration A(t) response of the system can be computed by using relation

$$\frac{A}{\omega_n} = V = \omega_n D \tag{3.17}$$

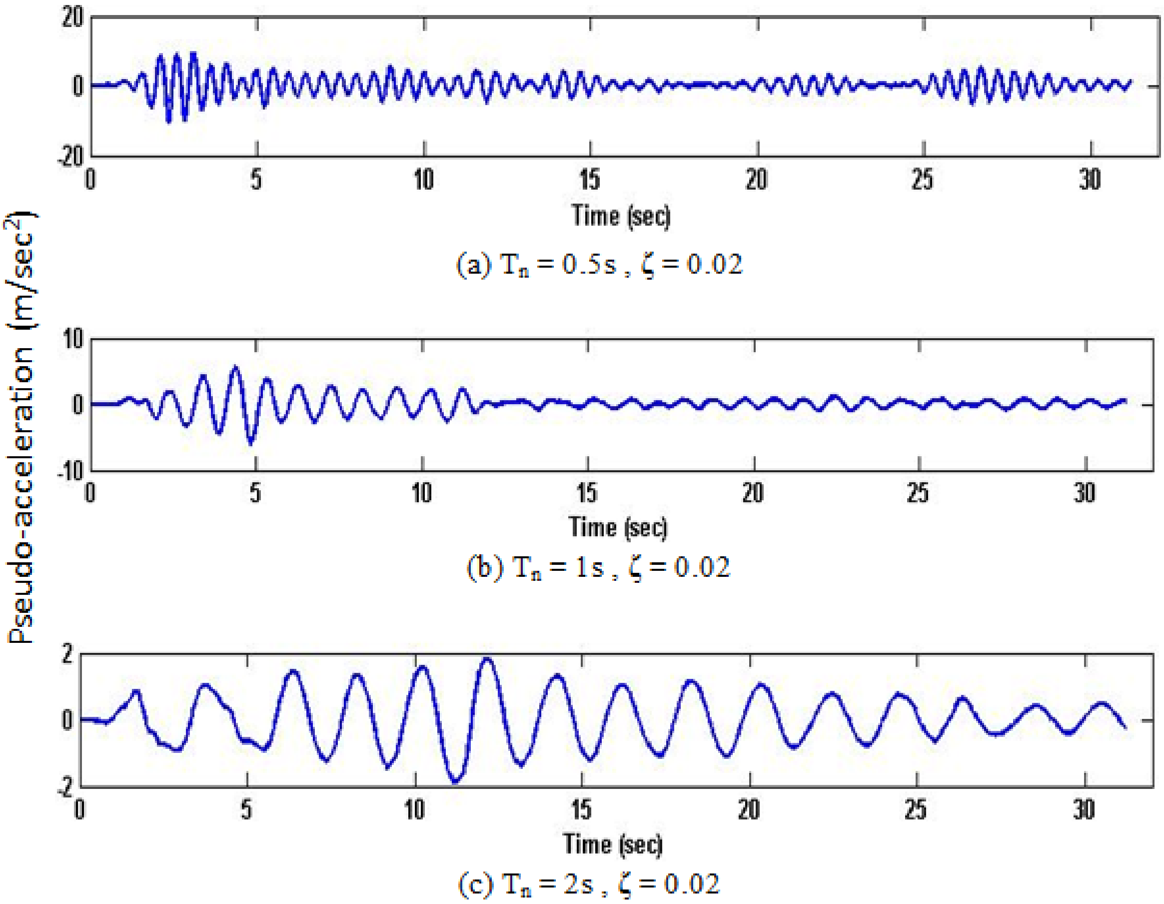


Figure 3.9: Pseudo-acceleration response of three SDOF systems with $\zeta = 2\%$ and Tn = 0.5, 1 and 2sec. of El Centro ground motion

Figure 3.8 and Figure 3.9 shows Pseudo-velocity and Pseudo-acceleration response spectrum for three SDOF systems. Table 3.1 shows the response quantities obtained under El Centro earthquake excitation.

It is observed that among these three systems, the longer the vibration period, the greater is the peak deformation. But it must be noted that this trend is neither perfect nor valid over the entire range of periods.

6. For each SDOF system, peak response is obtained for all three response quantities. Steps 2 to 5 are repeated for 3000 SDOF systems with each natural period taken at 0.001 sec. interval and with the damping of 2%.

Table 3.1: Response Quantity under El Centro earthquake

T_n	**Damping** ζ %	D (m)	V (m/sec)	A (m/sec^2)
0.5	2	0.0682	0.85549	10.75
1	2	0.15	0.9464	5.934
2	2	0.1899	0.5959	1.832

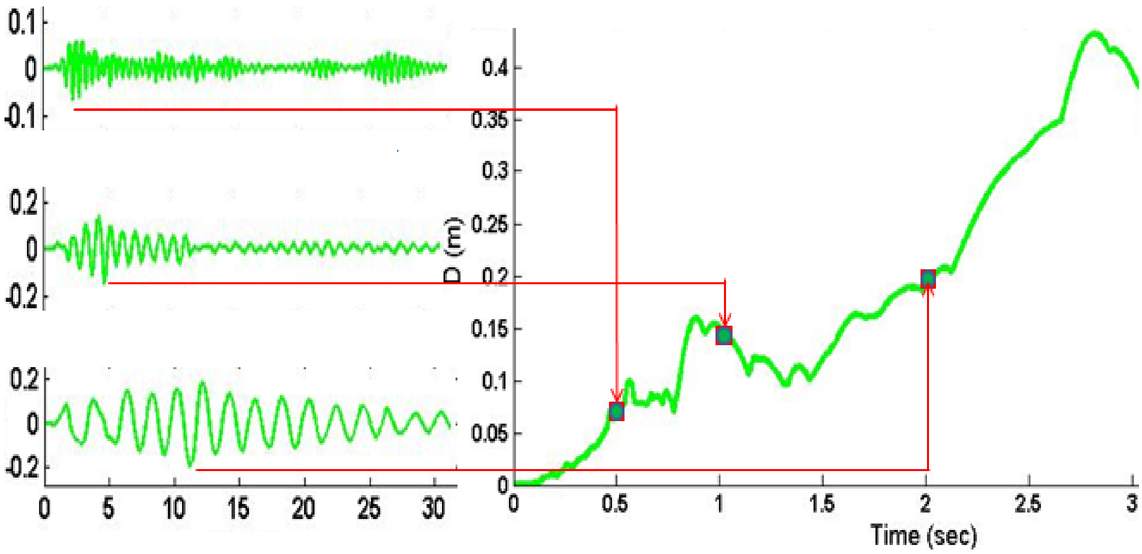


Figure 3.10: Deformation response spectrum of El Centro ground motion

7. A plot of peak value of response quantities for 3000 SDF systems subjected to El Centro ground motion is obtained which is shown in Figure 3.10, Figure 3.2 and Figure 3.3.

From Figure 3.10, we can directly read the maximum relative displacement of any structure of natural period Tn having damping value of 2%. Due to the direct relation from Equation 3.17, Pseudo-velocity and Pseudo-acceleration response spectrum is obtained.

3.4 Characteristics of Response Spectrum

A response spectrum is a convenient tool to assess the level of response that will induce in structures by a given ground motion which can be modelled as a single-degree-of-freedom system. It also provides a means to identify which structures will be affected the most by the given ground motion. The differences in the ground motions recorded at different sites and on different soils may be evaluated by a comparison of their response spectra.

Because of its usefulness, the concept of response spectrum constitutes the basis of many of the methods used in the analysis of earthquake resistant structures and the formulation of the design recommendations in building codes.

Response spectrum shows few characteristics which are worth consideration. They are briefly presented here.

1. Figure 3.11 shows that with increment in damping values, peaks of response spectrum reduces to a large extent of the ground motion. However, the amount of this reduction depends on various factors, which includes the period of the structure and the frequency content. Because of the basic characteristics of response spectrum at very short and very long periods, viscous damping does not have much influence in these period ranges. While in the intermediate period range, damping has its greatest effect on the response reduction. Figure 3.11 shows response spectrum for El Centro ground motion for different damping ratios. At damping values of 10% and more response spectrum shows smoothen curve.

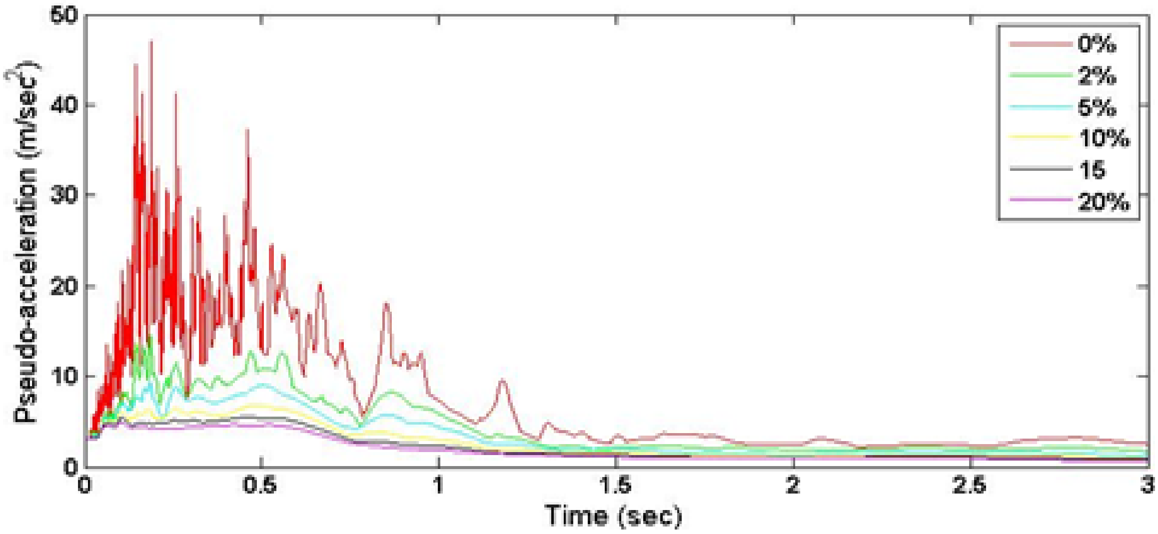


Figure 3.11: Normalized pseudo-acceleration response spectrum for El Centro ground motion for different damping (ζ =0%, 2%, 5%, 10%, 15%, 20%)

2. The difference between relative velocity and pseudo-velocity spectrum shows dependence on natural period of the system. Figure 3.12 shows velocity response spectrum where relative velocity ($\dot{u}$) and pseudo-velocity (V) is plotted against natural time period of all SDOF system. It is evident from Figure 3.12 that the difference between $\dot{u}$ and V are generally negligible for most of the typical natural period and damping ranges of engineering interest except for long period systems. For long period systems, V is less than $\dot{u}_o$ and difference between two are large. It can be understood by recognizing that as T_n becomes long, mass of the system stays still while the ground underneath moves. The absolute (or total) deformation of the mass will become very small and consequently the relative deformation of the mass with respect to the ground will approach the ground displacement. So, as $T_n \to \infty$, $D \to u_{go}$ and $\dot{u}_o \to \dot{u}_{go}$. Now, $D \to \dot{u}_{go}$ implies that $V \to 0$ as per Equation 3.17. And also for short period systems, V exceeds $\dot{u}_o$ and its difference increases as T_n becomes shorter.

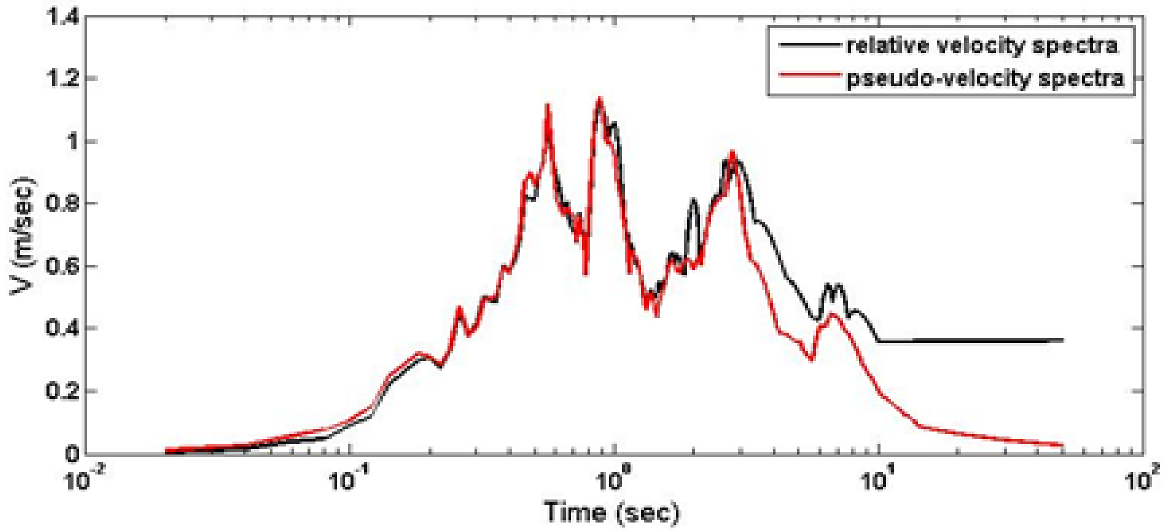


Figure 3.12: Comparison of pseudo-velocity and relative-velocity response spectru

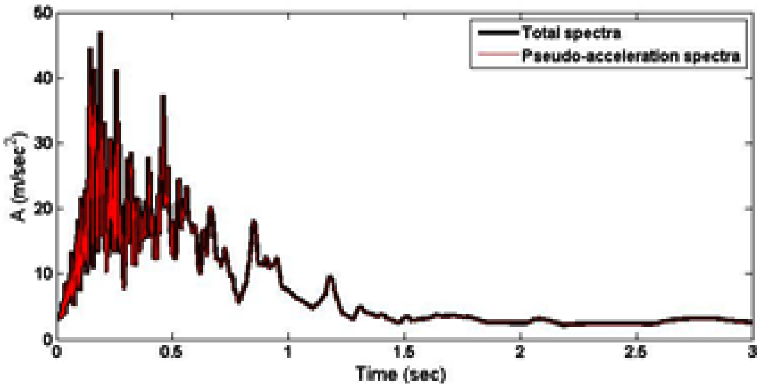
3. Figure 3.13 shows acceleration response spectrum where total acceleration ($\ddot{u}_o^t$) and pseudo-acceleration (A) is plotted against natural time period of all SDOF system with damping $\zeta = 0\%, 5\%, 10\%$ respectively. It is evident from Figure 3.13 (a) that, pseudo-acceleration and acceleration response spectra are identical for systems without damping. This can be proved as below

   Equation of motion is

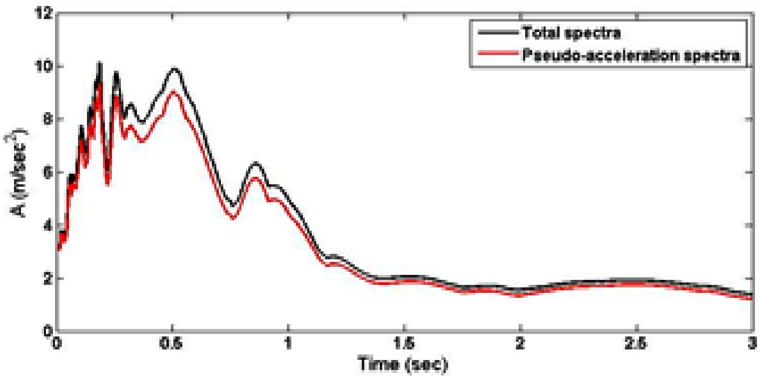
$$m\ddot{u}(t) + c\dot{u}(t) + ku(t) = -m\ddot{u}_g(t)$$

   But as c=0, therefore,

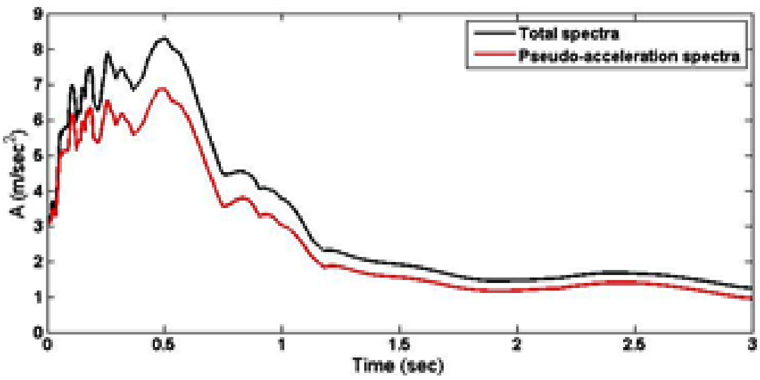
$$m\ddot{u}(t) + ku(t) = -m\ddot{u}_g(t)$$



(a) ζ = 0%



(b) ζ = 5%



(c) ζ = 10%

Figure 3.13: Normalized pseudo-acceleration response spectrum for El Centro ground motion for different damping (ζ =0%, 2%, 5%, 10%, 15%, 20%)

Dividing this equation by mass of the system and from the relation $\omega_n = \sqrt{\frac{k}{m}}$ taking $\ddot{u}_g$ on left side, we get

$$(\ddot{u} + \ddot{u}_g) = -\omega_n^2 u$$

From Equation 3.3, we have

$$\ddot{u}^t = -\omega_n^2 u = A$$

As damping increases difference between pseudo-acceleration and acceleration response spectra increases. This is because of fact that pseudo-acceleration is a derived quantity and depends on damping while total acceleration is calculated from Equation 3.10.

Tripartite Response Spectrum

The three response spectrum D, V and A for a given ground motion contains the same information, only the way of presenting them is different. If any one of the spectra is known, the other two are obtained by algebraic operations using Equation 3.2 and Equation 3.3. Each spectrum directly provides a physically meaningful quantity i.e. the deformation spectrum provides the peak deformation of a system, the pseudo-velocity spectrum is related directly to the peak strain energy stored in the system during the earthquake and the pseudo-acceleration spectrum is related directly to the peak value of the equivalent static force and base shear. So it is especially useful to show all of the three spectral quantities in a combined plot known as the tripartite plot.

Figure 3.14 demonstrates the computed Tripartite response spectrum of different damping values from the time history shown in Figure 3.5. In this figure all spectral quantities, like displacement, velocity and acceleration are displayed in a single graph (on log-log scale), known as the tripartite graph. The computed response spectrum shows correct behaviour at both short and long periods, that is, the pseudo-acceleration (A) approaches PGA at short periods and the relative deformation (D) approaches PGD at long periods. The pseudo-velocity (V) is read along the vertical axis, the pseudo-acceleration (A) is read along the -45 axis, and the relative deformation is read along the +45 axis, with respect to the natural period T along the horizontal axis. These quantities are related to each other as shown in Equation 3.17

For plotting tripartite graph, El Centro time history is taken as an example which has its PGA, PGV and PGD values as 3.1294 m/sec^2, 0.3312 m/sec and 0.21336 m respectively. From the figure it is seen that Ta marks the boundary between the high and intermediate frequency regions and Tb marks the boundary between the intermediate and low regions. The values of Ta and Tb are calculated using the relation as shown in Equation 3.17. Response quantities A and D obtained by solving equation of motion under earthquake excitation using Newmark-Beta method are converted to V using relation through MATLAB in order to show them in a single graph on a log-log scale.

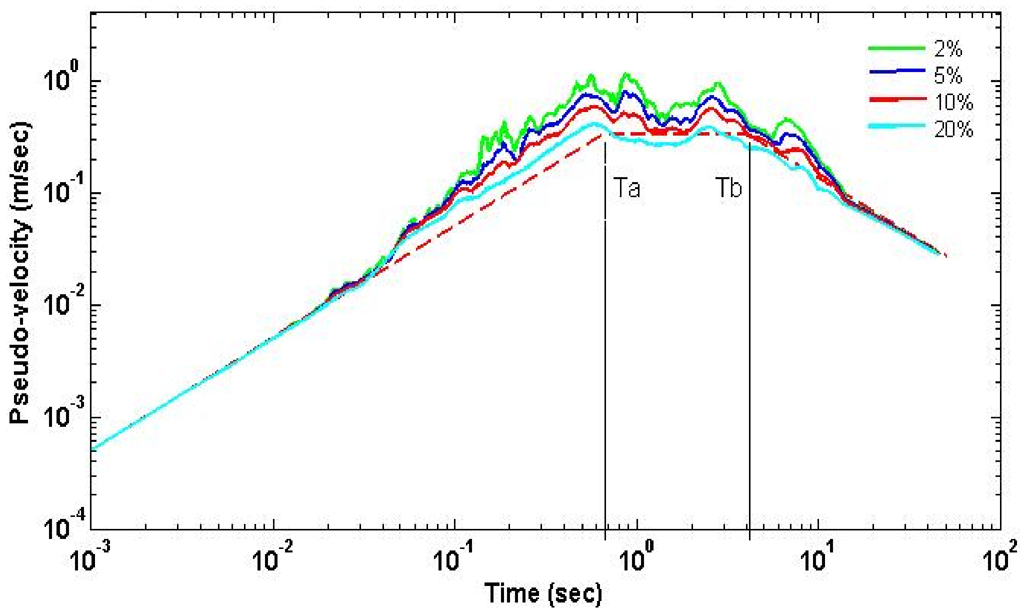


Figure 3.14: Combined D-V-A response spectrum for El Centro ground motion; ζ = 2%, 5%, 10% and 20%

3.5 Summary

The chapter deals with the generation of Response spectrum by solving equation of motion for SDOF system subjected to El Centro ground excitation. Newmark-Beta method is used for solving equation of motion through MATLAB. Response spectrum characteristics are also studied for basic understanding of response quantites like displacement, velocity and acceleration.

Chapter 4

Strong Ground Motion Parameters

4.1 General

Estimating seismic ground shaking is an important aspect in anticipating earthquake effects on people and structures. Every earthquake is a unique event, characterized by its intensity, duration and dominant periods. If an earthquake occurs in future, it cannot be represented from a previous earthquake even if it occurs at the same location and the ground motion is recorded at the same site. So a single ground motion characteristic is not sufficient to describe the effect that a ground motion will have on the response of a structure. Hence several parameters are used to characterize ground motions for design purposes.

This chapter presents the engineering characteristics, evaluation and selection process for strong ground motions based on the various parameters such as RMS acceleration, peak ground acceleration (PGA) and strong motion duration. The strong ground motions of various regions of Indian subcontinent are identified and compiled.

4.2 Characteristics of Ground Motion

Ground motion parameters are essential for describing the important characteristics of strong ground motion. Many parameters are proposed to characterize the strong ground motions. Amplitude, Frequency content and Duration of strong ground motion are the important characteristics for which the parameters are defined.

1) **Amplitude parameters**
   - Peak ground acceleration (PGA)
   - Peak ground velocity (PGV)
   - Peak ground displacement (PGD)
2) **Frequency content parameters**
   - Fourier spectra
   - Response spectra
3) **Duration of strong ground motion**
4) **Other parameters**
   - Root mean square acceleration (RMSa)
   - Arias intensity

The above parameters are briefly described as below :

1) **Amplitude parameters**

   Time history is the most common way to describe a ground motion. The ground motion parameters are acceleration, velocity and displacement. Out of the three amplitude parameters, only one of these is recorded directly and the others are computed from it by integration/differentiation. The acceleration time history displays more high frequency content (relatively), the velocity time history displays more intermediate frequency content (relatively), and the displacement displays more low frequency content (relatively). The peak acceleration provides a good indication of the high-frequency component of a ground motion. The peak velocity and peak displacement describe the amplitudes of the intermediate and low frequency components respectively.

2) **Frequency content**

It is generally described through the use of different types of spectra. Fourier spectra and power spectra directly illustrate the frequency content of the motion itself. Response spectra reflect the influence of the ground motion on structures of different natural periods. Since the frequency content of an earthquake motion will strongly influence the effects of that motion, characterization of the motion cannot be completed without consideration of its frequency content.

3) **Duration of motion**

The duration of strong ground motion have a strong influence on earthquake damage. An earthquake accelerogram generally contains all accelerations from the time the earthquake begins until the time the motion has returned to the level of background noise. For engineering purposes, only the strong motion portion of an accelerogram is of interest. Different approaches have been taken to evaluate the duration of strong motion in an accelerogram. Since the total duration of an accelerogram depends on the pre and post-event intervals, for digital records, it is not possible to define the duration of strong shaking as simply the time between the start and finish of an accelerogram. Many definitions of strong-motion duration have been proposed to isolate a certain portion of the accelerogram during which the strongest motion occurs. It is found that all of these definitions can be classified into one of three generic categories.

Bracketed duration

It is defined as the total time elapsed between the first and last excursions of a specified threshold acceleration.

Uniform duration

It is defined as the sum of the time interval during which the acceleration is greater than a given threshold.

Significant duration

It is defined as the time interval over which a portion of the total energy integral is accumulated. It is calculated as the integral of the square of the ground acceleration, velocity or displacement. If the integral of the ground acceleration is performed then the quantity is related to the Arias intensity, AI. It is defined as

$$AI = \frac{\Pi}{2g}\int_0^t a^2(t)dt \tag{4.1}$$

where a(t) is the acceleration time history, t is the total duration of the accelerogram and g is acceleration due to gravity.

4.3 Compilation of Ground motions for Indian subcontinent

With sufficient understanding developed from the work related to response spectrum as discussed in previous chapter, various earthquake excitations recorded at various places of the country, India are compiled. This are primarily available from authentic earth recording stations, web portal. Most of the time history data are available from NICEE, at IIT Kanpur. It is important to note that ground motion time histories data contain valuable characteristics and information.

A set of 184 Indian time histories (23 earthquake events) has been collected from different regions of the country for the detailed study. Table 4.1 shows various earthquake events at various recording stations in different regions of the country recorded by the instruments installed under the strong motion instrumentation programme. This programme was started in the mid-sixties by the Department of Earthquake Engineering, Indian Institute of Technology, Roorkee. Table 4.2 shows various earthquake ground motion records after 2005 of the country which are made available through web portal. Primarily the recorded time histories are grouped according to four zones North, East, South and West of the country which are shown in Table 4.3. It is seen from the Table 4.3 that large number of records are available for East region and North region. It is noted that records of strong earthquake ground motions are limited in number for West region followed by South region.

Table 4.1: Events recorded by Indian Strong Motion Instrument Network

Events	**Recording** Station	**Magnitude**	**PGA** (g)	**Recorded time** (sec)
Dharmsala	Bandlakhas	5.5	0.145	10.8
	Baroh		0.059	13.78
	Bhawarna		0.037	11.98
	Dharmsala		0.175	16.18
	Jawali		0.015	17.96
	Kangra		0.148	20.66
	Nagrotabagwan		0.149	20.3
	Shahpur		0.204	20.1
	Sihunta		0.051	17.62
North-East India	Baithalongso	5.2	0.045	12.56
	Dauki		0.089	17.9
	Khliehriat		0.031	13.4
	Nongkhlaw		0.055	29.64
	Nongpoh		0.054	14.08
	Nongstoin		0.019	8.54
	Panimur		0.039	11.02
	Pynursla		0.093	18.58
	Saitsama		0.113	20.66
	Ummulong		0.113	16.94
	Umrongso		0.027	11.76
	Umsning		0.101	20.06
India-Burma border 1987	Baithalongso	5.7	0.034	22.34
	Bamungao		0.019	29.48
	Berlongfer		0.072	42.76
	Bokajan		0.029	26.00
	Diphu		0.086	39.10
	Gunjung		0.042	16.04
	Haflong		0.055	13.54
	Hajadisa		0.078	16.56
	Hatikhali		0.031	36.22
	Laisong		0.042	16.78
	Nongpoh		0.017	20.48
	Panimur		0.04	10.96
	Saitsama		0.037	27.52
	Umrongso		0.02	12.24
India-Bangladesh border 1988	Baigao	5.8	0.022	12.92
	Baithalongso		0.03	11.72
	Bamungao		0.016	8.86
	Dauki		0.027	9.52
	Gunjung		0.036	13.02

Events	**Recording** Station	**Magnitude**	**PGA** (g)	**Recorded time** (sec)
	Haflong		0.035	10.12
	Hatikhali		0.024	11.80
	Katakhal		0.009	10.38
	Khliehriat		0.079	15.08
	Mawphlang		0.081	28.16
	Nongkhlaw		0.107	45.28
	Nongpoh		0.027	17.72
	Pynursla		0.049	34.60
	Saitsama		0.066	15.76
	Shillong		0.048	11.74
	Ummulong		0.056	24.52
	Umrongso		0.046	14.86
	Umsning		0.039	23.86
India-Burma border 1988	Baigao	6.8	0.0221	54.82
	Baithalongso		0.154	78.08
	Bamungao		0.093	38.58
	Berlongfer		0.301	119.70
	Bokajan		0.151	57.78
	Cherrapunji		0.052	21.28
	Dauki		0.108	34.84
	Diphu		0.282	81.74
	Doloo		0.064	38.26
	Gunjung		0.094	63.90
	Hajadisa		0.092	64.20
	Harengajao		0.065	30.50
	Hojai		0.108	63.78
	Jellalpur		0.029	15.86
	Jhirighat		0.107	42.34
	Kalain		0.057	29.70
	Katakhal		0.063	35.18
	Khliehriat		0.07	61.5
	Koomber		0.049	25.46
	Loharghat		0.058	38.36
	Mawkyrwat		0.046	22.68
	Mawphlang		0.119	52.14
	Mawsynram		0.085	23.70
	Nongkhlaw		0.142	70.98
	Nongstoin		0.052	52.96
	Panimur		0.168	72.06
	Pynursla		0.054	47.54

Events	**Recording** Station	**Magnitude**	**PGA** (g)	**Recorded time** (sec)
	Saitsama		0.211	81.10
	Shillong		0.075	34.78
	Silchar		0.064	46.80
	Ummulong		0.09	66.14
	Umrongso		0.076	64.74
	Umsning		0.122	70.60
India-Burma border 1990	Baigao	6.1	0.056	9.92
	Baithalongso		0.061	22.00
	Bamungao		0.029	14.62
	Berlongfer		0.145	62.84
	Diphu		0.092	32.24
	Gunjung		0.051	13.96
	Hajadisa		0.054	18.94
	Hojai		0.041	13.04
	Laisong		0.062	9.04
	Maibang		0.064	16.44
	Panimur		0.077	15.68
	Saitsama		0.062	26.52
	Ummulong		0.046	11.86
	Umrongso		0.036	15.22
Uttarkashi 1991*	Almora	6.5	0.018	21.34
	Barkot		0.095	31.74
	Bhatwari		0.253	36.16
	Ghansiali		0.118	42.34
	Karnprayag		0.062	22.26
	Kosani		0.029	13.36
	Koteshwar		0.101	33.70
	Koti		0.021	15.96
	Purola		0.075	35.70
	Rudraprayag		0.053	39.70
	Srinagar		0.067	41.10
	Tehri		0.073	31.96
	Uttarkashi		0.242	39.92
Chamba	Chamba	4.9	0.146	18.24
	Rakh		0.029	9.18
India-Burma border 1995	Baigao	6.4	0.057	12.18
	Bamungao		0.016	12.60
	Berlongfer		0.072	81.72
	Diphu		0.081	28.58
	Haflong		0.031	12.94

Events	**Recording** Station	**Magnitude**	**PGA** (g)	**Recorded time** (sec)
	Hatikhali		0.044	18.84
	Hojai		0.022	16.50
	Khliehriat		0.022	13.32
	Umrongso		0.023	15.96
Xizang-India border	Ukhimath	4.8	0.038	15.20
India-Bangladesh border 1997	Doloo	5.7	0.077	27.42
	Jellalpur		0.117	25.60
	Jowai		0.084	27.36
	Katakhal		0.107	26.58
	Nongpoh		0.048	47.38
	Nongstoin		0.048	39.02
	Pynursla		0.028	28.62
	Shillong		0.072	25.06
	Silchar		0.095	26.92
	Ummulong		0.155	28.66
	Umsning		0.077	27.34
Chamoli 1999*	Almora	6.4	0.027	9.04
	Barkot		0.017	14.98
	Chinaylisaur		0.052	25.68
	Ghansiali		0.073	26.32
	Gopeshwar		0.199	25.42
	Joshimath		0.071	25.06
	Lansdowne		0.005	7.12
	Roorkee		0.056	43.525
	Tehri		0.054	23.76
	Ukhimath		0.091	24.78
	Uttarkashi		0.054	14.76
Kachchh	Ahmedabad	7.0	0.106	133.525

Table 4.2: Ground motion data after 2005 available through web portal

Events	**Recording** Station	**Magnitude**	**PGA** (g)	**Recorded time** (sec)
Chamoli 2005*	Bageshwar	5.2	0.05425	29.995
	Chamoli		0.4113	44.61
	Champawat		0.0311	36.93
	Pauri		0.105	33.745
	Roorkee		0.02341	66.215
	Rudraprayag		0.21455	36.965
	Tehri		0.05708	31.435
	Uttarkashi		0.10588	39.095
Uttarkashi 2007*	Nathpa	5.0	0.04903	40.70
	Roorkee		0.01914	63.58
Andaman Islands 2008*	Port blair	6.7	0.22505	63.41
Nagaland	Tinsukia	5.1	0.02263	66.10
Uttarakhand	Champawat	5.1	0.01678	70.10
	Dharchula		0.04355	65.005
	Ghansiali		0.0185	65.015
	Joshimath		0.04852	68.51
	Kapkot		0.04722	67.435
	Munsiari		0.09464	70.085
	Pithoragarh		0.03444	76.56
Andaman Islands 2010*	Port blair	7.8	0.04073	181.435
India-Myanmar border (Manipur)	Coochbihar	6.4	0.03903	80.005
	Guwahati		0.18383	164.795
	Jorhat		0.03932	95.10
	Jowai		0.14172	93.05
	Khokhrajhat		0.06996	100.72
	Naogaon		0.32113	135.445
	Sibsagar		0.03212	67.795
Assam	Golaghat	5.4	0.08996	66.69
	Jorhat		0.04763	81.48
	Khokhrajhar		0.05925	110.995
Phek (Nagaland)	Golaghat	5.8	0.14703	76.555
	Jorhat		0.10267	97.29
	Tinsukia		0.04046	65.00
Kohima (Nagaland)	Golaghat	5.5	0.16254	79.475
	Jorhat		0.0901	128.87
	Naogaon		0.05932	84.595

* Two different records at the same location are included in the study.

Table 4.2 shows the ground motion records which took place after 2005. In order to describe accurately the highly irregular variation of acceleration, the time variation was choosen to be 0.005second and 0.02 second for the records shown in Table 4.1 and 4.2. Listing of the earthquakes in each region of the country are presented in Table 4.3.

Table 4.3: Classification of Earthquake records into four regions

Region	**Events**	**Recording Station**
East	North-East India	12
	India-Burma border 1987	14
	India-Bangladesh border 1987	18
	India-Burma border 1988	33
	India-Burma border 1990	14
	India-Burma border 1995	9
	India-Bangladesh border 1997	11
	Nagaland	1
	India-Myanmar border (Manipur)	7
	Assam	3
	Phek	3
	Kohima	3
North	Dharmsala	9
	Uttarkashi 1991	13
	Chamba	2
	Xizang-India border	1
	Chamoli 1999	11
	Chamoli 2005	8
	Uttarkashi	7
	Uttarakhand	7
South-East	Andaman Island 2008	1
	Andaman Island 2010	1
West	Kachchh	1

4.4 Evaluation of Strong Ground Motion

A number of parameters have been proposed to express the characteristics of strong ground motions. In the present study, parameters and characteristics taken into account are Duration of motion, RMS acceleration and PGA. Based on these parameters, ground motions are categorized as strong ground motions.

Peak ground acceleration (PGA)

PGA is the most commonly used measure of the intensity of shaking at a site and is taken to be the largest absolute value of the acceleration recorded at a site. It is defined mathematically as

$$PGA = max|a(t)| \tag{4.2}$$

where a(t) is the acceleration time history.

Ground motions with high peak accelerations are usually, but not always, more damaging than those with lower peak acceleration. Very high peak accelerations that last for only a very short period of time may cause little damage to many types of structures. Generally, PGA is a poor measure of ground-motion intensity which can be seen from Figure 4.1. Figure 4.1 shows the time histories having the same PGA. It is observed that the recorded duration is different and acceleration intensity also varies with time for various recording stations of past Indian recorded earthquake events. So only PGA as a parameter is not choosen but RMSa and duration parameters are also taken into account for selection of strong ground motions.

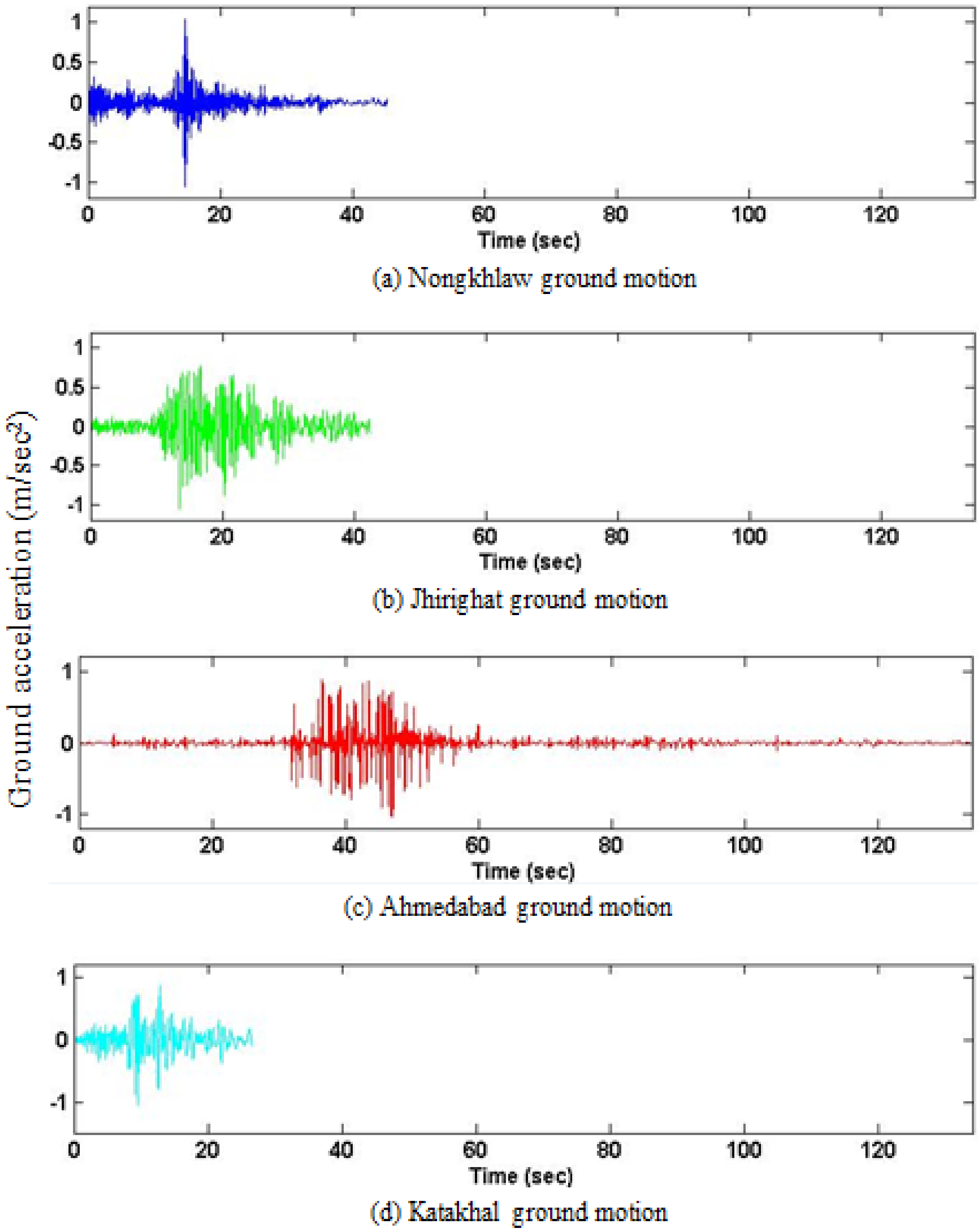


Figure 4.1: Ground motions with different recorded time and acceleration intensity

R.M.S. acceleration

It is a parameter that includes the effect of the amplitude and frequency content of a strong motion record. It is used as the basis for evaluation of strong motion duration. It is given in mathematical form as

$$a_{rms} = \sqrt{\frac{1}{T_d}\int_0^{T_d}[a(t)]^2 dt} \tag{4.3}$$

where a_{rms} is root mean square acceleration, T_d is duration of motion, a(t) is the acceleration time history.
a_{rms} value is calculated by using Simpson's $1/3^{rd}$ rule through MATLAB.

Duration

It is defined as the time between the first and last exceedances of a threshold acceleration. In this study, RMS acceleration is used as a basis for evaluation of strong motion duration.

Typical example of Port blair ground excitation is given here to show how duration of given event is determined. Recorded duration is 181.435 seconds. PGA value of this ground excitation is $0.04073 m/sec^2$. It is seen from the Figure 4.2 that strong motion duration is obtained as the time interval between the first acceleration data point which exceeds RMSa and the last acceleration data point after which no acceleration data point exceeds RMSa. RMS acceleration value of this ground excitation is calculated which is $0.00496 m/sec^2$. Based on this rms value, strong motion duration is calculated which is 127.566 sec. Strong motion duration for discrete acceleration data is calculated through MATLAB.

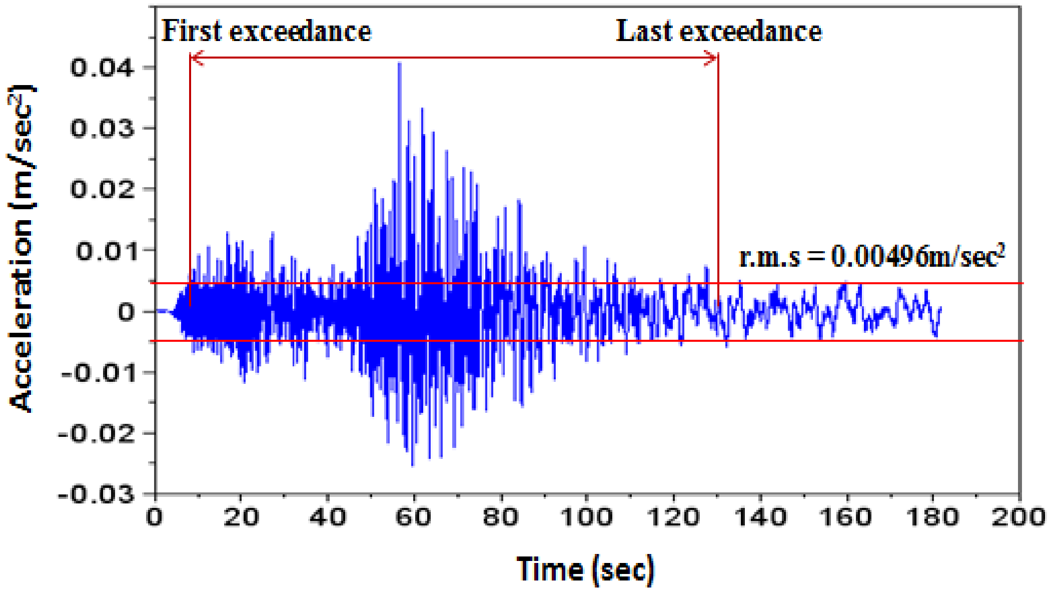


Figure 4.2: Port blair time history

In the present study, there are total 184 ground motion records from past 23 earthquake events of India, out of which 67 are classified as strong ground motions based on the peak ground acceleration values and strong motion duration which is calculated by selecting r.m.s. acceleration, a threshold value, as explained above. Table 4.4 shows all the 67 classified strong ground motions from 23 earthquake events for different regions of our country.

Table 4.4: Available Indian Strong Ground Motion Earthquake Records

Region	**Recording Station**	**PGA** (m/sec^2)	**Recorded Time (s)**	**R.M.S. value** (m/sec^2)	**Strong Motion Duration (s)**
Dharmsala	Bhawarna	0.365	11.98	0.06042	11.26
	Jawali	0.149	17.96	0.03455	17.90
	Shahpur	2.00	20.10	0.19083	4.22
North-East India	Nongkhlaw	0.539	29.64	0.0878	20.98
	Pynursla	0.91	18.58	0.11357	12.96
	Saitsama	1.11	20.66	0.12153	9.84
	Ummulong	1.11	16.94	0.12409	10.54
India-Burma border 1987	Bamungao	0.194	29.48	0.04464	29.16
	Berlongfer	0.706	42.76	0.12665	35.70
	Diphu	0.843	39.10	0.13374	36.16
	Hatikhali	0.305	36.22	0.06453	35.56
	Saitsama	0.364	27.52	0.084	25.36
India-Bangladesh border 1988	Mawphlang	0.796	28.16	0.166	24.76
	Nongkhlaw	1.05	45.28	0.1084	35.22
	Pynursla	0.487	34.60	0.0689	29.86
	Ummulong	0.553	24.52	0.08593	23.84
	Umsning	0.39	23.86	0.0735	23.82
India-Burma border 1988	Baithalongso	1.51	78.08	0.23523	66.36
	Berlongfer	2.95	119.70	0.2949	44.86
	Hajadisa	0.902	64.20	0.15697	51.10
	Khliehriat	0.688	61.50	0.11547	57.06
	Panimur	1.65	72.06	0.2455	62.36
	Saitsama	2.07	81.10	0.28524	58.10
	Ummulong	0.886	66.14	0.1717	53.56
	Umrongso	0.748	64.74	0.14623	55.26
	Umsning	1.20	70.60	0.18582	56.72
India-Burma border 1990	Baithalongso	0.603	22.00	0.13487	21.54
	Berlongfer	1.42	62.84	0.15972	21.82
	Diphu	0.898	32.24	0.16487	20.12
	Saitsama	0.61	26.52	0.12	22.96
Uttarkashi	Bhatwari	2.48	36.16	0.35314	11.04
	Rudraprayag	0.523	39.70	0.13157	32.22
	Srinagar	0.654	41.10	0.11265	37.24
	Uttarkashi	2.37	39.92	0.34458	10.72
Chamba	Chamba	1.43	18.24	0.1635	5.40
	Rakh	0.29	9.18	0.0541	5.90
India-Burma border 1995	Berlongfer	0.707	81.72	0.08521	60.46
	Diphu	0.790	28.58	0.16141	21.60
	Hatikhali	0.437	18.84	0.09236	17.04

Xizang-India border	Ukhimath	0.371	15.20	0.06097	4.76
India-Bangladesh border 1997	Katakhal	1.05	26.58	0.20927	19.22
	Nongpoh	0.476	47.38	0.05278	40.30
	Nongstoin	0.469	39.02	0.07386	31.12
	Pynursla	0.279	28.62	0.05826	25.20
Chamoli 1999	Ghansiali	0.714	26.32	0.1619	26.22
	Gopeshwar	1.95	25.42	0.267	15.78
	Roorkee	0.554	43.525	0.07948	37.95
	Ukhimath	0.891	24.78	0.14311	21.20
Kachchh	Ahmedabad	1.04	133.525	0.11335	54.765
Chamoli 2005	Chamoli	0.411	44.61	0.05032	17.173
	Roorkee	0.023	66.215	0.00347	54.666
Uttarkashi 2007	Nathpa	0.049	40.70	0.00907	34.955
	Roorkee	0.019	63.58	0.0036	56.095
Andaman Islands 2008	Port blair	0.225	63.41	0.0384	35.972
Nagaland	Tinsukia	0.023	66.10	0.00314	45.066
Uttarakhand	Champawat	0.017	70.10	0.00164	42.127
	Munsiari	0.095	70.085	0.0083	20.628
	Pithoragarh	0.034	76.56	0.00354	41.757
Andaman Islands 2010	Port blair	0.041	181.435	0.00496	127.566
India-Myanmar (Manipur) border	Guwahati	0.184	164.795	0.01416	85.557
	Jorhat	0.039	95.10	0.00826	94.285
	Naogaon	0.321	135.445	0.026	75.712
Assam	Golaghat	0.09	66.69	0.0179	42.262
	Khokhrajhar	0.059	110.995	0.00722	77.99
Phek (Nagaland)	Golaghat	0.147	76.555	0.0146	58.0812
Kohima (Nagaland)	Golaghat	0.162	79.475	0.0139	66.826
	Jorhat	0.09	128.87	0.01	94.62

Chapter 5

Response Spectrum generation for Indian subcontinent

5.1 General

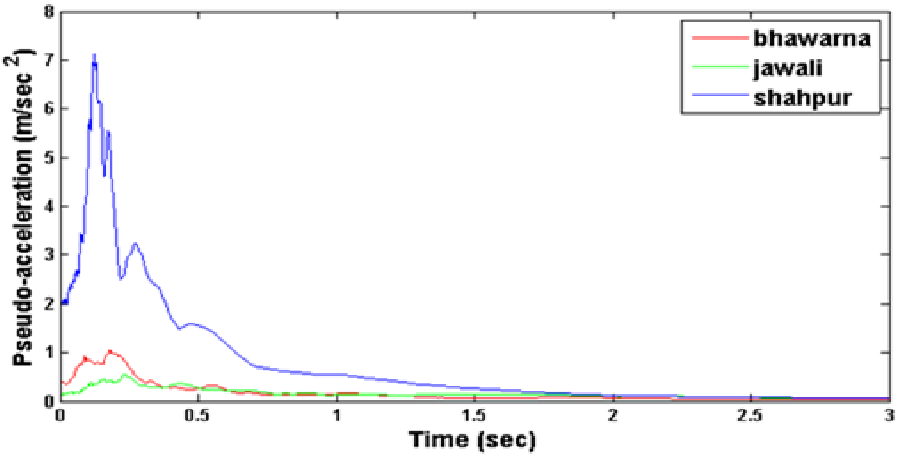
Strong motion records are three - component (two horizontal components and a vertical component) time histories recorded by accelerometers in analogue or digital form. These records are used to conduct time history dynamic analyses and derive response spectra.

Response spectrum provides the maximum response of a structure to a particular earthquake ground motion at different frequencies or periods. The chapter deals with the development of Response spectrum for classified 67 strong ground motion time histories that has been selected from 184 available recorded Indian earthquake time histories (23 events) based on a detailed statistical study performed on specified ground motion parameters. Statistical analysis is performed by normalizing the time histories to unit value of peak ground acceleration parameter and Mean, Mean plus one standard deviation and Maximum response spectrum are generated for all recorded events. Comparison of developed Response spectrum is done for four proposed regions of Indian subcontinent.

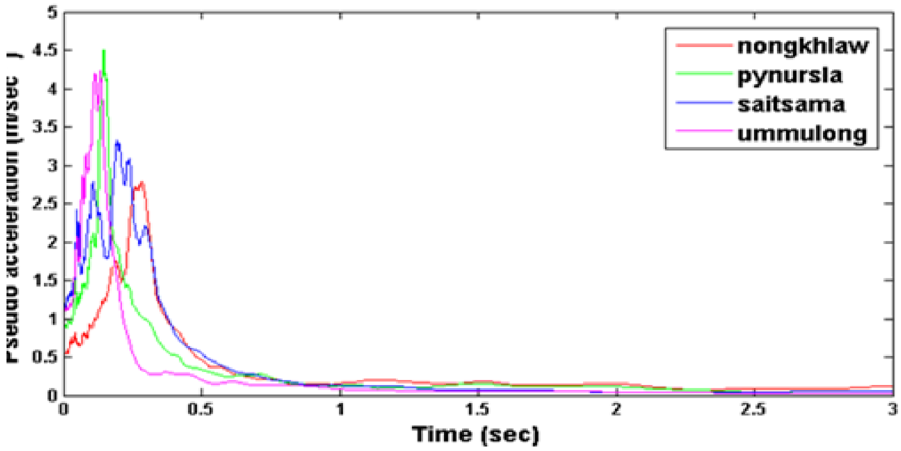
5.2 Generation of Response Spectrum

Response spectrum for a specified earthquake records are used to obtain the response of a structure to an earthquake ground motion with similar characteristics. As the main objective of study is to compare representative pseudo-acceleration response spectrum with code based design acceleration spectrum so only pseudo-acceleration response spectrum are presented for the past Indian earthquake records as listed in Table4.4 of Section 4.4. Response Spectrum is generated using the steps presented in Section 3.3. The study is limited to response spectrum determined for 5 per cent damping although it can readily be extended to include other values of damping. All the plots are developed for 3000 SDOF systems having natural time period limited to 3 second with interval of 0.001 second considering longitudinal component of an earthquake. A code is written in MATLAB for development of Response spectrum.

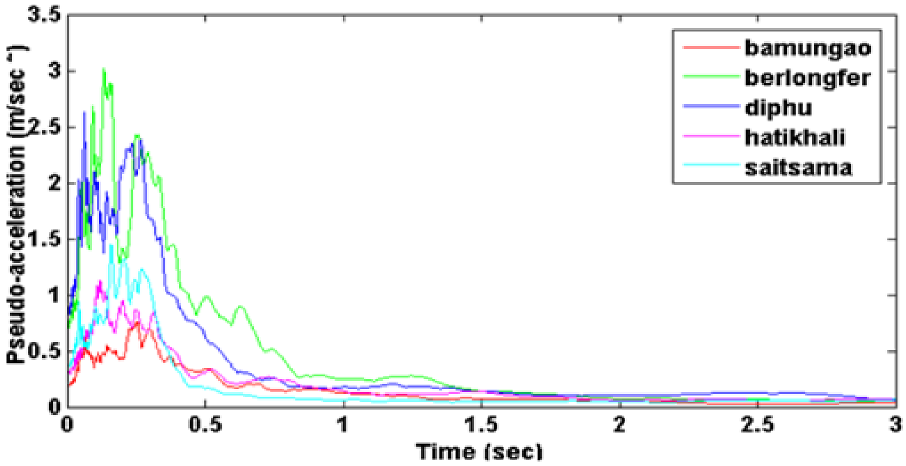
Figure 5.1 shows response spectrum curves for 23 recorded earthquake events of India.



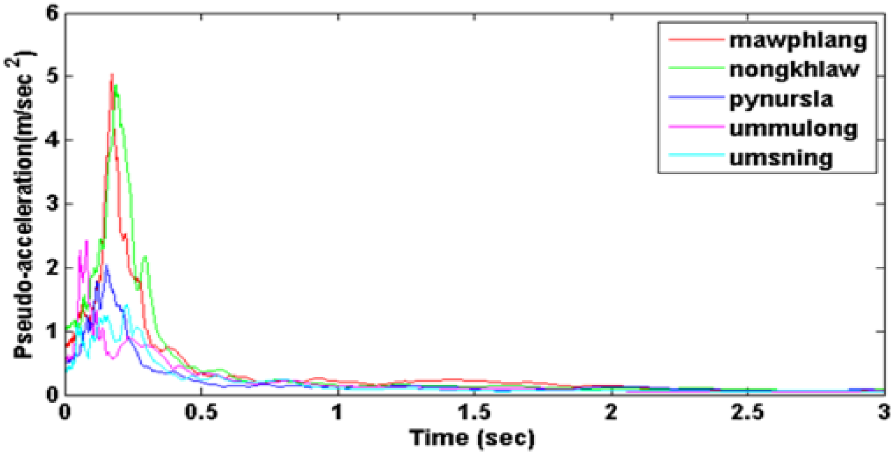
Dharmsala



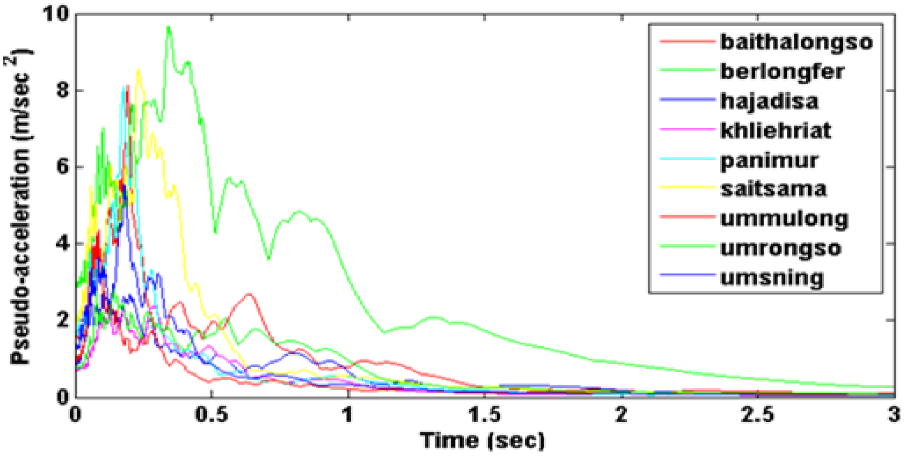
North-East India



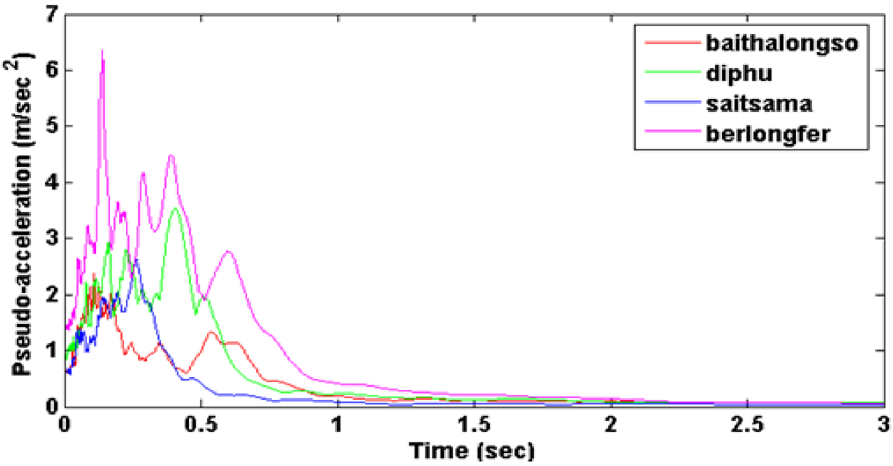
India Burma border 1987



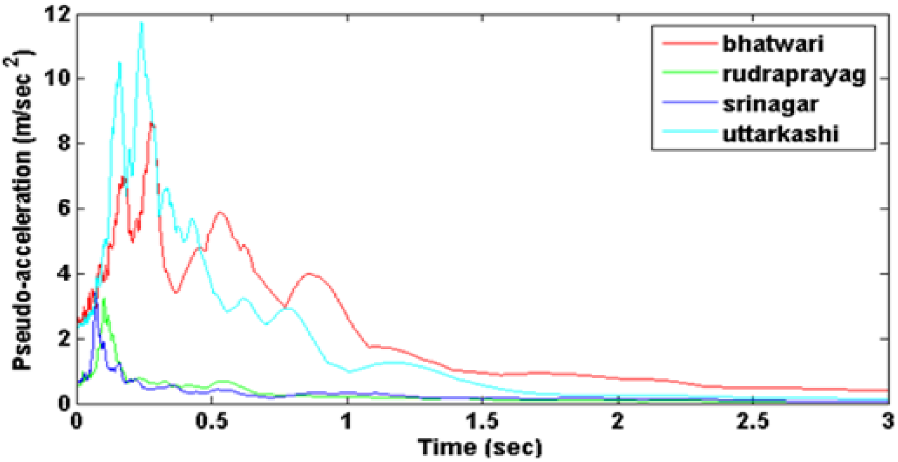
India Bangladesh border 1988



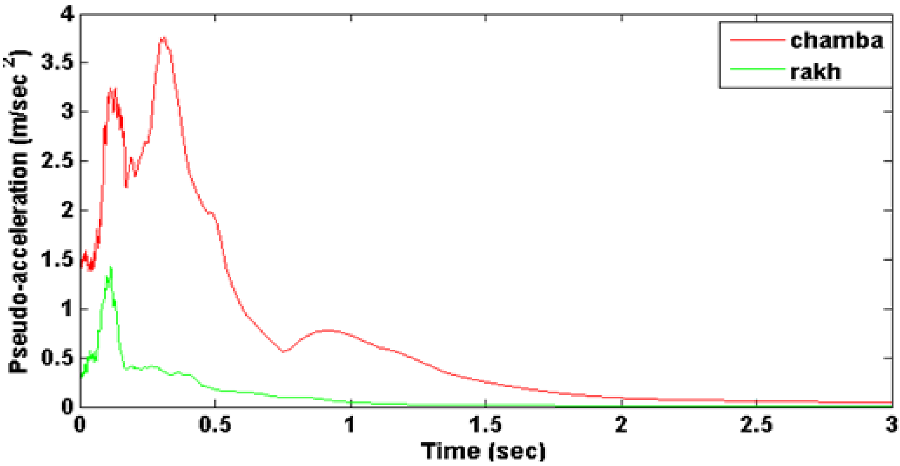
India Burma border 1988



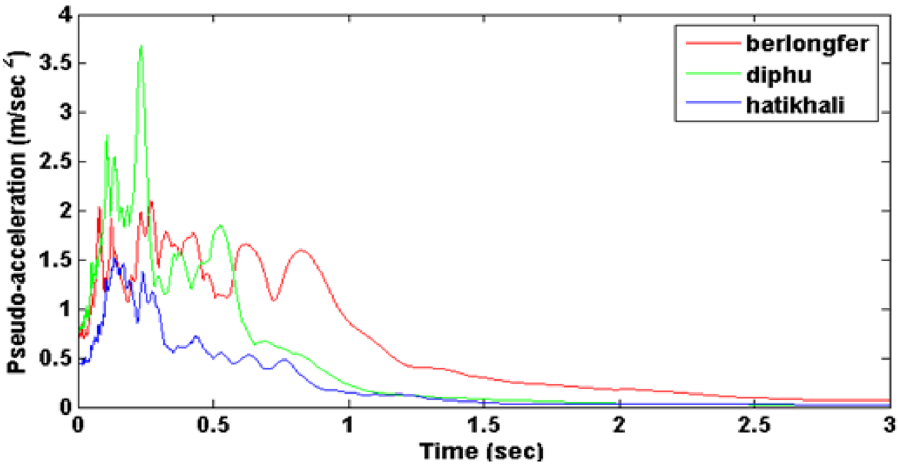
India Burma border 1990



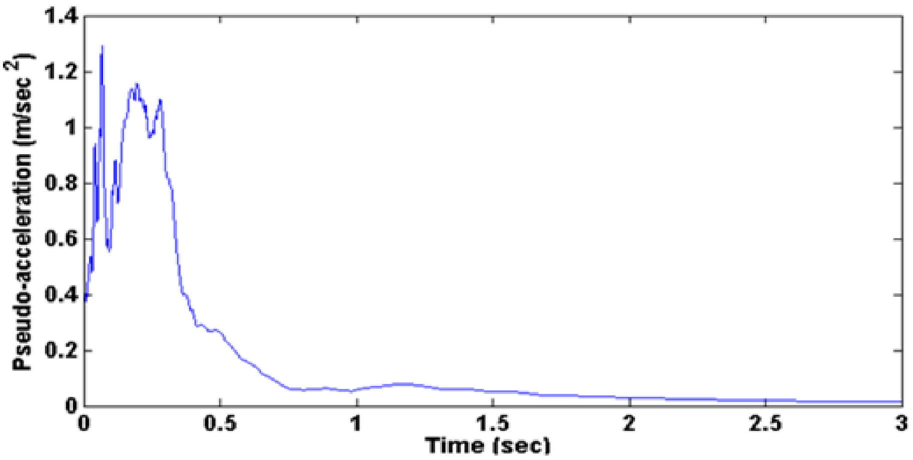
Uttarkashi 1991



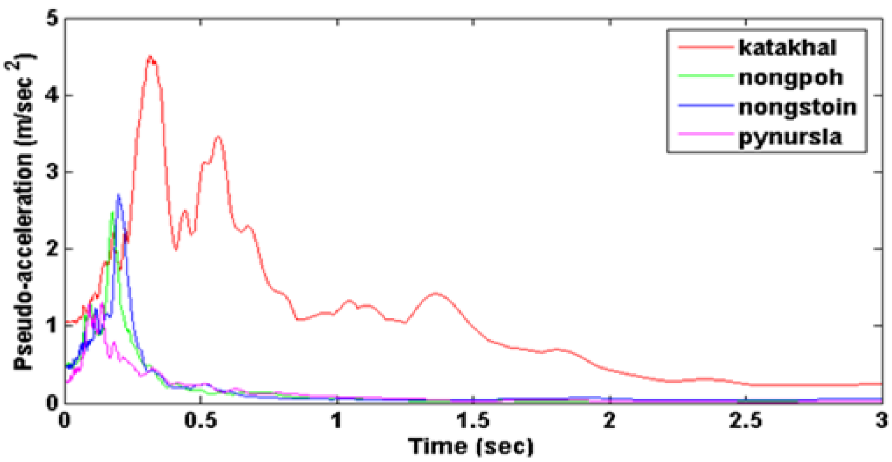
Chamba



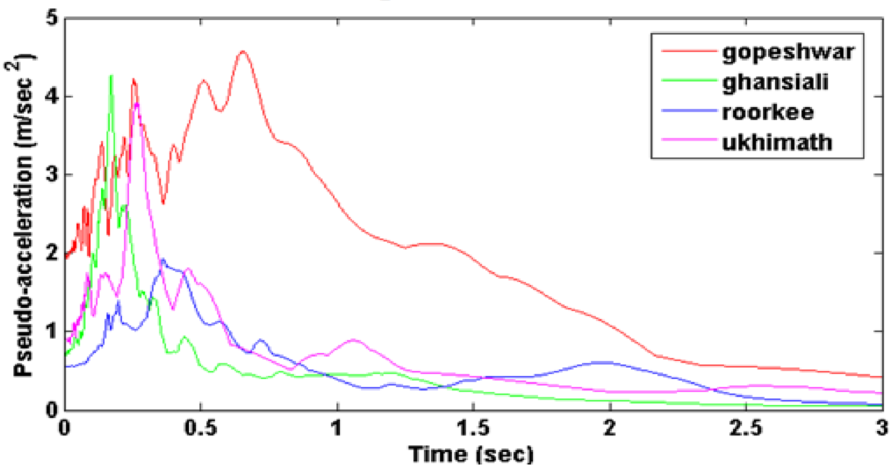
India Burma border 1995



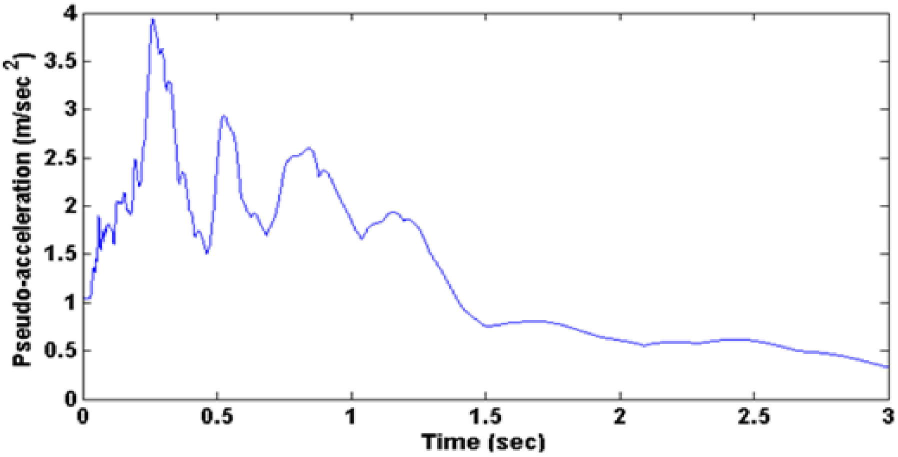
Xizang India border



India Bangladesh border 1997



Chamoli 1999



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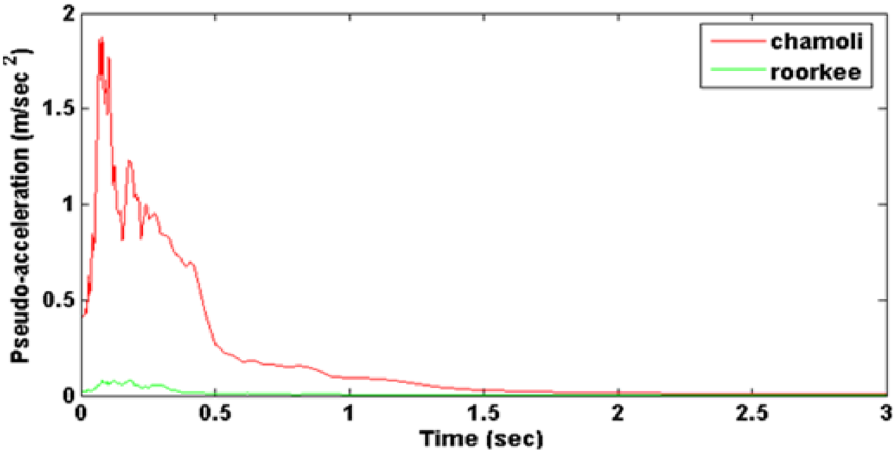
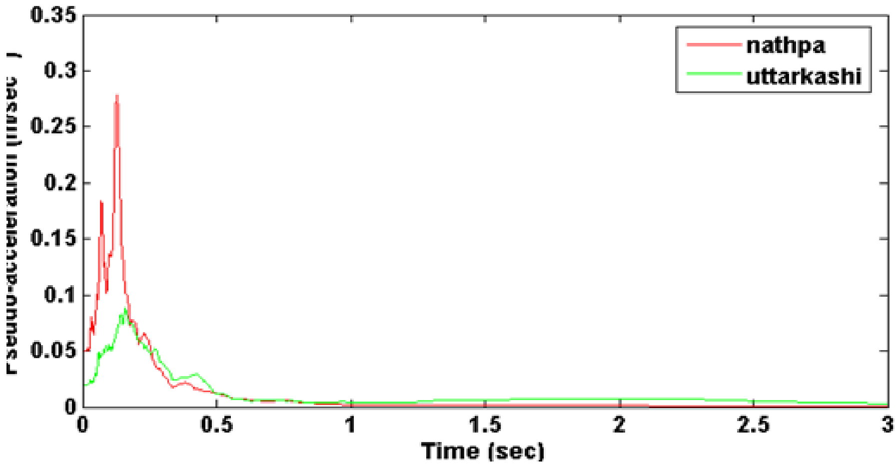
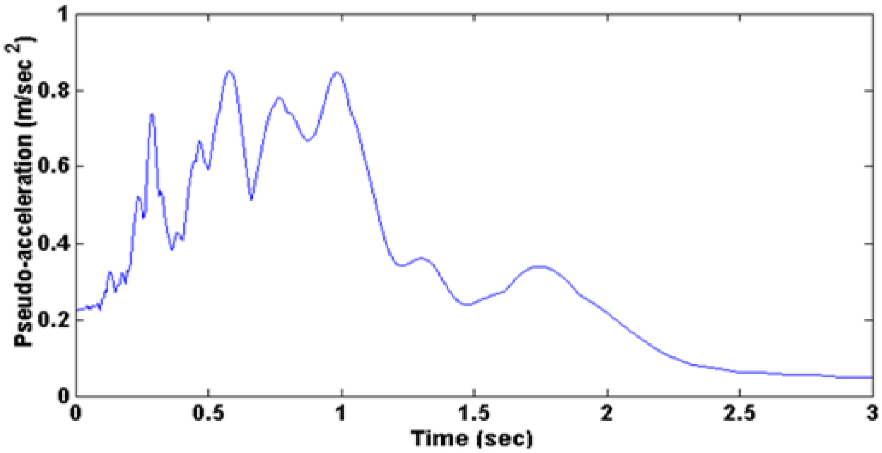


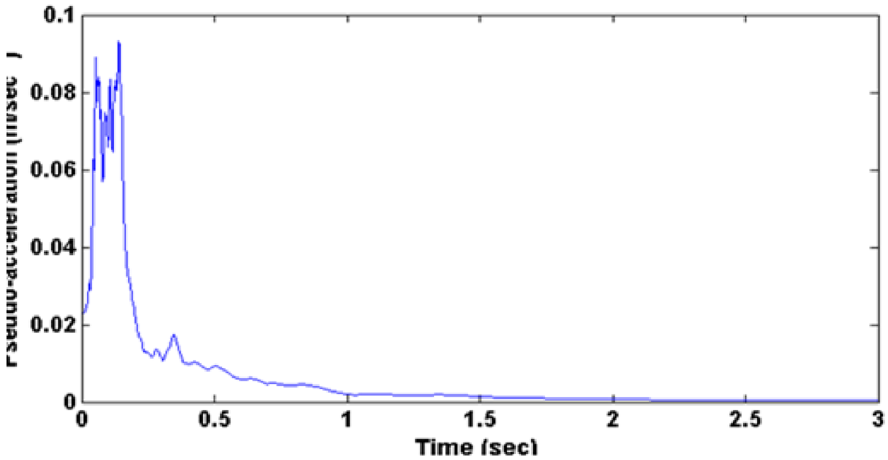
Chamoli 2005



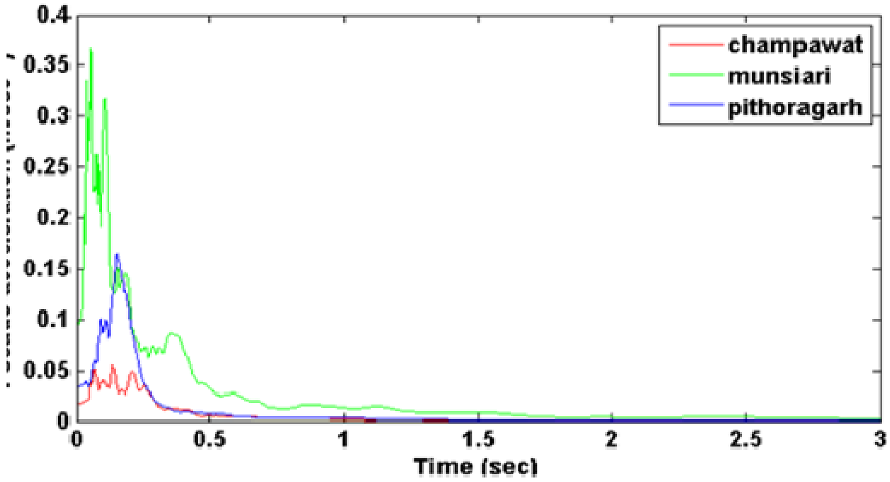
Uttarkashi 2007



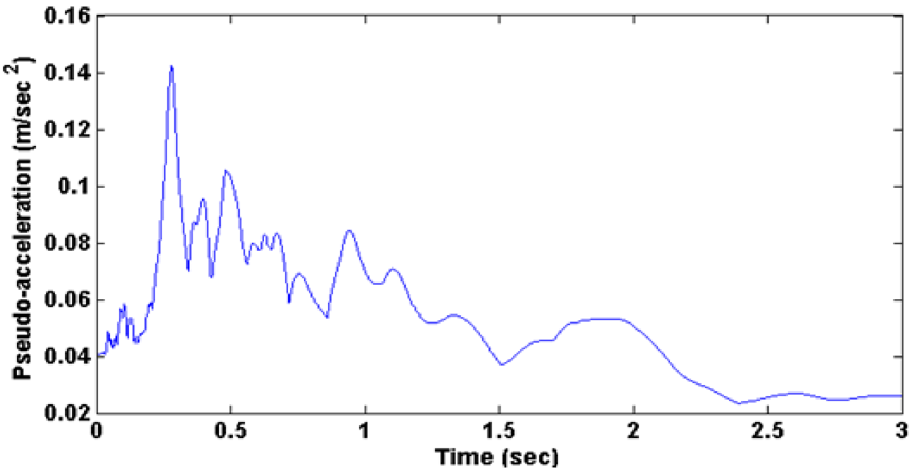
Andaman Island 2008



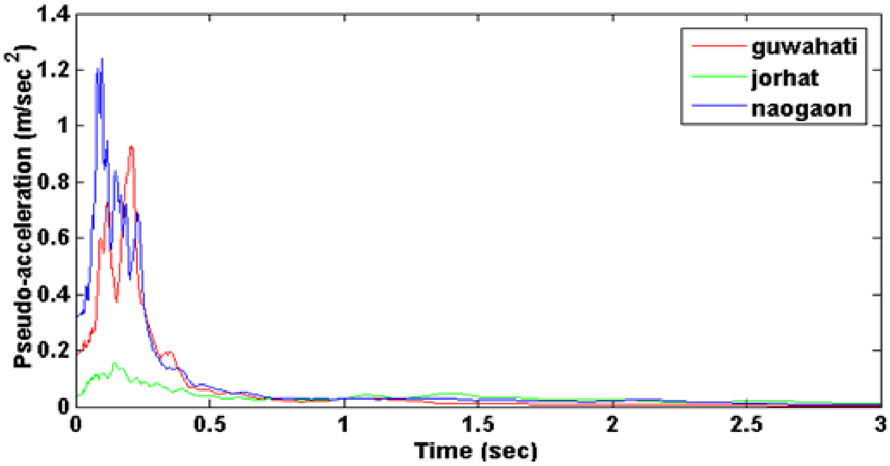
Tinsukia Nagaland



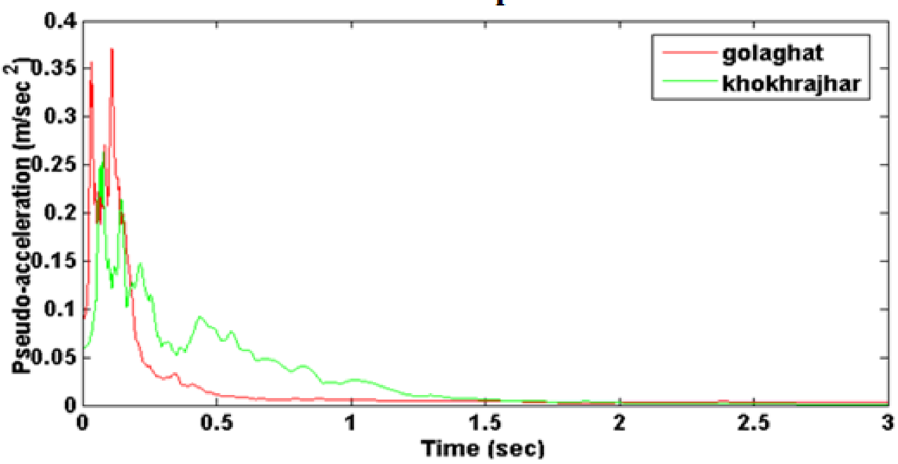
Uttarakhand



Andaman Island 2010



Manipur



Assam

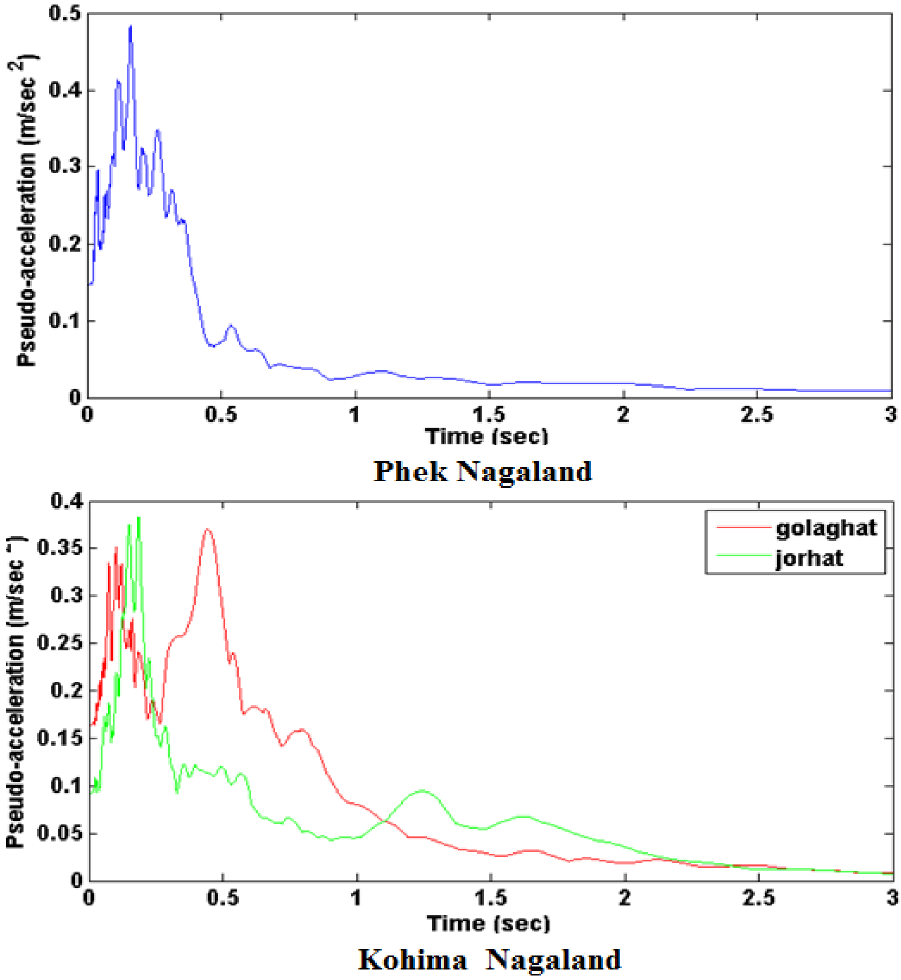


Figure 5.1: Response spectrum for 23 recorded earthquake events of India

The records used in this study included stations located on rock deposits. It is observed that response of all possible SDOF system is different for each recorded ground motions. Therefore it is important to collapse all response spectrum to a single response spectrum. The statistical approach is adopted to collapse number of response spectrum to a single response spectrum which is shown in the subsequent section. It is carried out for all possible SDOF system at each time period for the spectral ordinates (D, V and A).

5.3 Statistical Analysis of Response Spectrum

Response spectrum for a particular earthquake record cannot be used for design directly because the response of a structure to another earthquake record will surely be different even though the recorded ground motion contains some similarities. So for this reason response spectrum from records with common characteristics are averaged and then smoothed before they are used in design. For averaging and smoothing response spectrum, statistical approach is carried out for each ground motion. Statistical analysis is performed for the strong ground motions shown in Table 4.4.

Mean, Mean plus one standard deviation and Maximum response spectrum

Suppose I is the number of ground motions for an earthquake event. The response spectrum for each ground motion is computed. At each period T_n there are as many spectral values as number I of ground motion records. D_i, V_i , and A_i are the deformation, pseudo-velocity and pseudo-acceleration spectral ordinates. Statistical analysis of these data is carried out for the spectral ordinate which provide its mean value, its standard deviation and maximum value at each period T_n. Connecting all the mean values will give mean response spectrum, connecting all mean plus one standard deviation gives mean plus one standard deviation response spectrum and connecting all maximum values will give maximum response spectrum.

Here Dharmsala earthquake event is taken as an example which contains 9 recording stations. As Dharmsala event has 9 stations so I=9 and at each period T_n, there will be 9 spectral values D1, D2, D3, ..D9 . Similarly, spectral ordinates are obtained for V and A.

$$Mean = (D1 + D2 + ...D9)/9. \tag{5.1}$$

By connecting all mean values for each T_n, it will give mean response spectrum.

$$Standard \;\; deviation(\sigma) = \sqrt{\frac{\sum(x - \dot{x})^2}{I - 1}} \tag{5.2}$$

where $\dot{x}$ = mean value of all stations at a particular time period

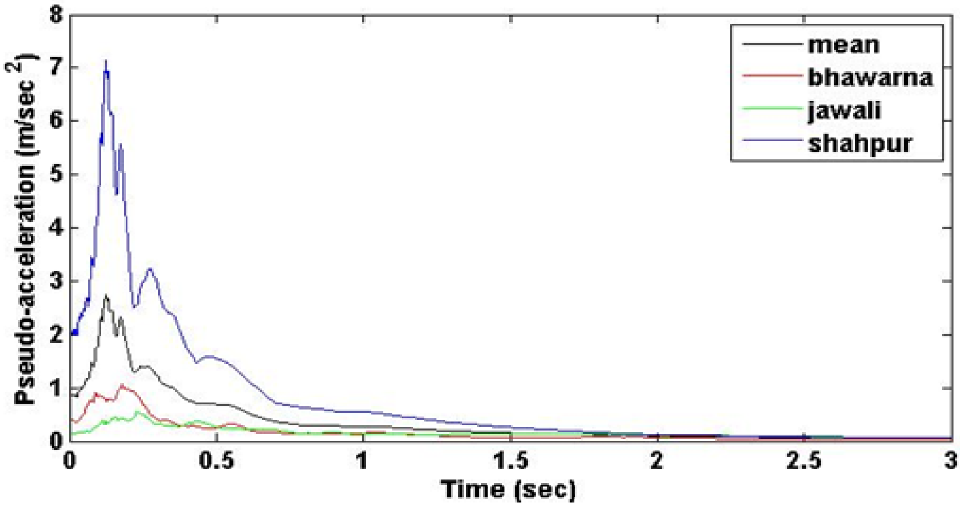
x = spectral ordinate for a station at same time period

By connecting all mean + 1 standard deviation values for each T_n, it will give mean + 1 standard deviation response spectrum.

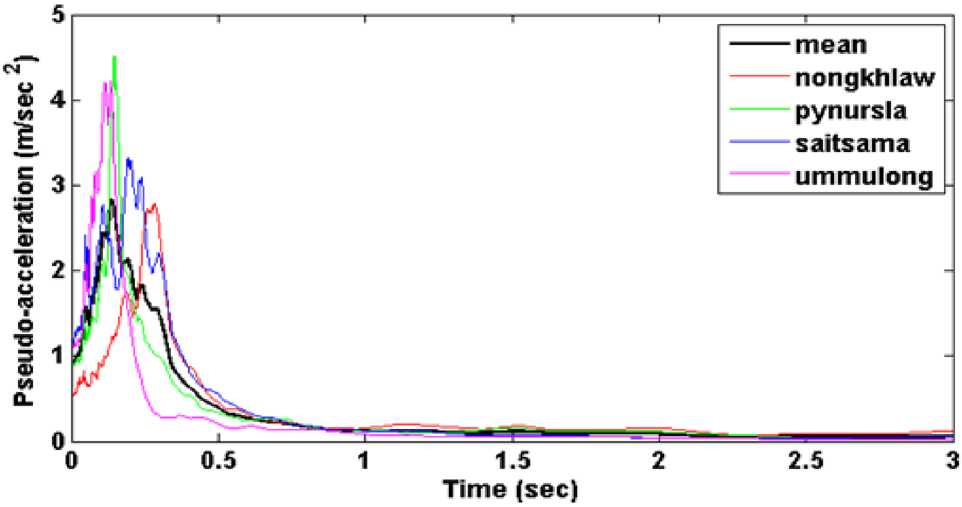
$$Maximum = max(D1, D2, ...D9) \tag{5.3}$$

By connecting all maximum values for each T_n, it will give maximum response spectrum.

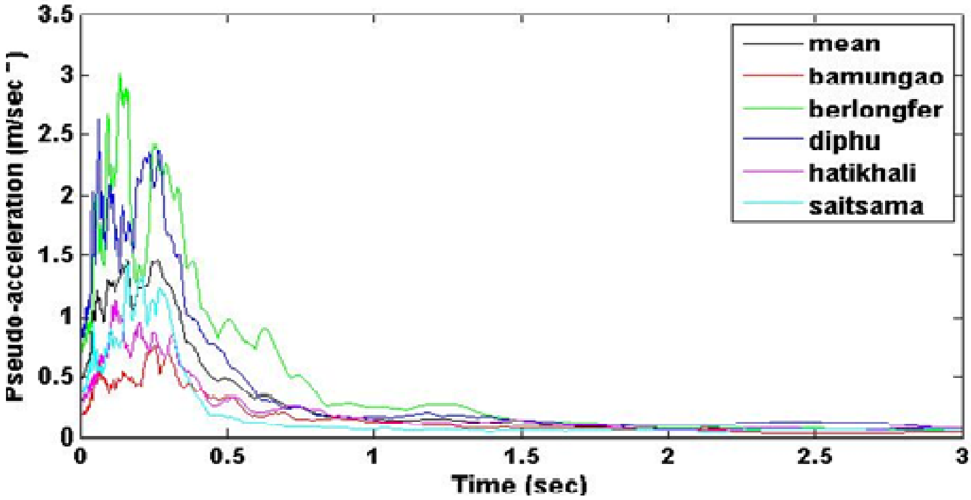
The plots of mean, mean plus one standard deviation and maximum response spectrum (5% damping) obtained for specified earthquake records of India are presented in the Figure 5.2, 5.3 and 5.4 respectively.



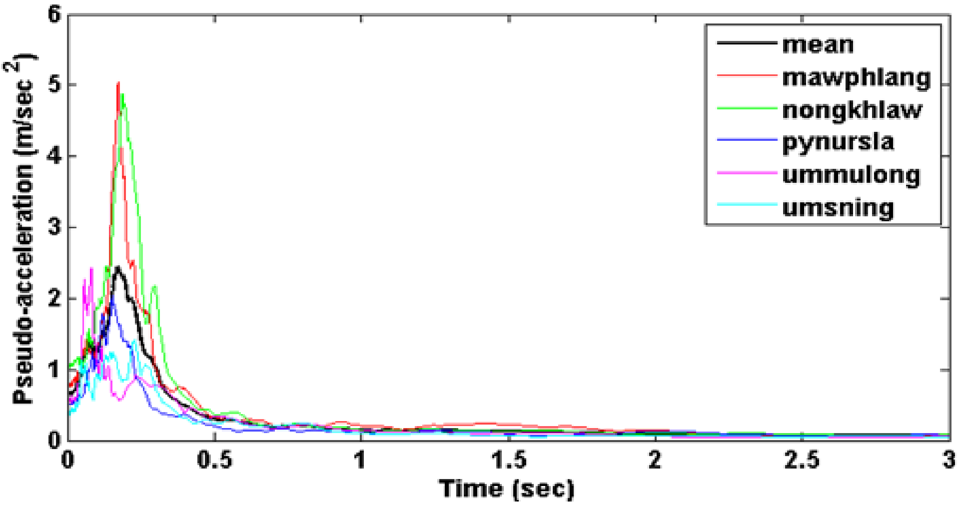
Dharmsala



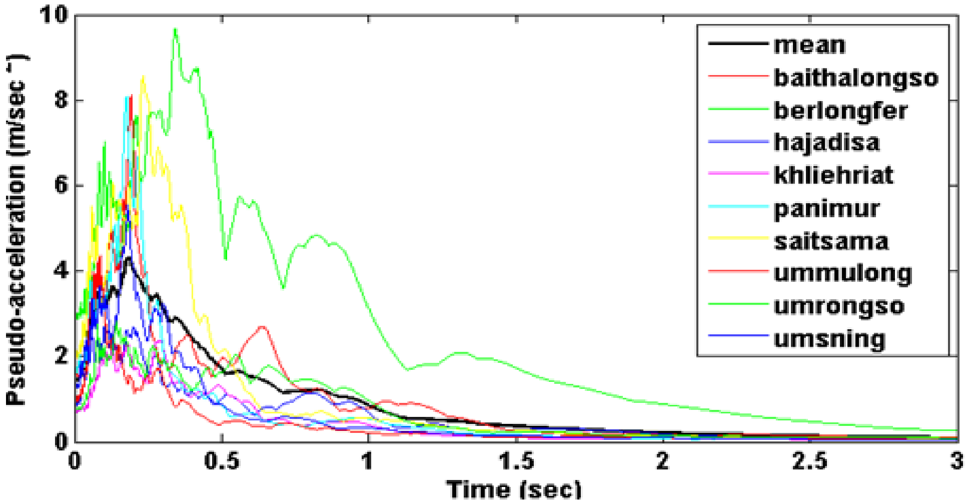
North-East India



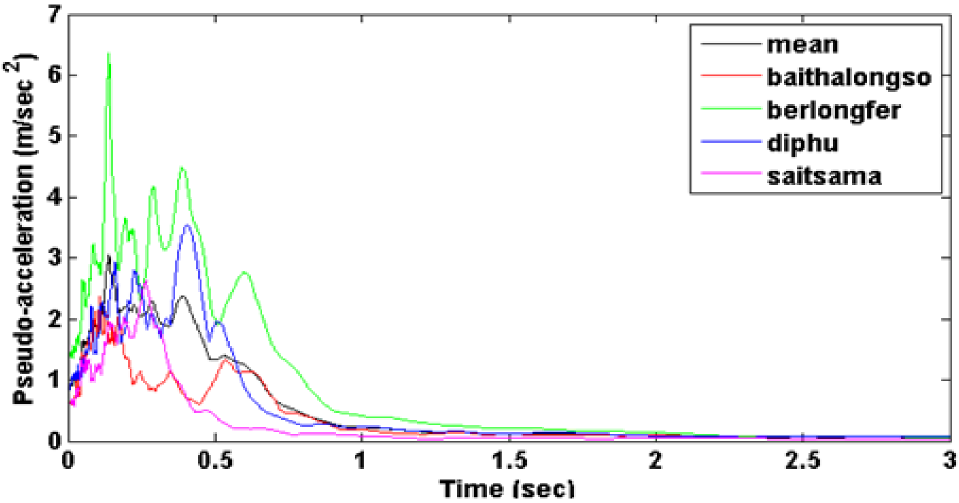
India Burma border 1987



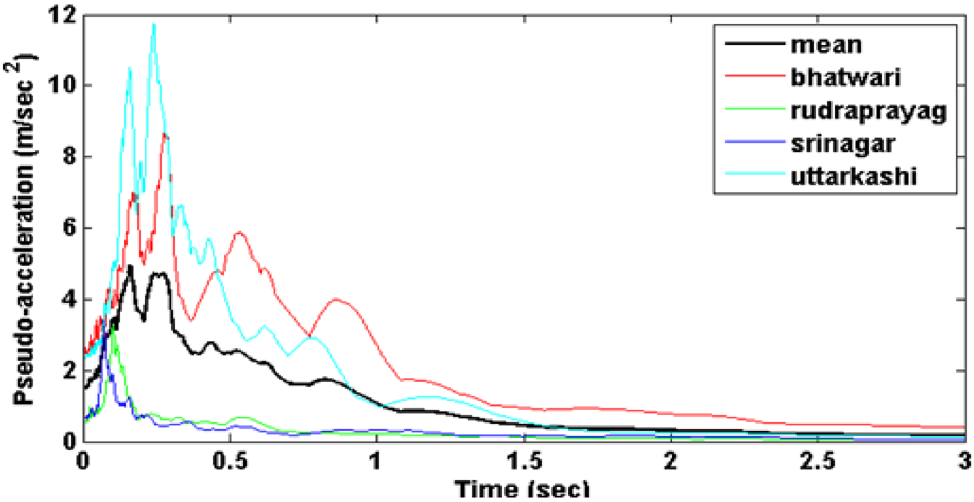
India Bangladesh border 1988



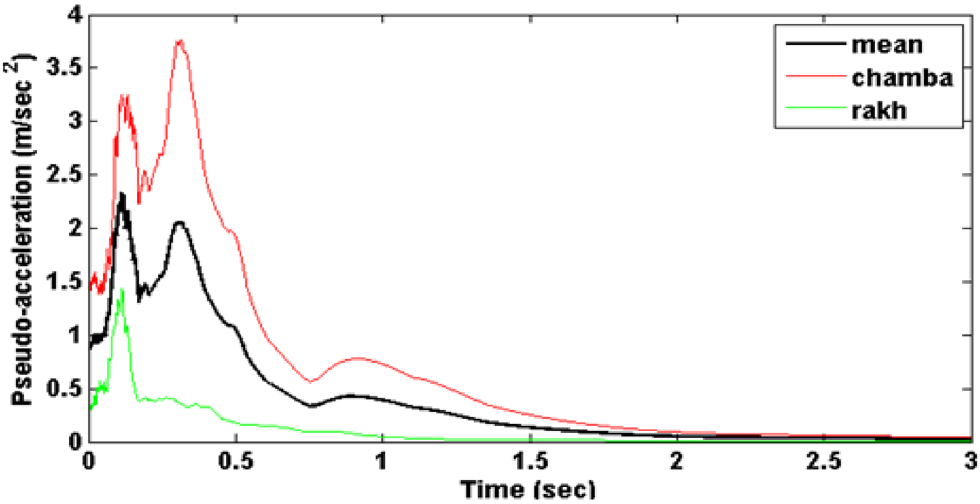
India Burma border 1988



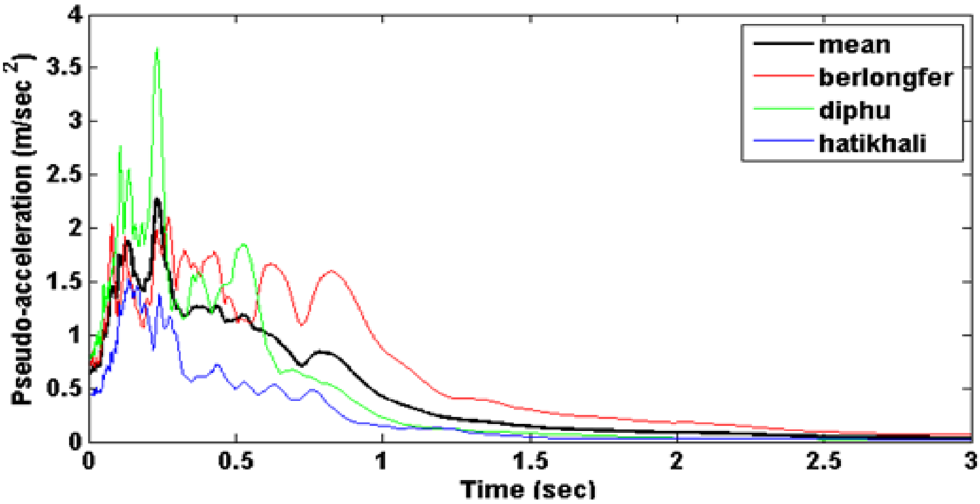
India Burma border 1990



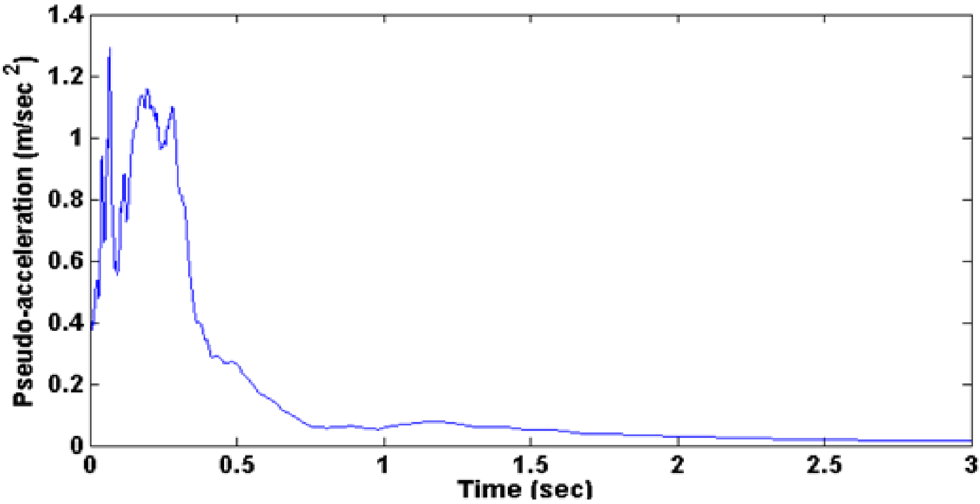
Uttarkashi 1991



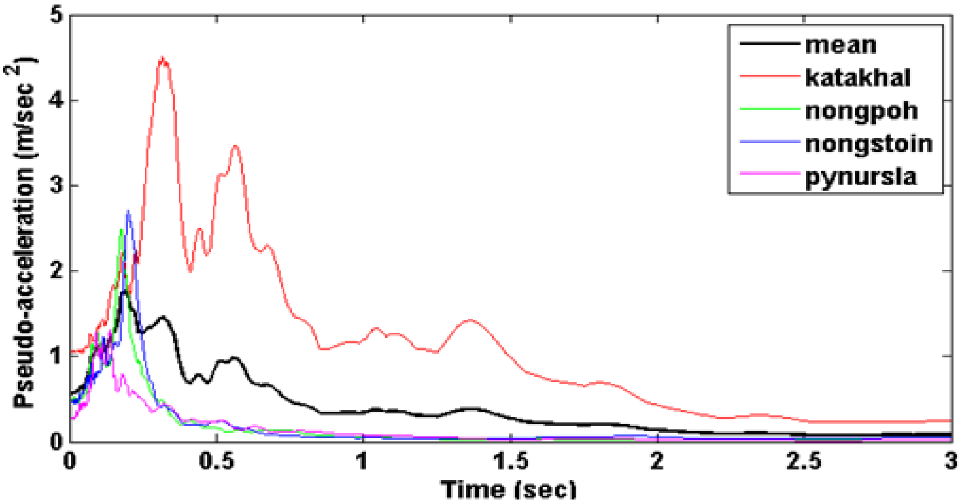
Chamba



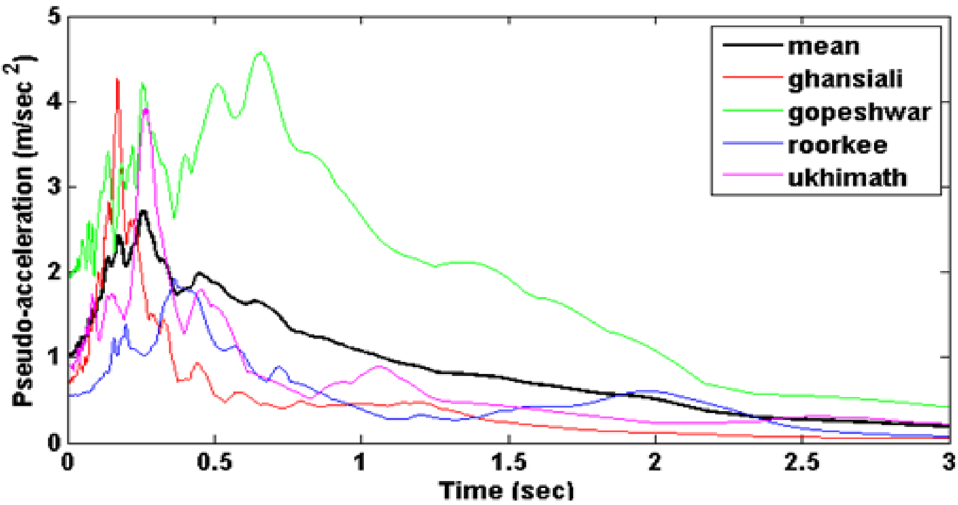
India Burma border 1995



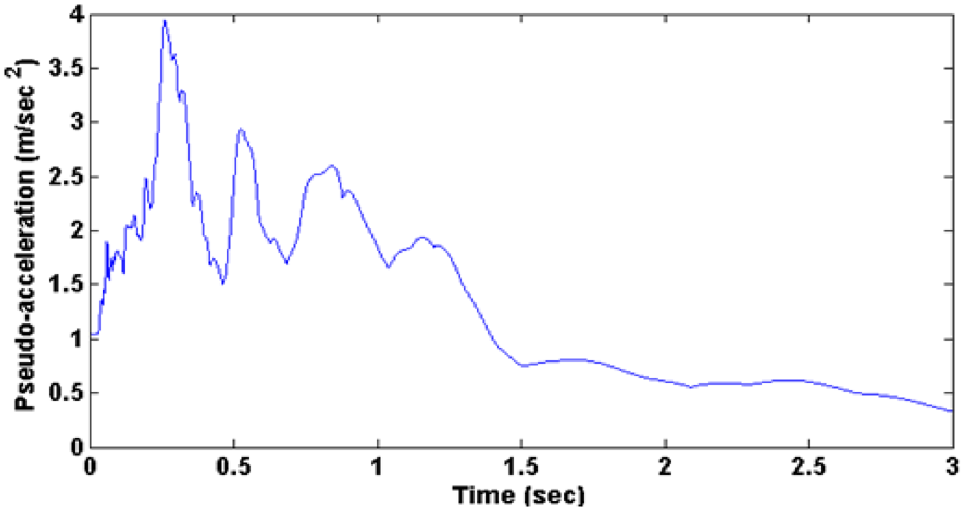
Xizang India border



India Bangladesh border 1997



Chamoli 1999



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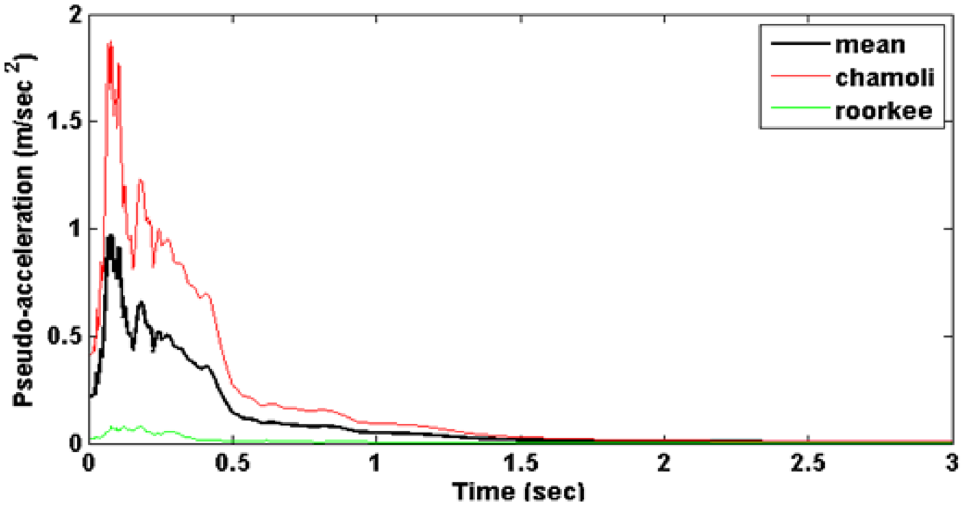
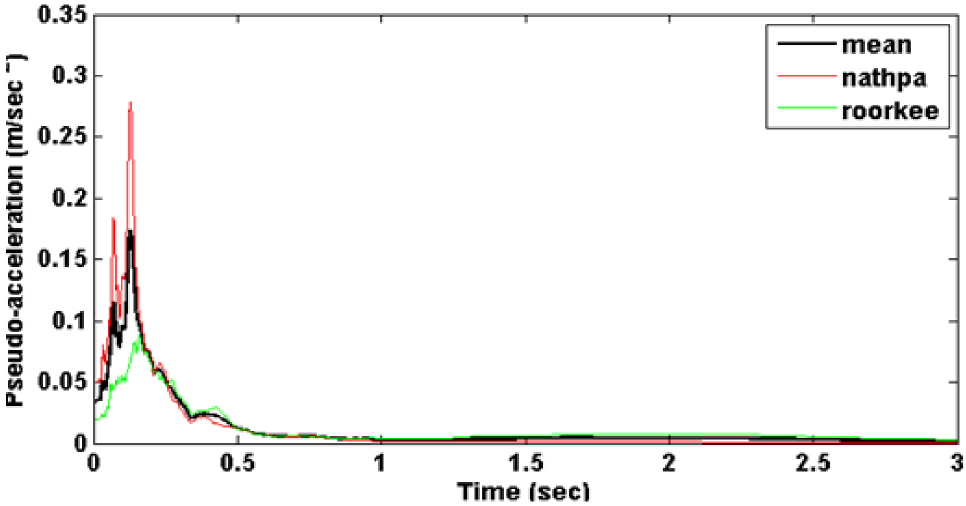
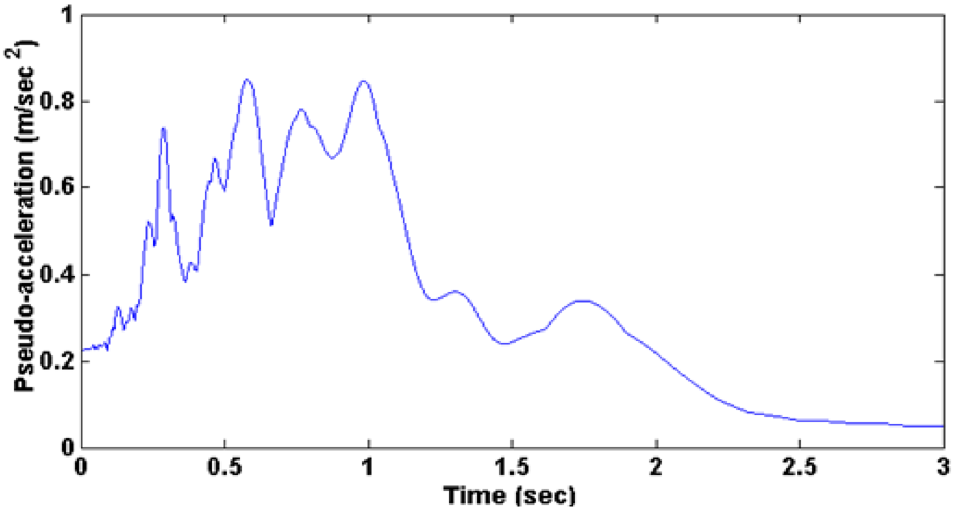


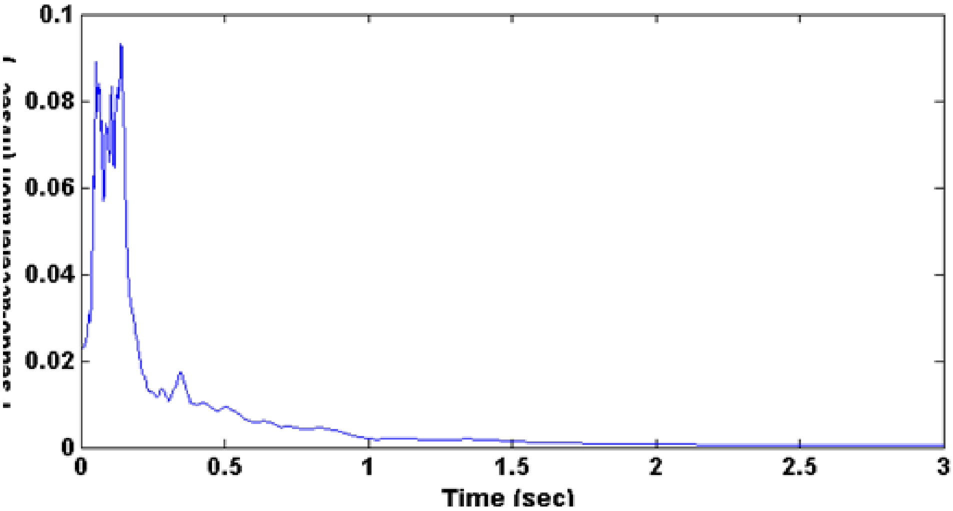
Chamoli 2005



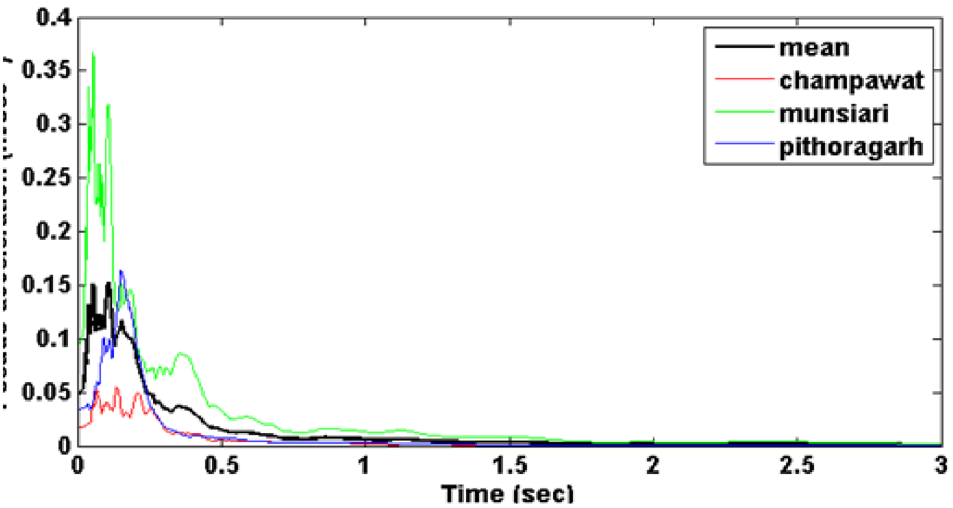
Uttarkashi 2007



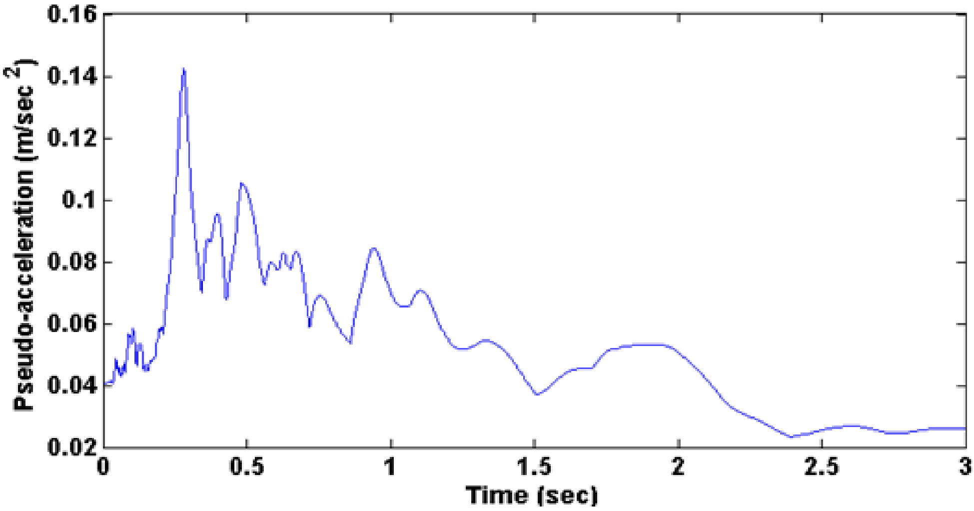
Andaman Island 2008



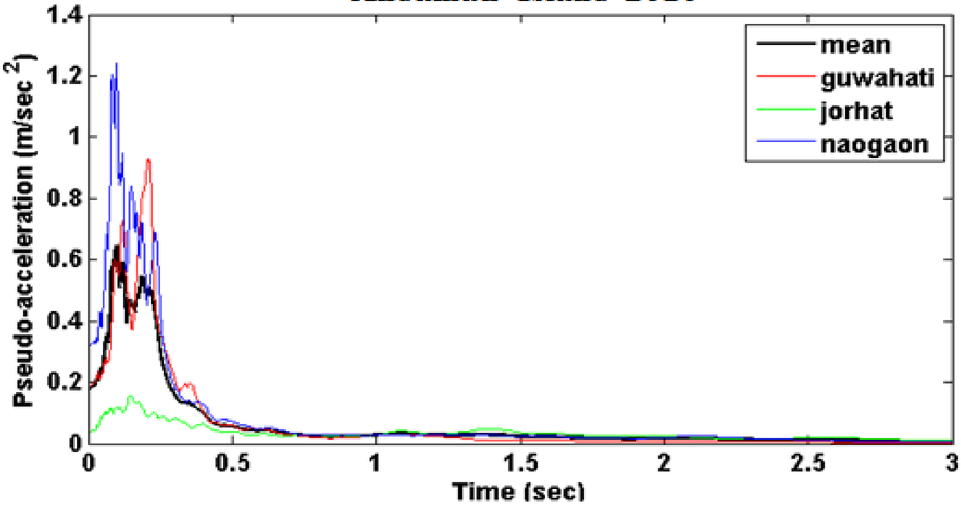
Tinsukia Nagaland



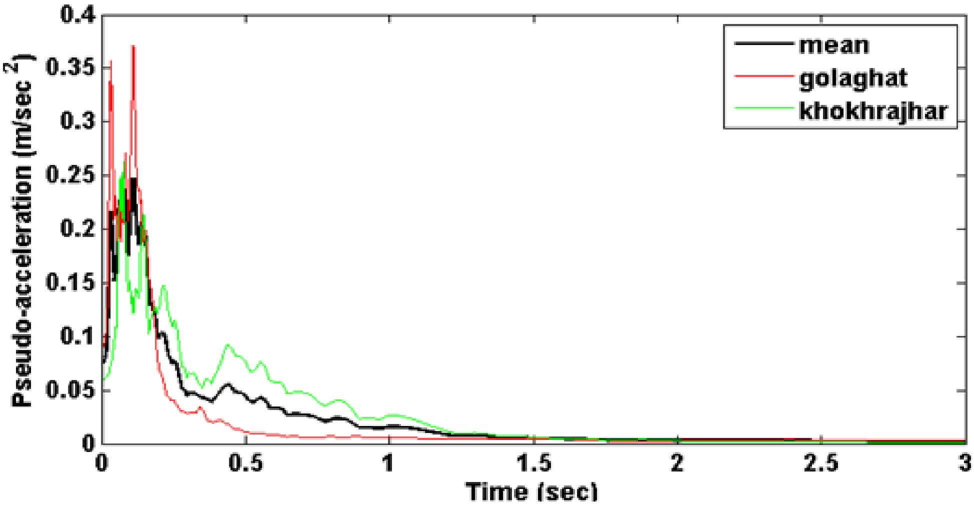
Uttarakhand



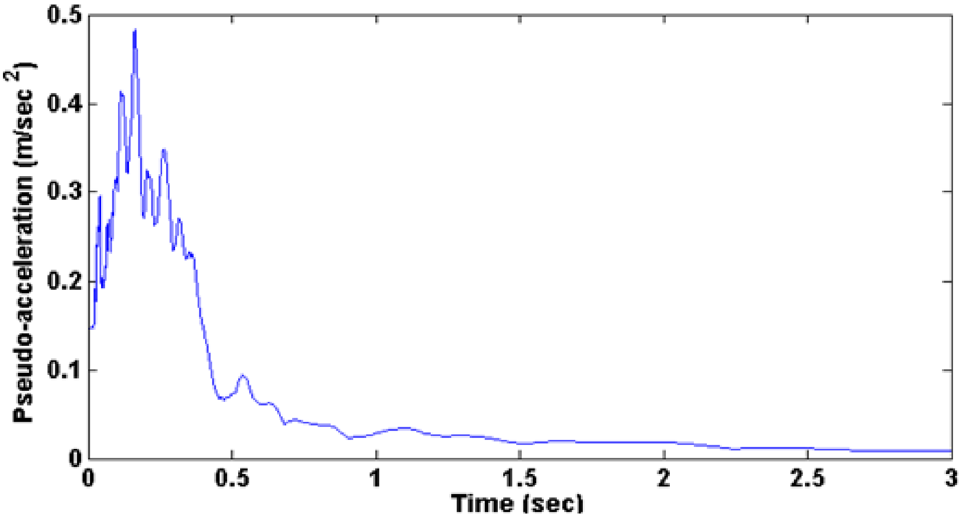
Andaman Island 2010



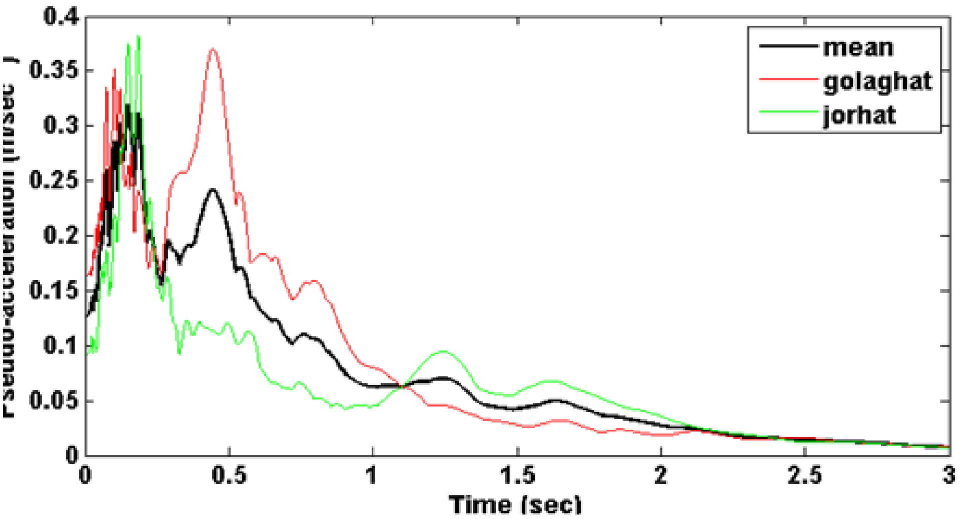
Manipur



Assam

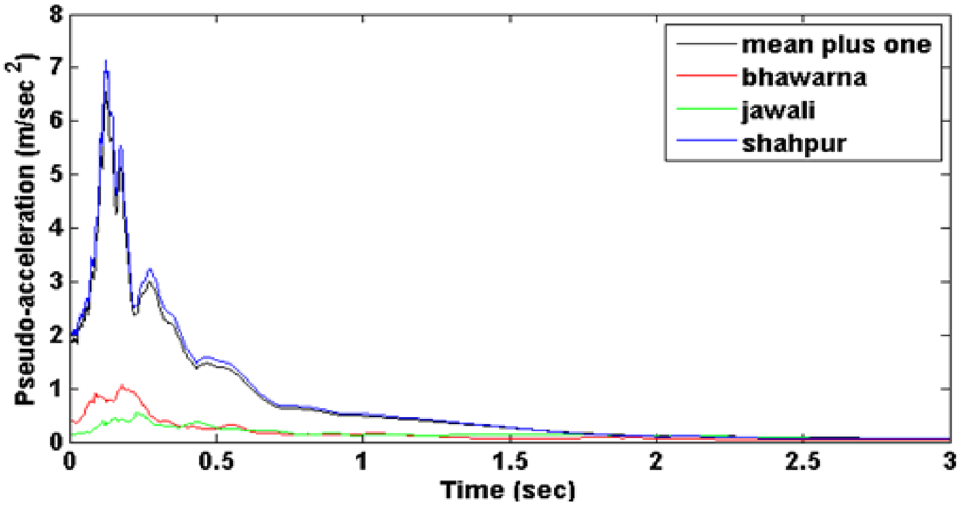


Phek Nagaland

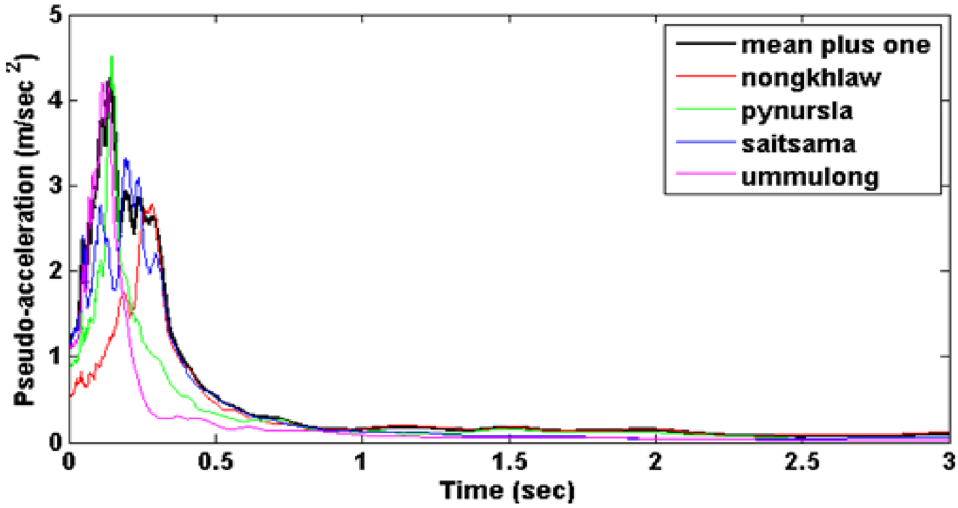


Kohima Nagaland

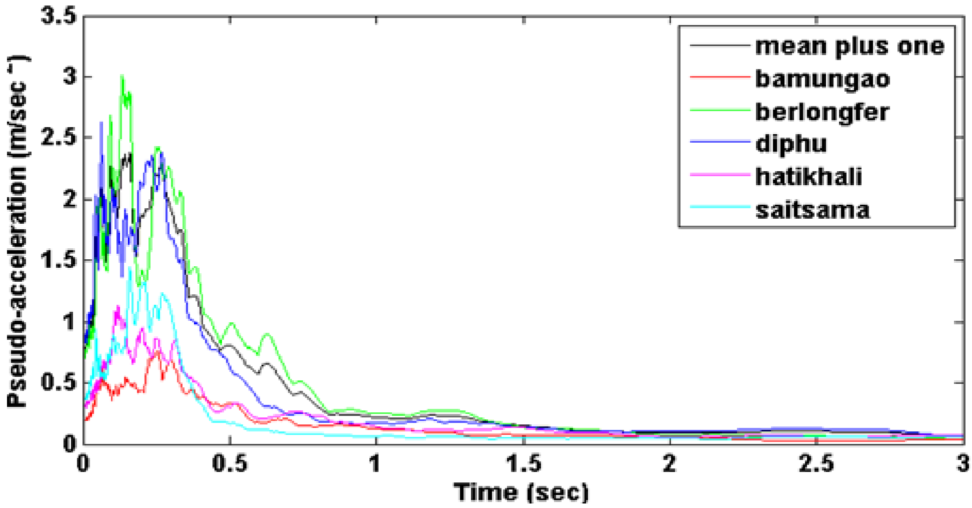
Figure 5.2: Mean response spectrum for 5% damping of horizontal ground motions



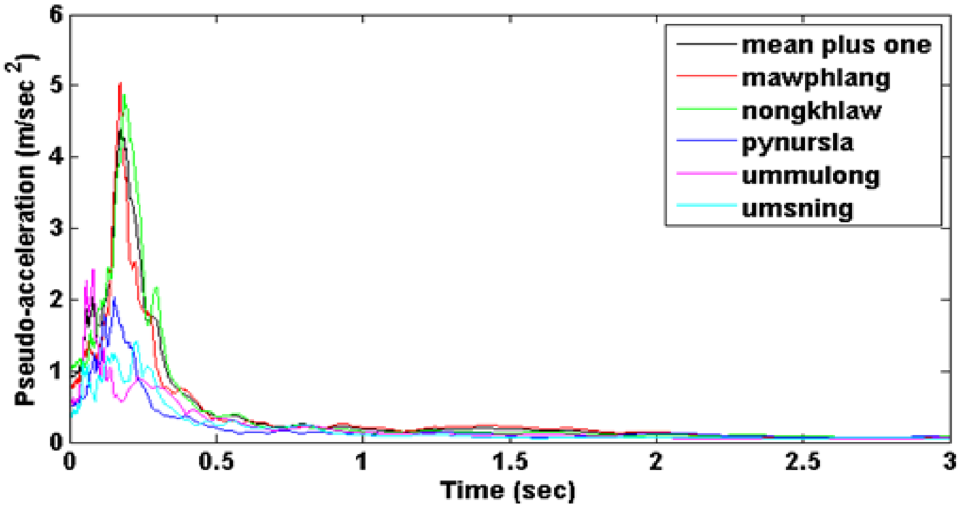
Dharmsala



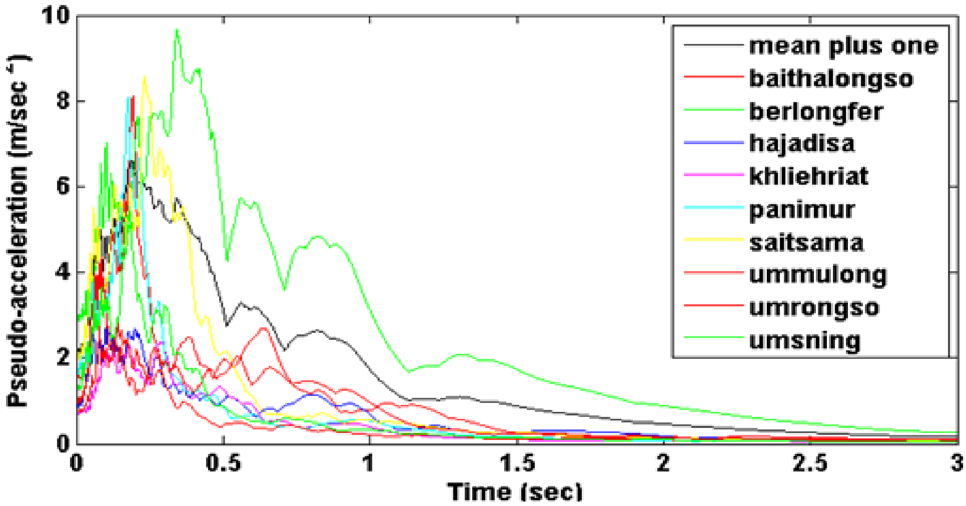
North-East India



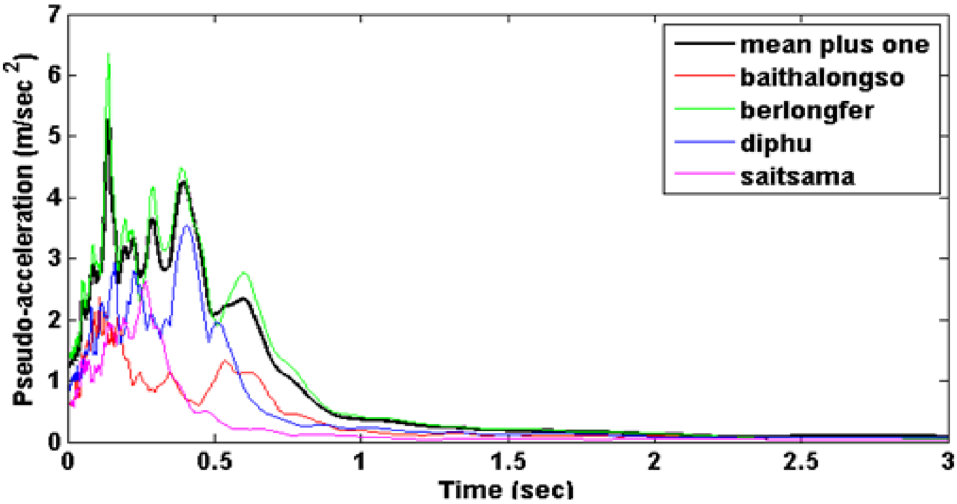
India Burma border 1987



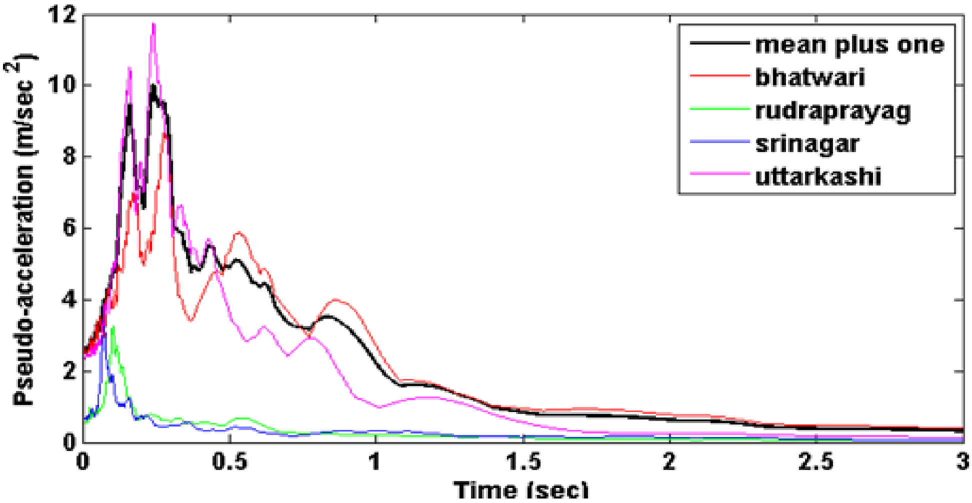
India Bangladesh border 1988



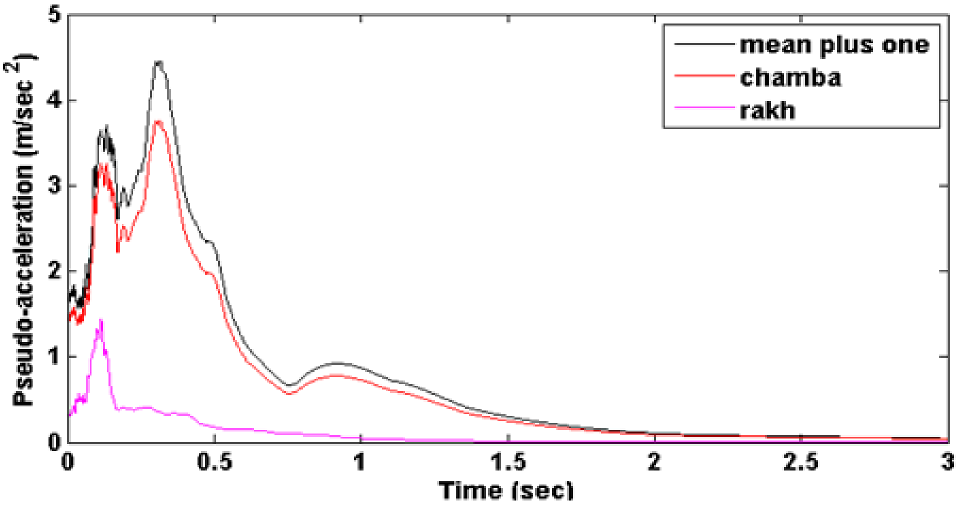
India Burma border 1988



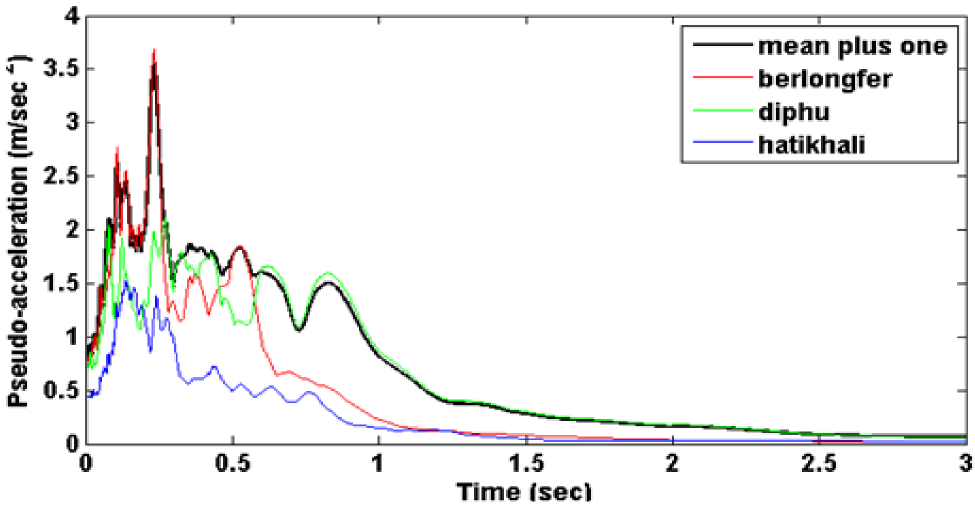
India Burma border 1990



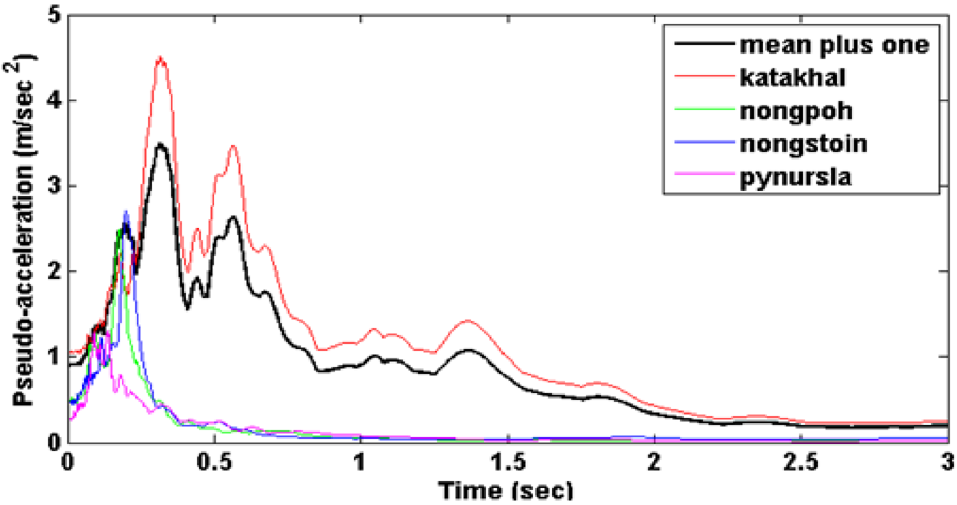
Uttarkashi 1991



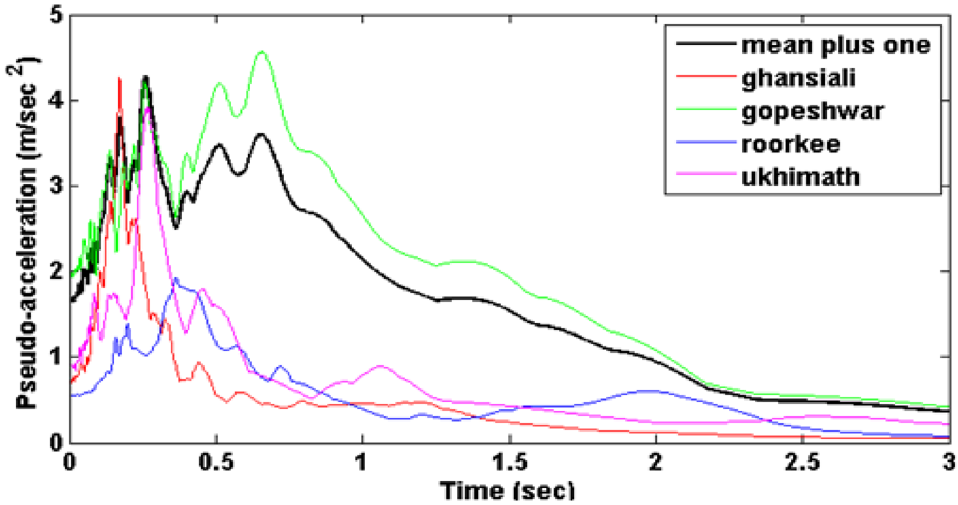
Chamba



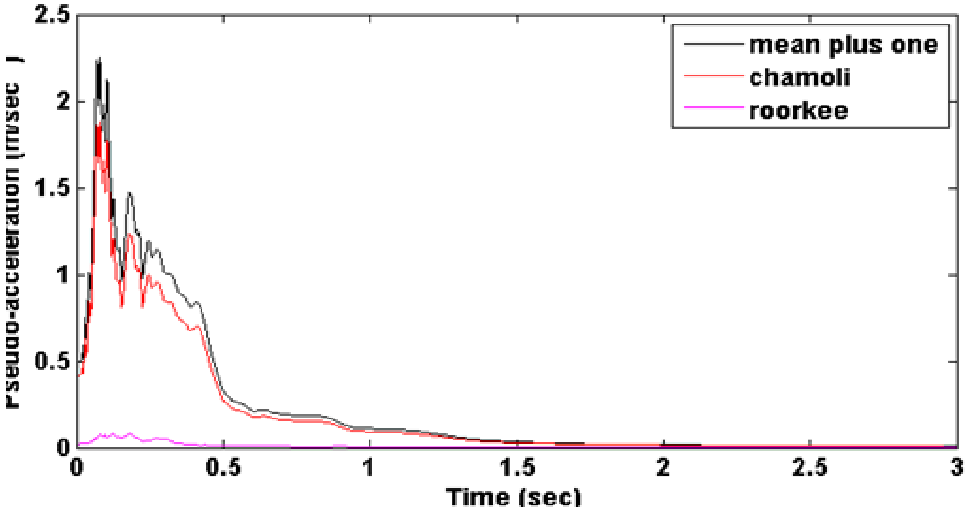
India Burma border 1995



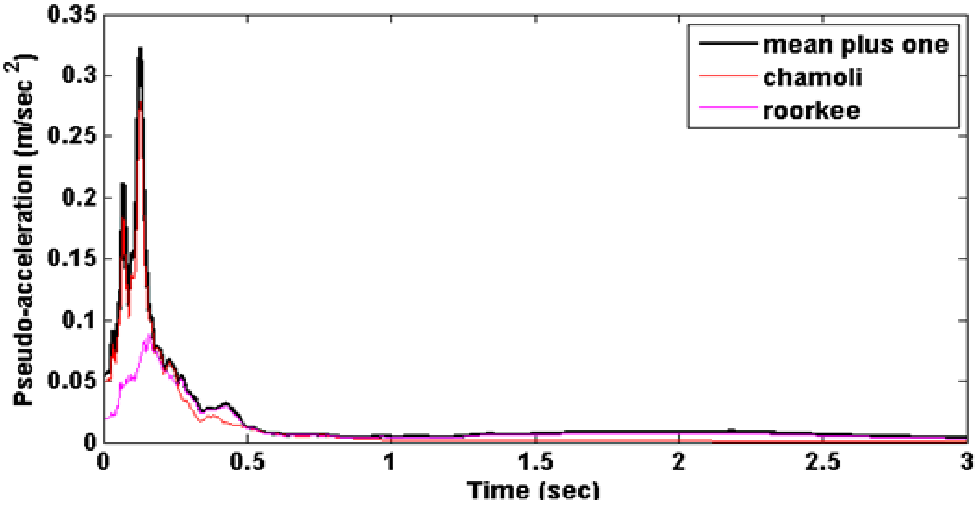
India Bangladesh border 1997



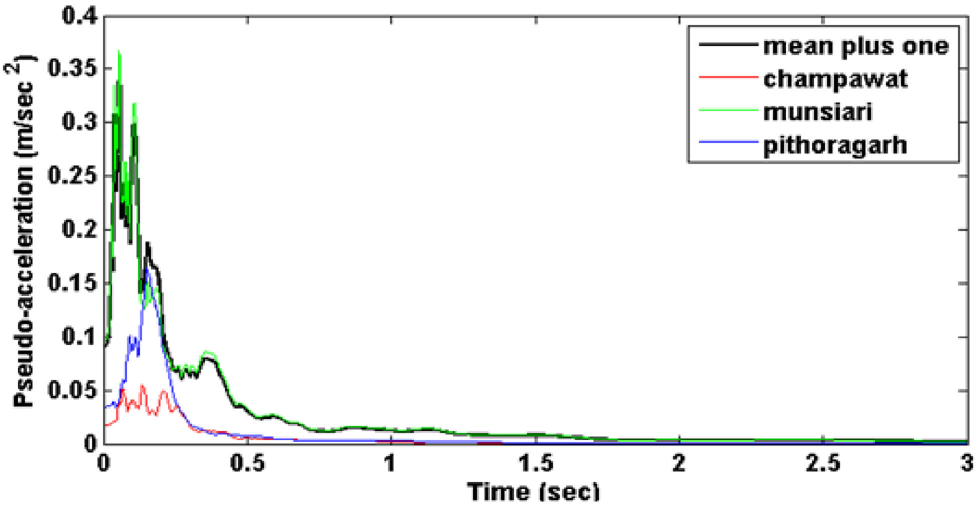
Chamoli 1999



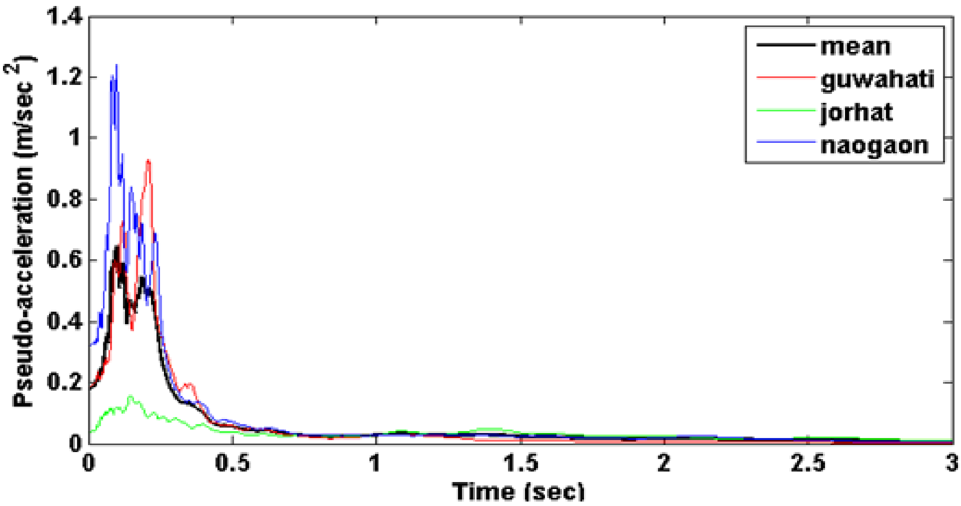
Chamoli 2005



Uttarkashi 2007



Uttarakhand



Manipur

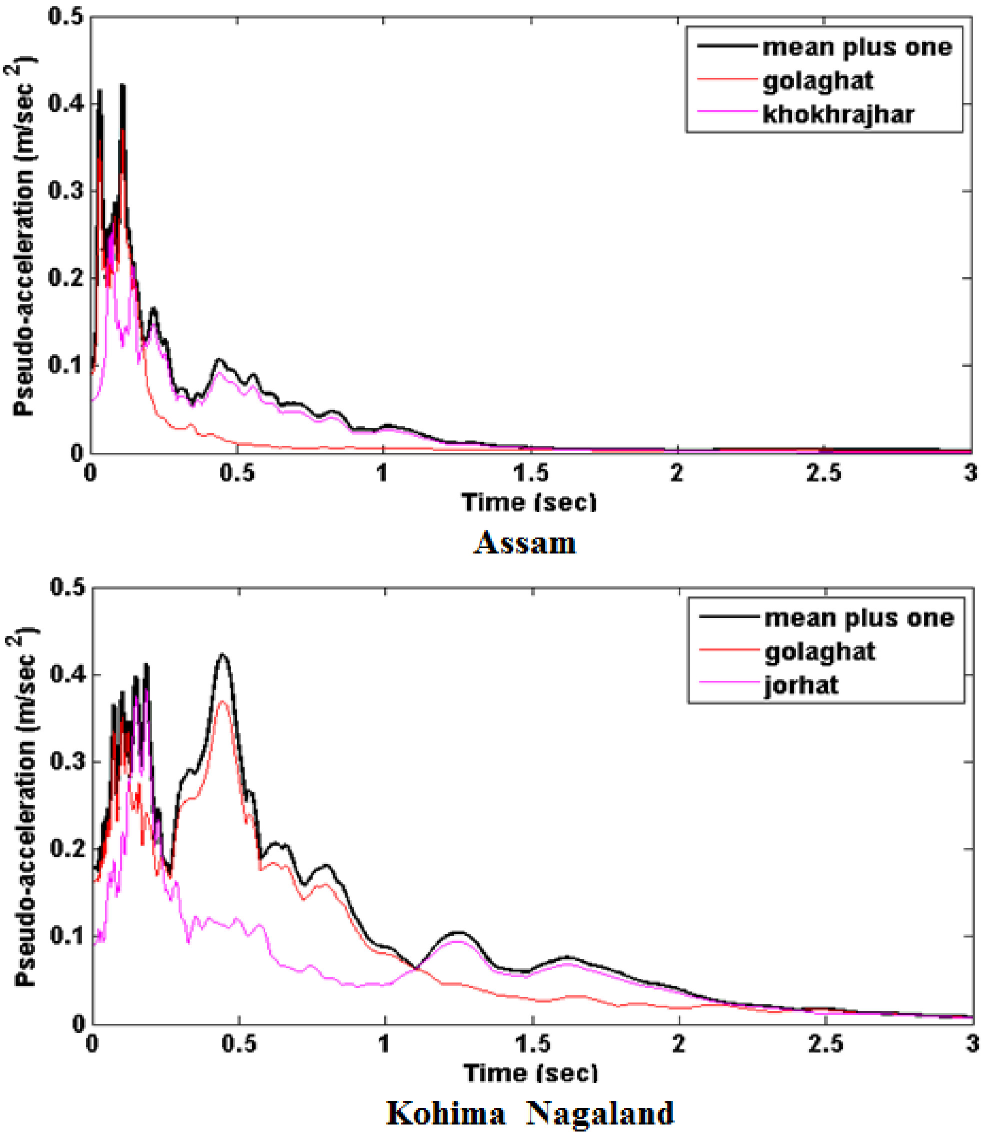
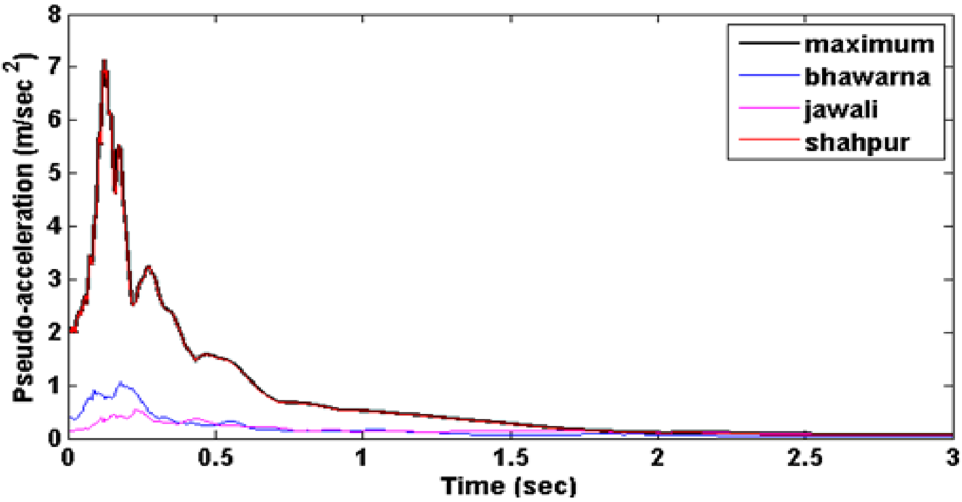
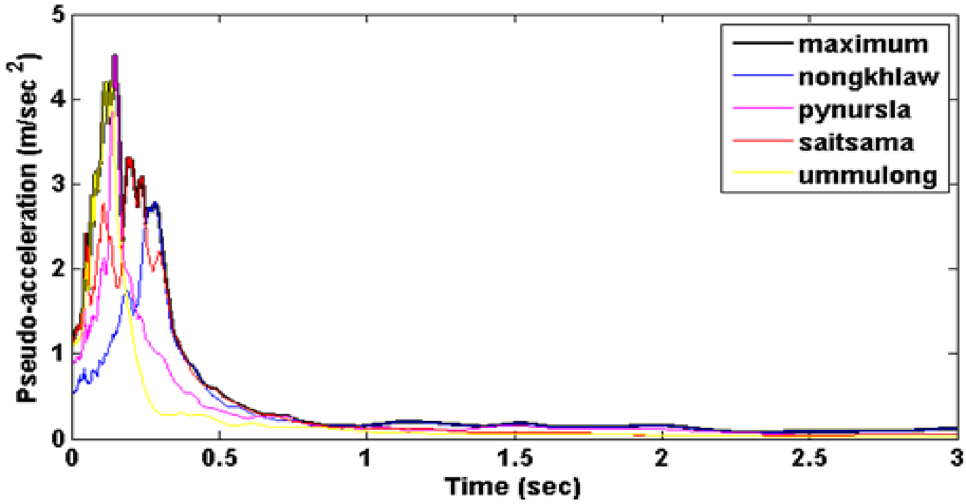


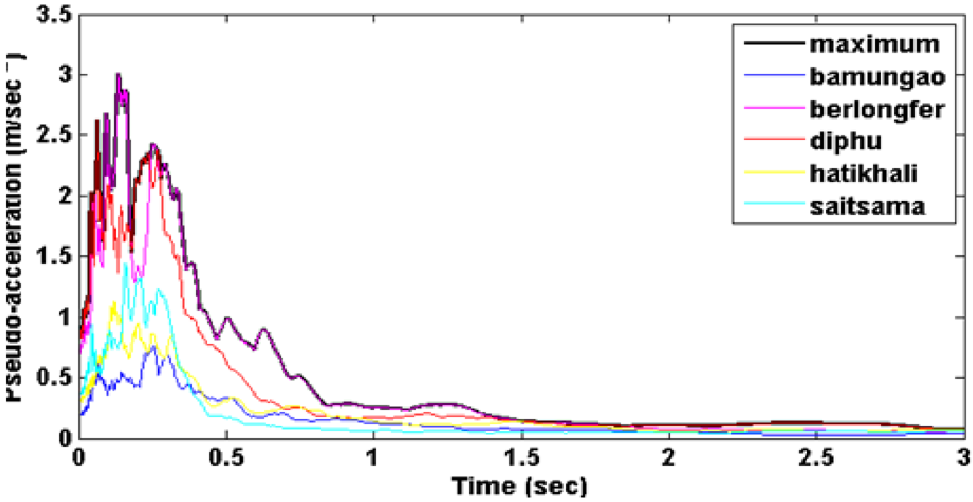
Figure 5.3: Mean +1 σ response spectrum for 5% damping of horizontal ground motions



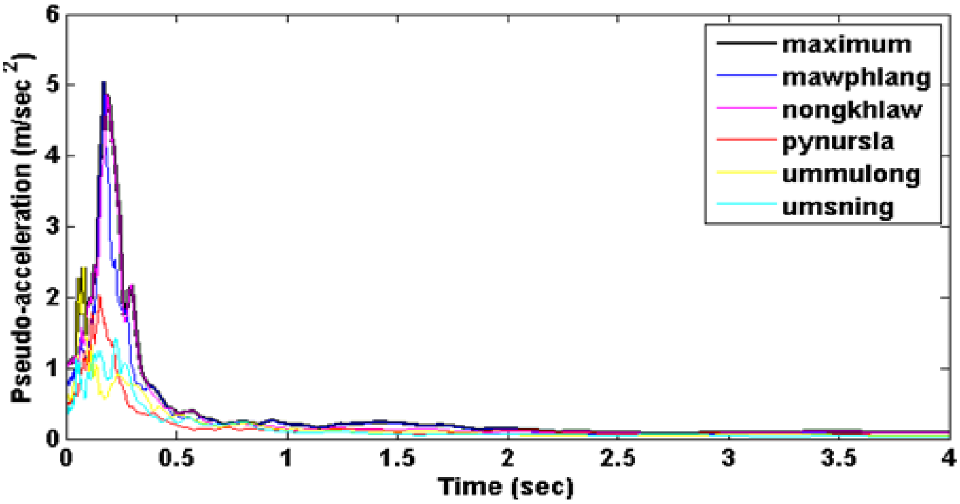
Dharmsala



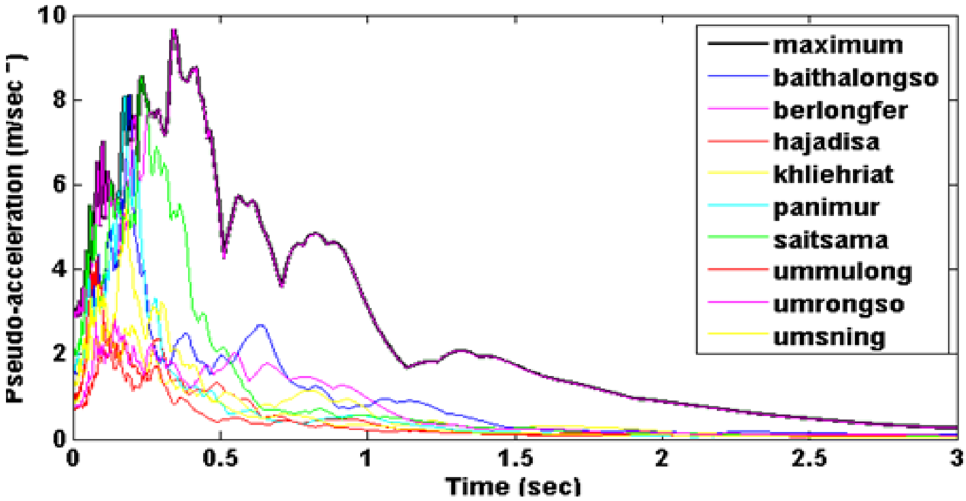
North-East India



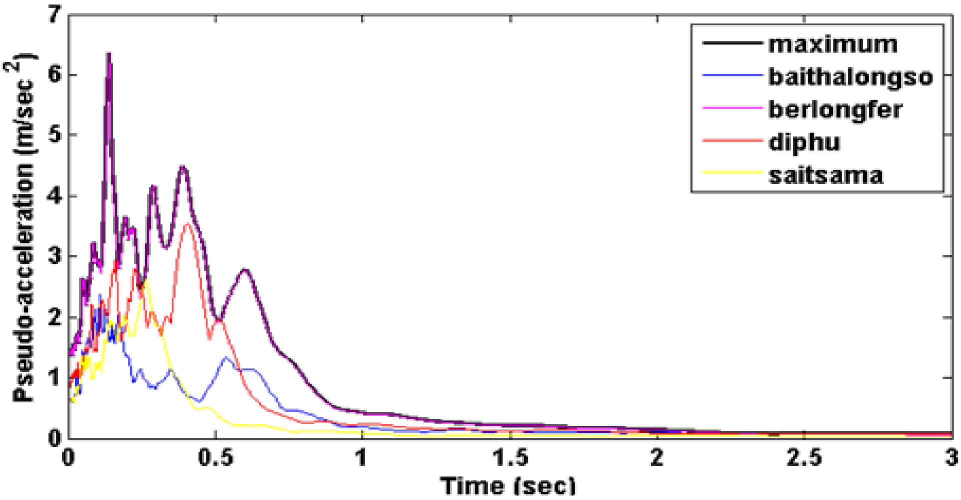
India Burma border 1987



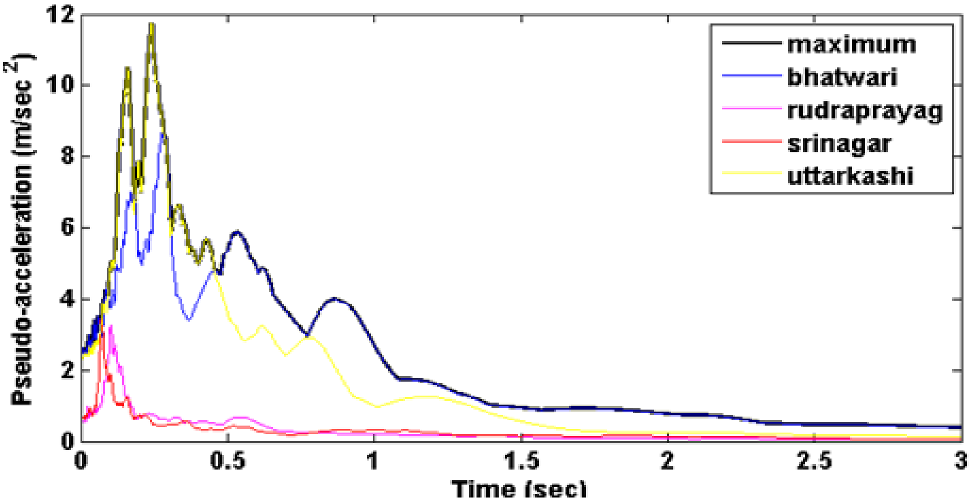
India Bangladesh border 1988



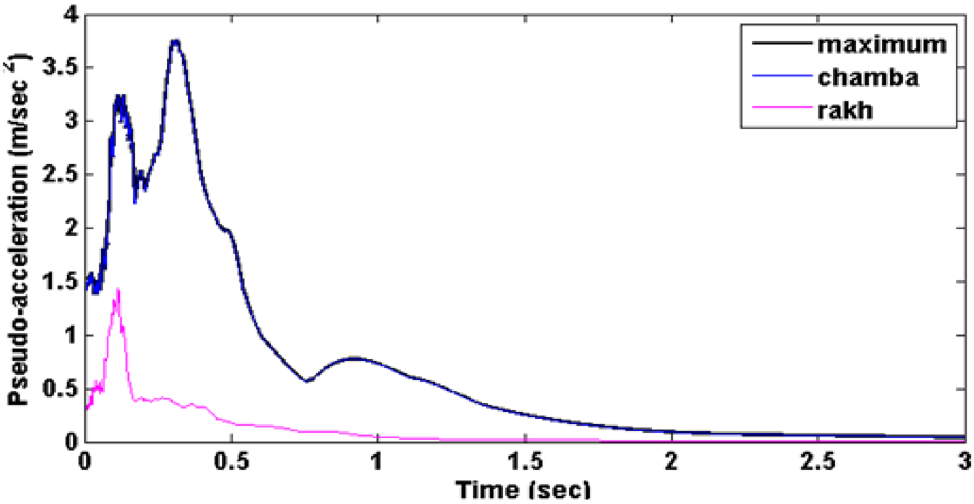
India Burma border 1988



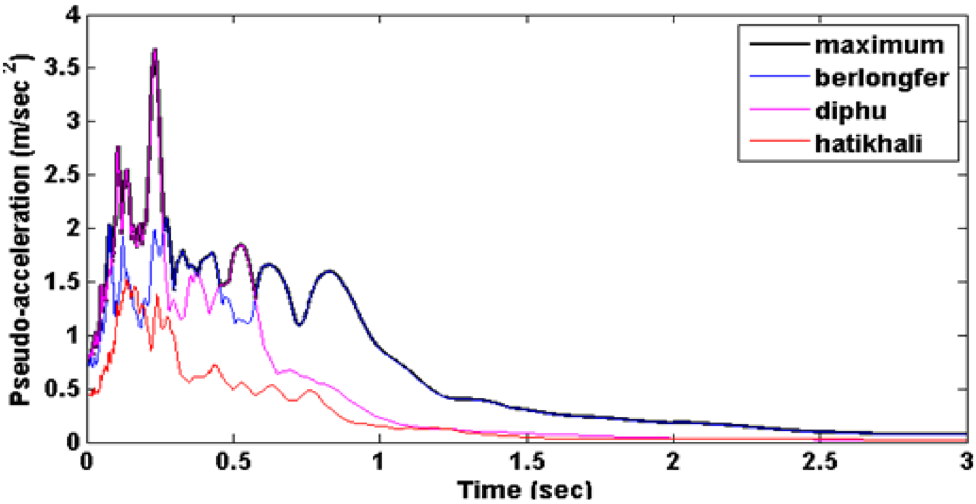
India Burma border 1990



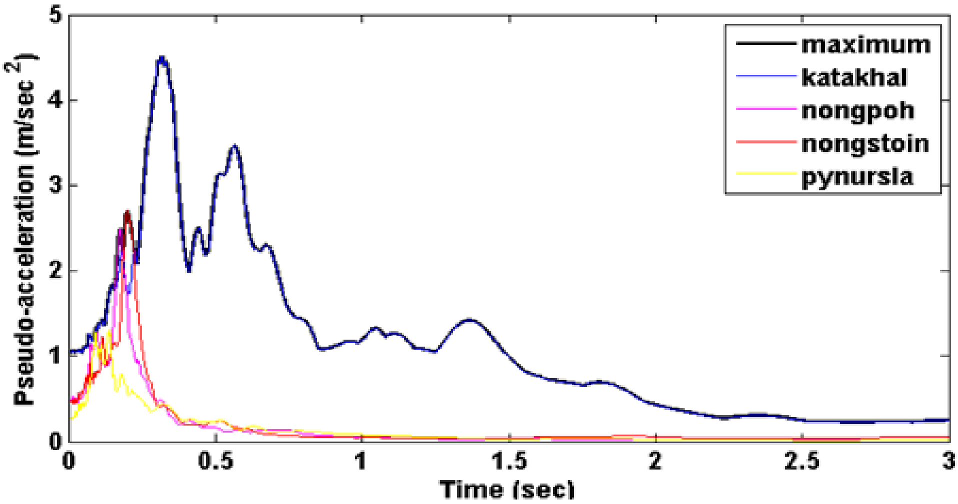
Uttarkashi 1991



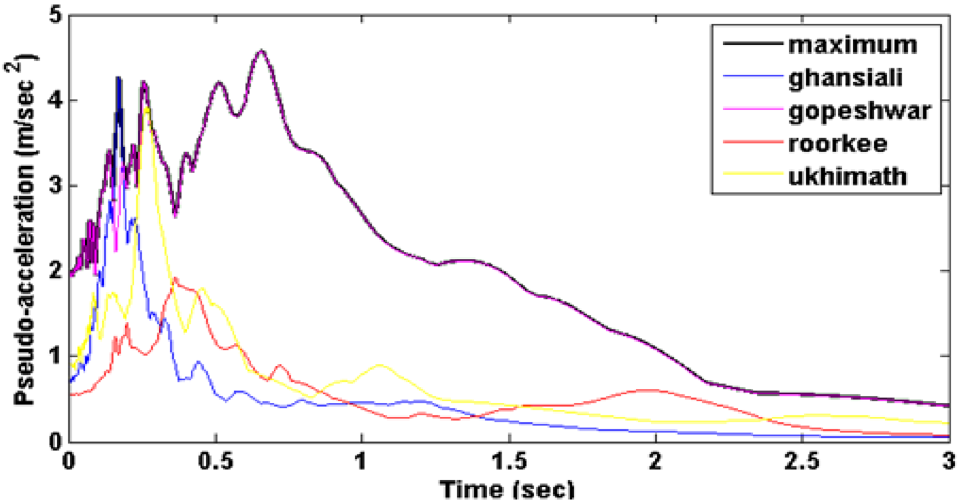
Chamba



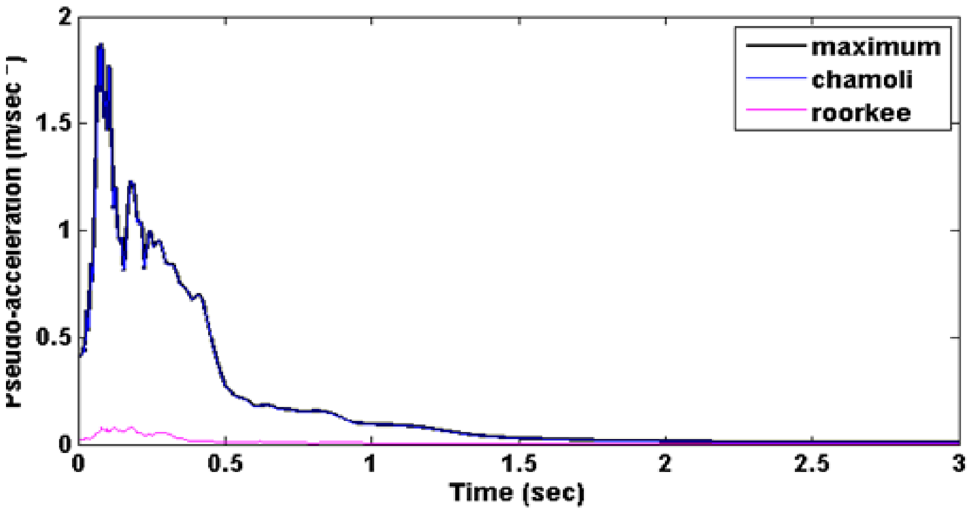
India Burma border 1995



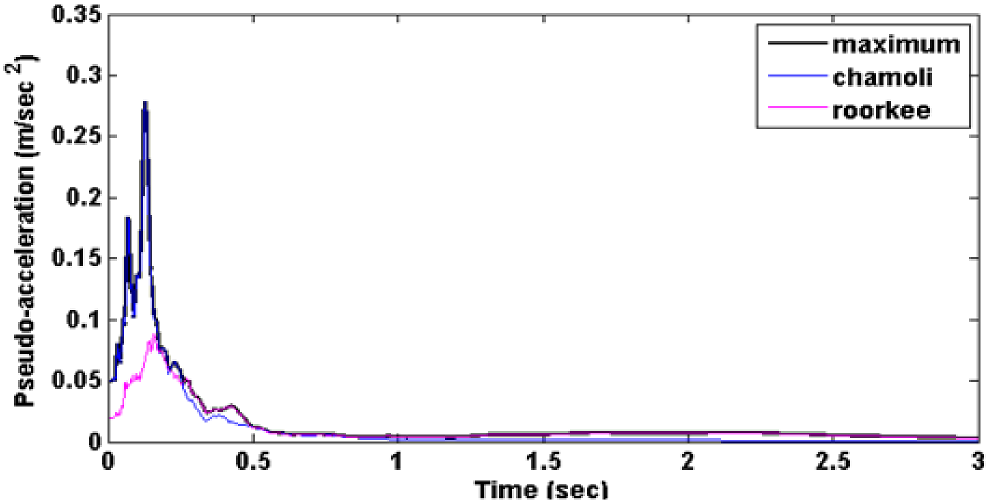
India Bangladesh border 1997



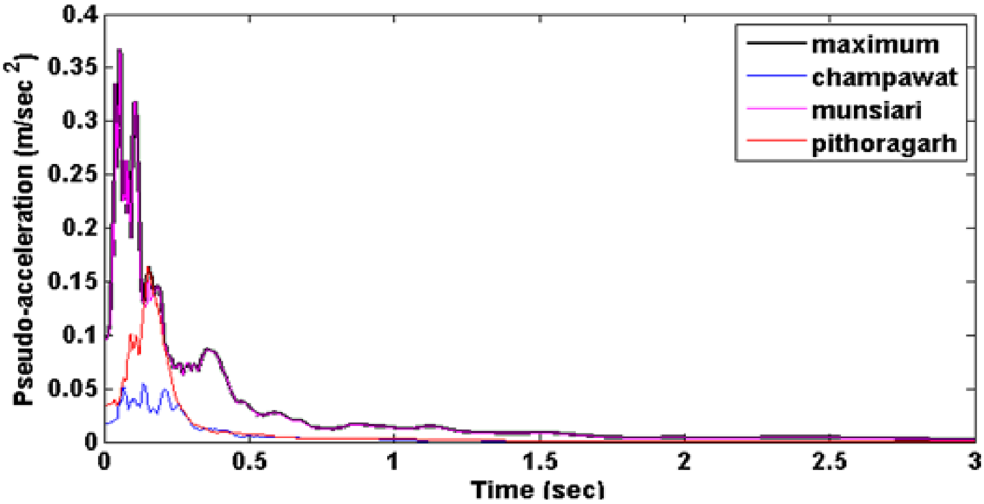
Chamoli 1999



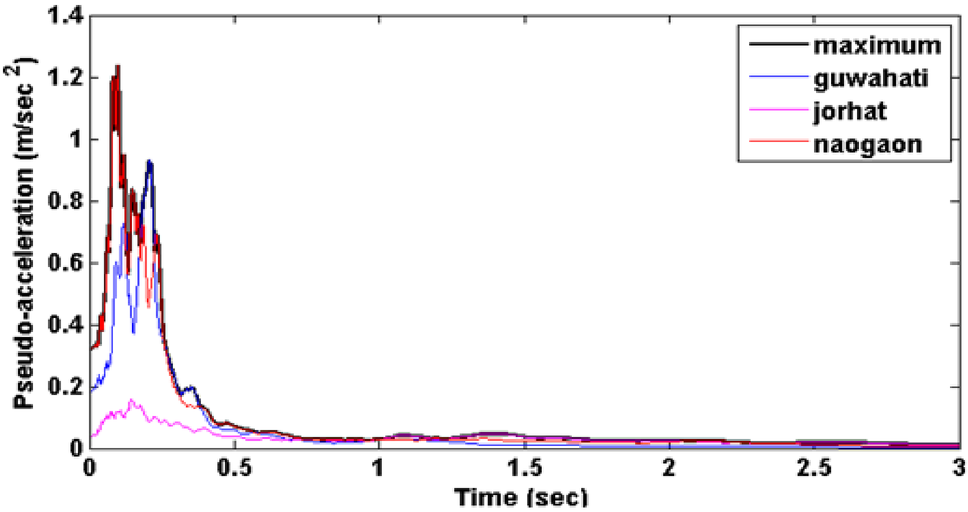
Chamoli 2005



Uttarkashi 2007



Uttarakhand



Manipur

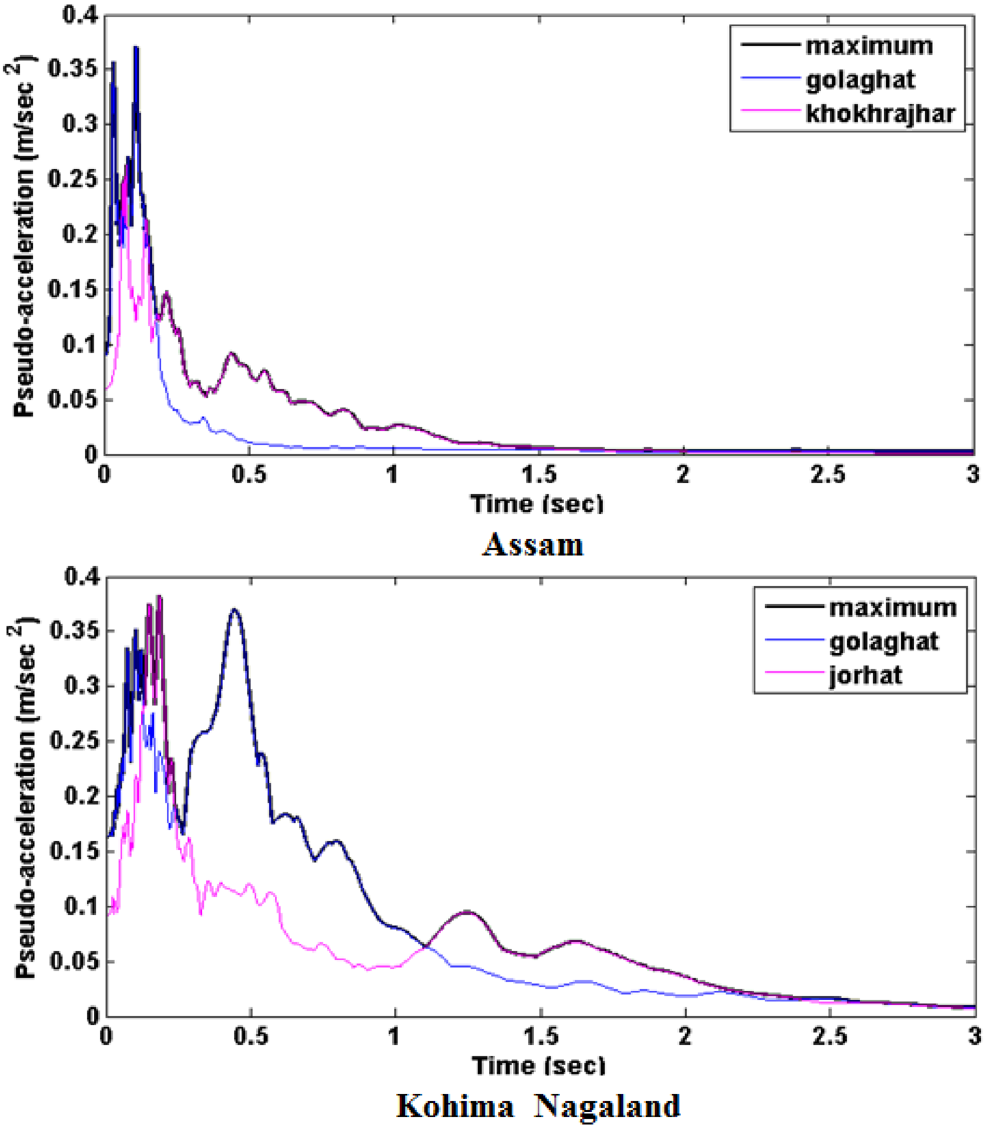


Figure 5.4: Maximum response spectrum for 5% damping of horizontal ground motions

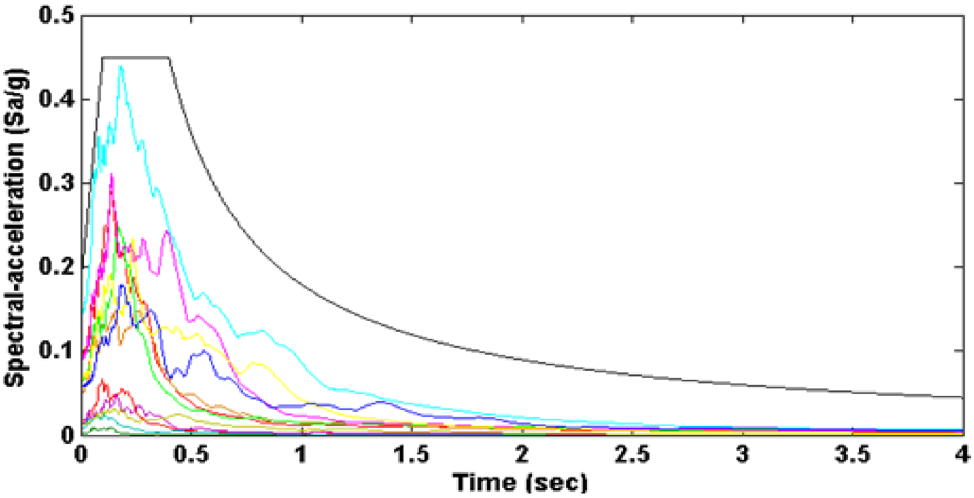
It is observed that the mean, mean plus one standard deviation and maximum response spectrum are much smoother than the response spectrum for an individual ground motion.

From the Figure 5.2, 5.3 and 5.4, following points are concluded:

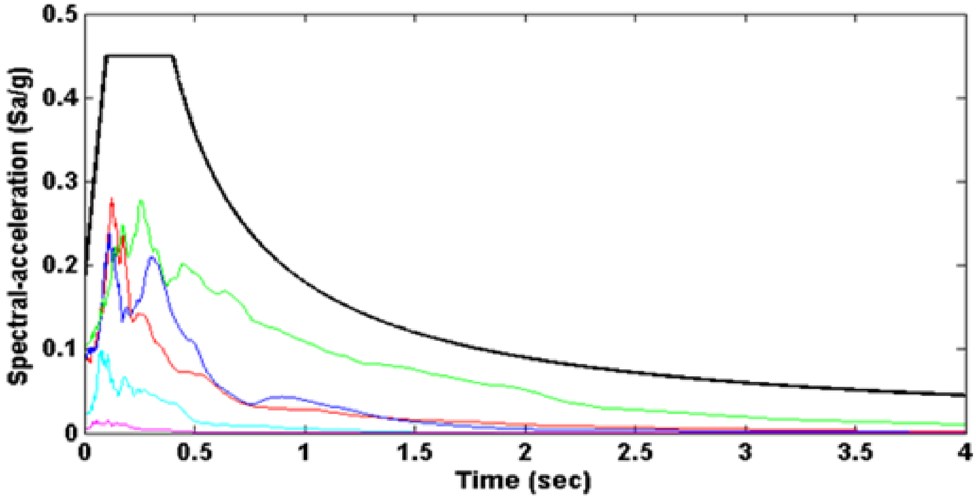
- All the three mean, mean plus one standard deviation and maximum response spectrum shows that system with higher natural time period (beyond 1.5 second) are least affected.
- Mean Response Spectrum derived shows that most of the strong motion response are below mean response spectrum for all the regions of the country.
- On the other hand, Mean plus one standard deviation response spectrum, it was found that all strong ground motion response are well below.
- Thus it would be good enough to represent strong ground motion using Mean plus one standard deviation response spectrum

5.4 Response Spectrum comparison with Code based spectrum for four regions of India

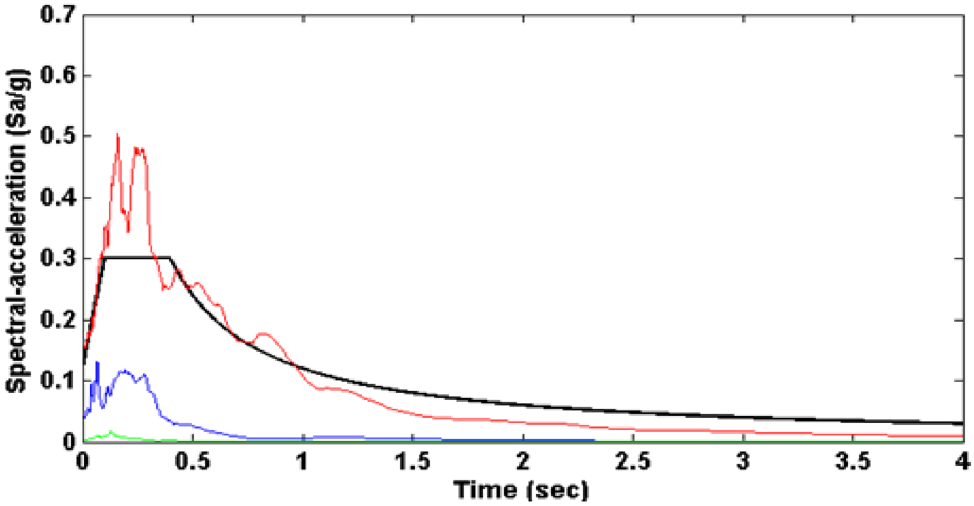
With the help of the plots presented in above section, response spectrum for four regions North, East, South and West of India as classified in Section 4.3 are compared. Most of the records are obtained at sites in the east and north region of the country but a limited number of records are available in south and west region. Response spectrum of IS:1893-2002 is multiplied with 0.5 times zone factor to obtain spectral response for a particular site. It is compared with representative response spectrum obtained for all regions of the country. Figure 5.5 - 5.7 shows response spectrum for all regions and its comparison with design spectrum given in code.



East region



North region zone 5



North region zone 4

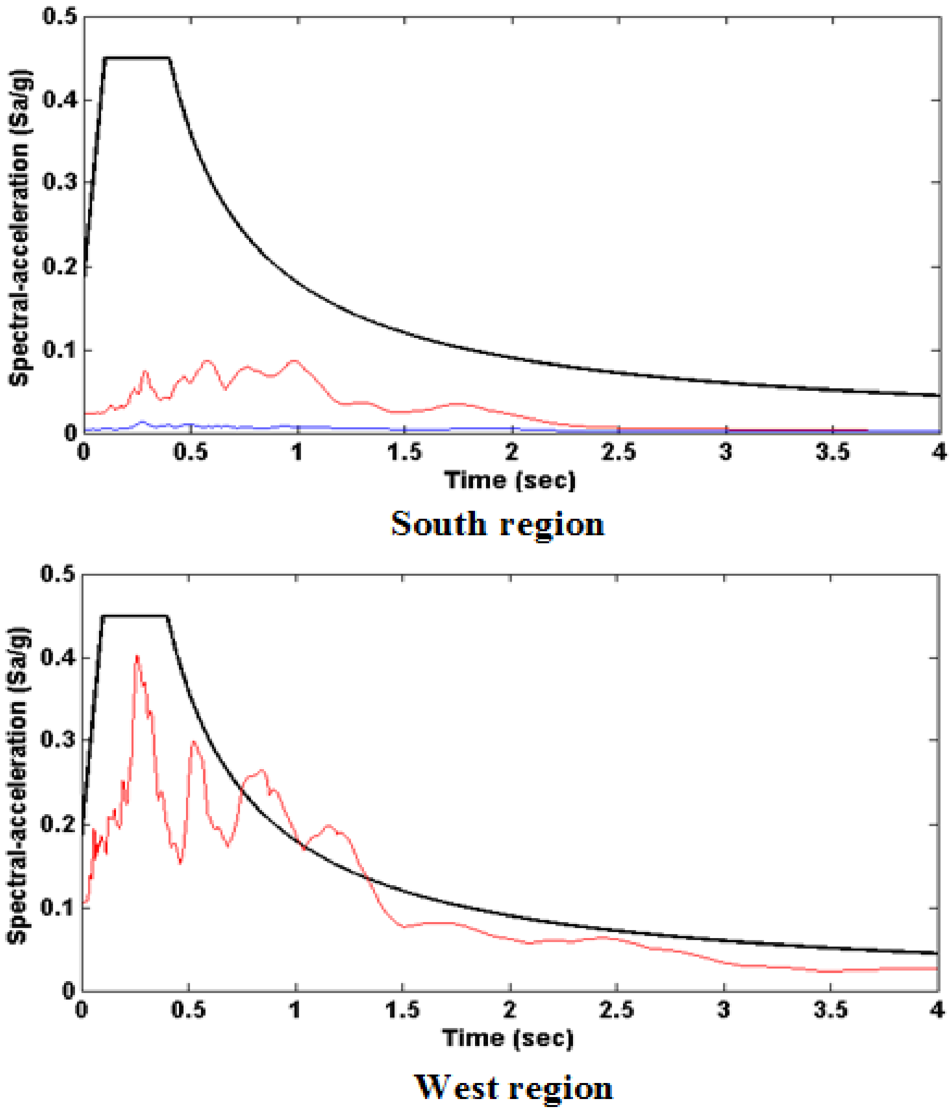
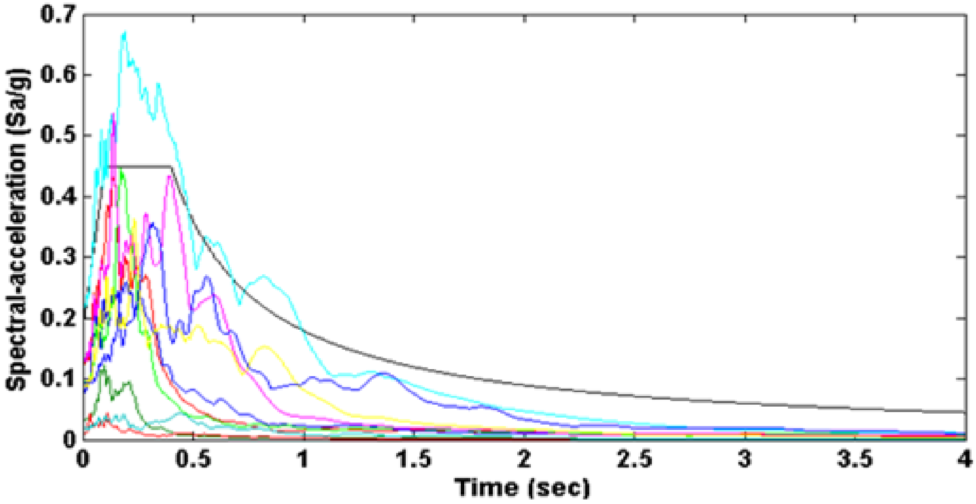


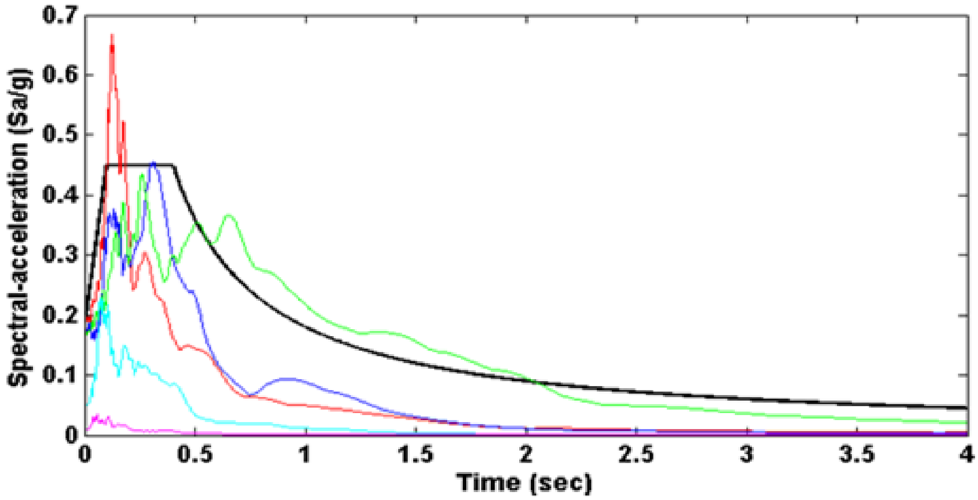
Figure 5.5: Mean response spectrum for different regions of the country

Observations

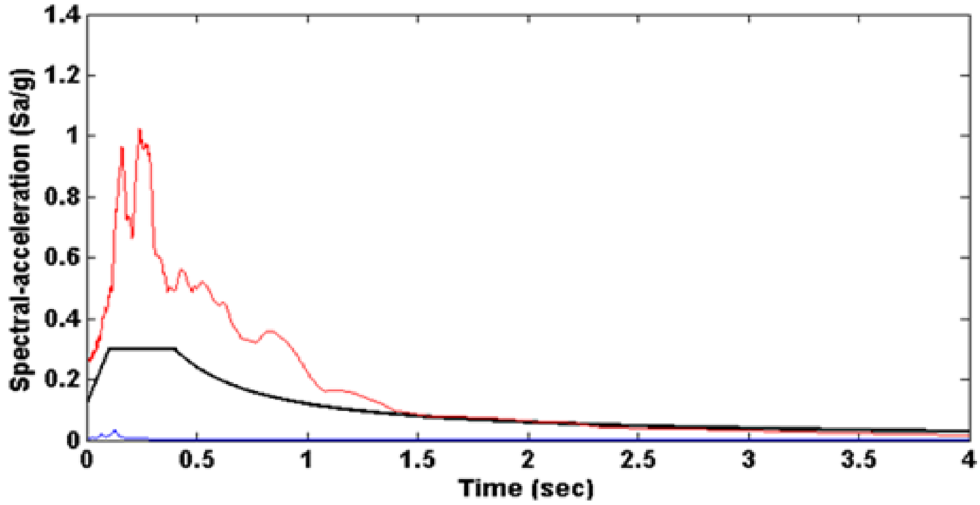
- It is seen that spectral acceleration for east and north region (zone 5) is lower than required by the code based design spectrum.
- Acceleration amplification for west region extends over a large frequency region.
- Based on limited strong motions, the current design spectrum of IS:1893-2002 is too conservative for south region of the country.
- Spectral acceleration for east region is lower than required by the code based design spectrum.
- For stiff systems (Tn$<$0.65s), west region shows less response compared to code design spectrum while for flexible systems (Tn$>$0.65s) it is in good agreement with design spectrum.



East region

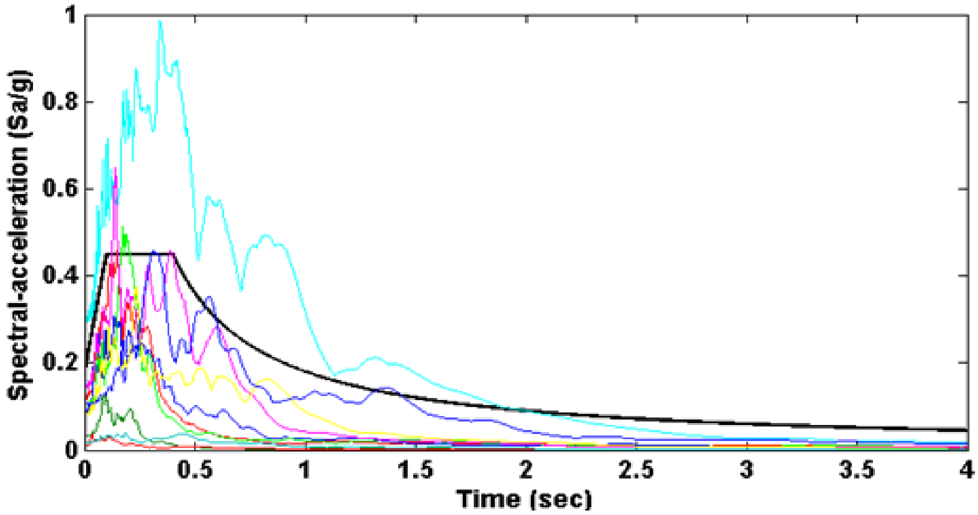


North region zone 5

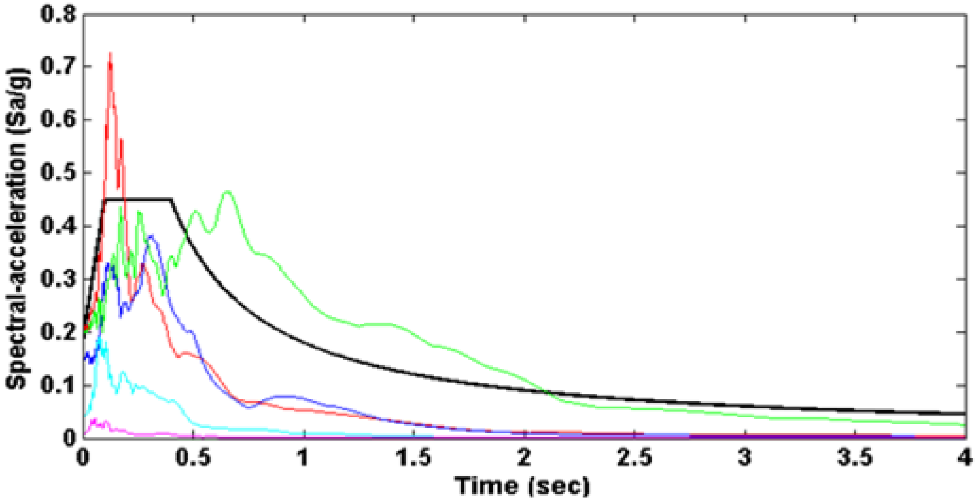


North region zone 4

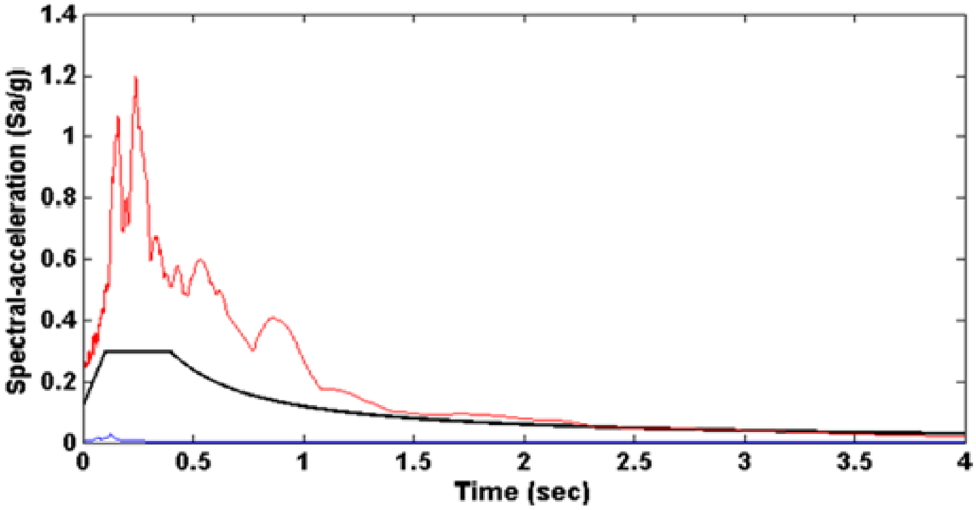
Figure 5.6: Mean +1 σ response spectrum for different regions of the country



East region



North region zone 5



North region zone 4

Figure 5.7: Maximum response spectrum for different regions of the country

5.5 Observations

Records of strong earthquake ground motions are limited in number for West region followed by South region. So it is difficult to predict the deficient and conservative part of the design spectrum given in IS:1893-2002. Two response spectrum in zone 4 of north region are not reliable to predict seismic force, as one is showing relatively more response than other. Based on limited strong ground motions, the current design spectrum of IS:1893-2002 is too conservative for south region of the country. It is observed that response spectrum of western part is generally in good agreement with code spectrum for flexible systems.

Mean response spectrum derived shows that almost all strong motions fall below design spectrum for all regions of the country. It is also seen that spectral acceleration for east region from mean $+1\ \sigma$ response spectrum is lower than required by the code design spectrum.

It is readily apparent from the Figure 5.5,5.6 and 5.7 that there are wide differences in spectral shapes for all three proposed response spectrum depending on region. Particularly at periods greater than 0.5 sec spectral amplifications are much higher for South and West region than for North and East region.

The analysis shows clear differences in spectral shapes for different regions of India, indicating the need for consideration of these effects in selecting earthquake-resistant design criteria. The plots indicate that the acceleration amplification for west region extends over a larger frequency region than the amplification for other categories (south, north and east region). The maximum acceleration amplification for north and east region is greater than the maximum amplification for either south and west region for stiff system for all proposed response spectrum. This indicates that Mean and Mean plus one standard deviation response spectrum derived for North and East region shows sizable effect for stiff system with less natural time period.

5.6 Summary

Total 23 recorded earthquake events are studied and response spectrum is generated for all qualified strong ground motions. Analysis on a statistical basis is carried out and three representative response spectrum (mean, mean plus one σ and maximum) are developed. The proposed response spectrum for all regions are compared with design spectrum given in code.

Chapter 6

Case study of G+3 storey building

6.1 Introduction

The main purpose of dynamic analysis in earthquake engineering is the estimation of earthquake induced forces and deformations in structures under the action of earthquake ground motions. The chapter deals with the dynamic analysis of 4 - storey RC framed building. For the building as shown in Figure 6.1, the dynamic properties (natural periods, and mode shapes) for vibration are obtained by carrying out a free vibration analysis (Table 6.1). The design seismic force by the dynamic analysis method as outlined in cl. 7.8.4.5 of IS: 1893 (Part I)-2002 is carried out. In subsequent section, estimated lateral load using design response spectrum of IS:1893(Part-I)-2002 is compared with lateral load obtained from proposed response spectrum for different regions of the country for this building.

6.2 Geometry of G+3 storey building

- No. of Storey = G+3 Storey
- Story Height = 3 m
- Slab Thickness. = 120 mm
- No. of Bays in X-Direction = 3

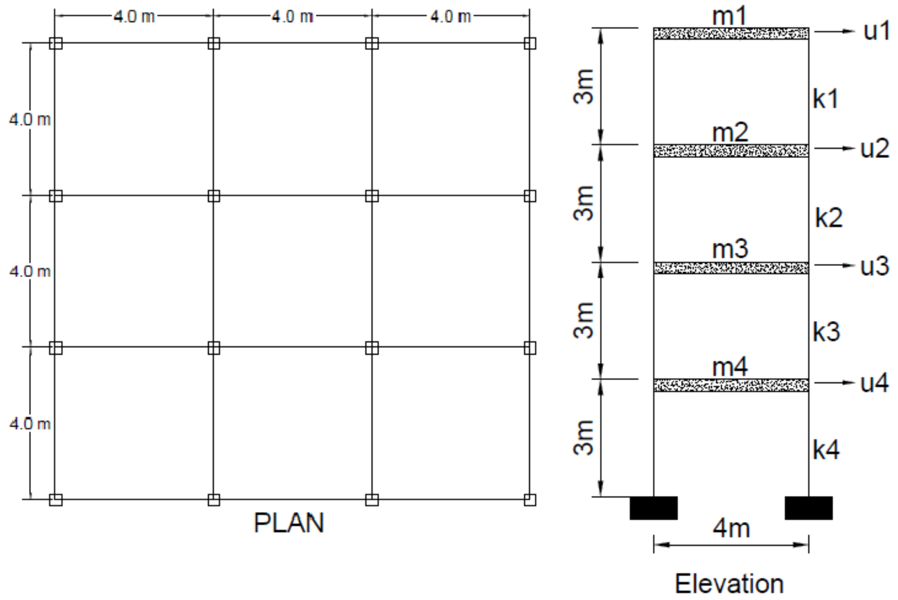


Figure 6.1: Plan and Elevation of G+3 storey building

- No. of Bays in Y-Direction = 3
- Bay Width in X-Direction = 4 m
- Bay Width in Y-Direction = 4 m
- Column Size = 0.3 m × 0.3 m
- Beam Size = 0.23 m × 0.3 m
- f_{ck}= 25 N/mm^2 (M 20 grade of concrete)
- f_y = 415 N/mm^2 (Fe 415 grade of steel)
- Live Load on Typical Storey = 3 kN/m^2

6.3 Analysis and Design of G+3 storey building

Dynamic analysis is performed to obtain the design seismic force and its distribution to different levels along the height of the building. It is performed either by the response spectrum method or by the time history method. In the present work, Response spectrum method of analysis is performed for estimating lateral load of the building.

Consider a four-storey reinforced concrete building as shown in Figure 6.1 located in seismic zone V having soil condition of rock deposit. The R.C. frames are without brick infilled panels. The mathematical model consists of square columns with infinitely rigid beams. The entire mass of each storey is assumed to be lumped at its level with total value of typical storey mass. Dynamic properties of the building like mass matrix and stiffness matrix is determined using lumped mass modelling approach.

Determination of Lumped mass matrix:

The mass matrix is a diagonal matrix in which each element represents the total equivalent entire mass of the storey as a concentrated lumped mass at that level. Therefore the lumped mass matrix is given by:

$$\mathrm{M} = \begin{bmatrix} 82935.78 & 0 & 0 & 0 \\ 0 & 82935.78 & 0 & 0 \\ 0 & 0 & 82935.78 & 0 \\ 0 & 0 & 0 & 66422.02 \end{bmatrix} \mathrm{kg}$$

Determination of Stiffness matrix:

The stiffness of columns with two ends fixed against rotation is given by: K_c=12EI_c/h^3. where h is storey height, I_c is moment of inertia of column section and E is modulus of elasticity.

So the stiffness matrix of the structure is given by:

$$K = \begin{bmatrix} 240000 & -120000 & 0 & 0 \\ -120000 & 240000 & -120000 & 0 \\ 0 & -120000 & 240000 & -120000 \\ 0 & 0 & -120000 & 120000 \end{bmatrix} \text{kN/m}$$

Natural vibration frequencies and corresponding vibration mode shapes:

The natural vibration frequencies and corresponding mode shapes are determined by solving the equation $[K - \omega_n^2 M]\phi = 0$ through MATLAB.

This equation is called an eigenvalue problem. The quantities ω_n^2 are the eigenvalues indicating the square of free vibration frequencies, while the corresponding displacement vectors ϕ represent the corresponding mode of vibrating system known as the eigenvectors or mode shapes. The mode having the lowest frequency is called the first mode or the fundamental mode, the next higher frequency is the second mode, etc.

Natural frequencies of various modes calculated through MATLAB are :

$\omega_1 = 13.81 rad/sec.$ $\omega_2 = 39.46 rad/sec.$ $\omega_3 = 59.62 rad/sec.$ $\omega_4 = 72.01 rad/sec.$

Mode shape matrix is given by: $\phi = \begin{bmatrix} 0.359 & -0.9443 & 1.222 & -1.239 \\ 0.671 & -0.8724 & -0.558 & 1.962 \\ 0.8944 & 0.1381 & -0.9675 & -1.87 \\ 1 & 1 & 1 & 1 \end{bmatrix}$

Mode shape vectors for different modes are:

$$\phi_1 = \begin{bmatrix} 0.359 \\ 0.671 \\ 0.8944 \\ 1 \end{bmatrix}, \phi_2 = \begin{bmatrix} -0.9443 \\ -0.8724 \\ 0.1381 \\ 1 \end{bmatrix}, \phi_3 = \begin{bmatrix} 1.222 \\ -0.558 \\ -0.9675 \\ 1 \end{bmatrix}, \phi_4 = \begin{bmatrix} -1.239 \\ 1.962 \\ -1.87 \\ 1 \end{bmatrix}$$

Determination of natural time period:

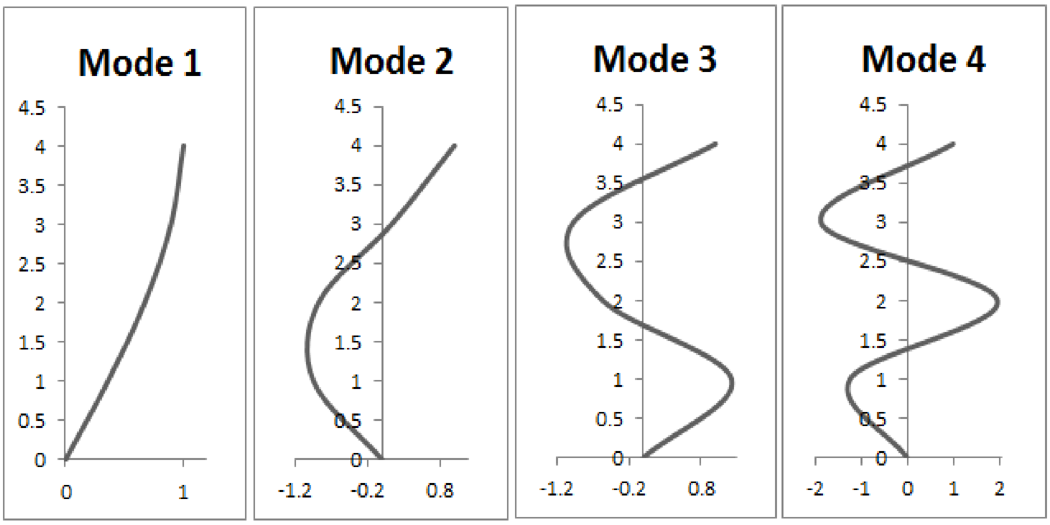


Figure 6.2: Four mode shapes for the building

The period (T) of motion is given as a function of frequency as $T = 2\pi/\omega_n$.
It means that each mode shape of vibration has relative period.
The period of 1^{st} , 2^{nd}, 3^{rd} and 4^{th} mode shapes are given respectively: (0.4547, 0.159, 0.1053, 0.0872 sec). Note that the period of the first mode shape is the biggest one (T=0.4547 sec) which is called the fundamental period. The next lesser one is come with second mode shape.

Determination of dynamic quantities:

The seismic force is obtained by dynamic analysis method as outlined in cl. 7.8.4.5 of IS 1893(Part-I)-2002.

For the building shown in Figure 6.2, the dynamic properties (natural periods, and mode shapes) for vibration have been obtained by carrying out a free vibration analysis (Table 6.1). Modal mass and modal participation factors of each mode are obtained and presented in Table.

It is seen from the Table 6.1 that the first mode excites 89.64% and second mode excites 8.2% of the total mass. Hence, in this case, codal requirements on number of

Table 6.1: Calculation of modal mass and modal participation factor

Storey Level i	**Weight W_i (kN)**	**Mode 1**			**Mode 2**		
		ϕ_{ik}	$W_i\phi_{ik}$	$W_i\phi_{ik}^2$	ϕ_{ik}	$W_i\phi_{ik}$	$W_i\phi_{ik}^2$
4	651.6	1.000	651.6	651.6	1.000	651.6	651.6
3	813.6	0.8944	727.68	650.84	0.1381	112.36	15.52
2	813.6	0.671	545.92	366.31	-0.8724	-709.78	619.22
1	813.6	0.359	292.1	104.86	-0.9443	-768.28	725.49
Σ	3092.4		2217.3	1773.61		-714.1	2011.83
M_k		**282566.77 kg**			**25837.92 kg**		
% of Total weight		**89.64%**			**8.2%**		
P_k		**1.25**			**-0.3549**		

Storey Level i	**Weight W_i (kN)**	**Mode 3**			**Mode 4**		
		ϕ_{ik}	$W_i\phi_{ik}$	$W_i\phi_{ik}^2$	ϕ_{ik}	$W_i\phi_{ik}$	$W_i\phi_{ik}^2$
4	651.6	1.000	651.6	651.6	1.000	651.6	651.6
3	813.6	-.9675	-787.16	761.6	-1.87	-1521.43	2845.07
2	813.6	-0.558	-454	253.3	1.962	1596.3	3131.91
1	813.6	1.222	994.22	1214.94	-1.239	-1008.05	1248.97
Σ	3092.4		404.66	2881.44		-281.58	7877.55
M_k		**5793.07 kg**			**1026 kg**		
% of Total weight		**1.83%**			**0.325%**		
P_k		**0.14**			**-0.036**		

modes to be considered such that at least 90% of the total mass is excited, will be satisfied by considering the first and second mode of vibration.

The lateral load Q_{ik} acting at i^{th} floor in the k^{th} mode is

$$Q_{ik} = A_{ik}\phi_{ik}P_kW_i \tag{6.1}$$

The value of A_k for different modes is obtained from clause 6.4.2 of IS:1893(Part-I)-2002.

It is very important to know that the SRSS method is fundamentally sound when the modal frequencies are well separated. However, when the frequencies of major contributing modes are very close together, the SRSS method can give poor results, in which case the more general complete quadratic combination (CQC) method is

Table 6.2: Lateral load calculation by modal analysis method

Storey Level i	**Weight W_i (kN)**	**Mode 1**			**Mode 2**		
		ϕ_{ik}	Q_{ik}	V_{ik}	ϕ_{ik}	Q_{ik}	V_{ik}
4	651.6	1.000	65.16	65.16	1.000	-20.81	-20.81
3	813.6	0.8944	72.77	137.93	0.1381	-3.6	-24.41
2	813.6	0.671	54.6	192.53	-0.8724	22.67	-1.74
1	813.6	0.359	29.21	221.74	-0.9443	24.54	22.8

Storey Level i	**Weight W_i (kN)**	**Mode 3**			**Mode 4**		
		ϕ_{ik}	Q_{ik}	V_{ik}	ϕ_{ik}	Q_{ik}	V_{ik}
4	651.6	1.000	8.21	8.2100	1.000	-1.955	-1.955
3	813.6	-.9675	-9.92	-1.71	-1.87	4.564	2.61
2	813.6	-0.558	-5.72	-7.43	1.9620	-4.8	-2.19
1	813.6	1.222	12.53	5.1	-1.239	3.024	0.833

used. Since all of the modes here are well separated (clause 3.2 IS:1893(Part-I)-2002), the contribution of different modes is combined by the SRSS (square root of the sum of the square) method. V_4= 68.92 kN, V_4= 140.11 kN, V_4= 192.69 kN, V_4= 222.97 kN

Clause 7.8.2 says that the base shear obtained by dynamic analysis (V_B = 222.97 kN) is compared with that obtained from empirical fundamental period as per Clause 7.6. If V_B is less than that from empirical value, the response quantities are to be scaled up.

Base shear calculated using empirical fundamental period is 230.3 kN. Now dynamic analysis gives us base shear as 222.97 kN which is lower. Hence all the response quantities are to be scaled up in the ratio (230.3/222.97 = 1.033). Thus the seismic forces obtained by dynamic analysis are scaled and results are presented in Table 6.3.

Table 6.3: Base shear at different storeys

Floor level i	Q (static)	Q (dynamic scaled)	Storey Shear V (static)	Storey Shear V (dynamic scaled)
4	110.1	71.19	110.1	71.19
3	77.29	76.1	187.39	144.73
2	34.31	59.59	221.7	199.05
1	8.59	40.27	230.3	230.3

It is noticed that even though the base shear by the static and dynamic analysis are comparable, there is considerable difference in the lateral load distribution with building height. So it is considered as one of the advantage of dynamic analysis.

6.4 Lareral load estimation of G+3 storey building

The seismic zone map for India divides the country into four seismic zones (II, III, IV and V). So the recorded time histories listed in Table 4.4 are assigned to four groups corresponding to seismic zones II, III, IV and V based on the location of recording station. Thus Uttarkashi and Xizang-India border time histories correspond to seismic zone IV while remaining time histories correspond to seismic zone V.

Fundamental Time Period

The seismic response of a structure depends upon its fundamental time period. The fundamental time period of structures as per IS: 1893 (Part I)-2002 is evaluated from the empirical expression given in Clause. 7.6.1 of IS: 1893 (Part I)-2002 and is expressed in Equation 6.2 as follows

$$T_a = 0.075h^{0.75} \tag{6.2}$$

Where T_a corresponds to fundamental natural period of vibrations in seconds and h for height of building in meter.

Further the time period of frame structure is calculated through dynamic analysis shown in Section 6.3. For this RC building, time period using IS: 1893 (Part I)-2002 and that obtained using dynamic analysis are 0.4835 sec. and 0.4547 sec. respectively.

Spectral Acceleration Coefficient Comparison

According to IS: 1893 (Part I)-2002, Sa/g is Structural Response Factor denoting the acceleration response of the structure subjected to earthquake ground vibrations and it depends on the natural period and damping of the structure. The values of Sa/g are obtained from design acceleration spectrum, which refers to maximum acceleration as a function of time period for a specified damping. These values of Sa/g are calculated considering the fundamental period of structure as obtained from dynamic analysis for different earthquake excitations. It is noted that design acceleration spectrum is developed by considering seismic zone factor whereas response spectrum derived for various regions of the country incorporates zone factor in itself. So design horizontal seismic coefficient of IS: 1893 (Part I)-2002 is compared with the Sa coefficient values for various regions of our country. In Table 6.4, 6.5, 6.6 and 6.7, Sa coefficient values of IS: 1893 (Part-I) 2002 are compared with the Sa coefficient values of three response spectrum (mean, mean plus one sigma, maximum) for past recorded 23 earthquake events.

Base Shear

Base shear is the total design lateral force at the base of a structure. It is m times pseudo-acceleration. So the peak values of shear at the base is given by

$$V_b = mA \tag{6.3}$$

Base shear considering IS: 1893 response spectrum and three representative response spectrum for different regions of the country are compared and shown in Table (6.4 - 6.7) and presented in Figure (6.3 - 6.8).

Table 6.4: Base shear and Spectral acceleration comparison for East region of the country with code based design spectrum

Region		T_n	Response spectrum			T_n	Response spectrum			IS: 1893-2002	
		Program calculated	mean	mean +1 σ	max imum	Empirical equation	mean	mean +1 σ	max imum	static analysis	dynamic analysis
North-East India	A		0.4698	0.6431	0.6244		0.4212	0.5756	0.5896	3.656	3.88
	V_B		29.62	40.54	39.37		26.55	36.29	37.17	230.5	244.62
Ind- Bur 1987	A		0.4914	0.8156	0.9048		0.48	0.7892	0.9011	3.656	3.88
	V_B		30.98	51.42	57.04		30.26	49.76	56.81	230.5	244.62
Ind-Bangla 1987	A		0.3593	0.4495	0.4571		0.3171	0.393	0.4262	3.656	3.88
	V_B		22.65	28.34	28.82		19.99	24.78	26.87	230.5	244.62
Ind-Bur 1988	A		2.0279	4.1	7.2982		1.8105	3.5705	6.3	3.656	3.88
	V_B		127.85	258.49	460.12		114.14	225.11	397.19	230.5	244.62
Ind-Bur 1990	A		1.694	3.0354	3.2259		1.3492	2.1847	2.3531	3.656	3.88
	V_B		106.8	191.37	203.38		85.06	137.74	148.35	230.5	244.62
Ind-Bur 1995	A		1.19	1.655	1.4585		1.1297	1.6372	1.4912	3.656	3.88
	V_B	0.455	75.02	104.34	91.95	0.483	71.22	103.22	94.01	230.5	244.62
Ind-Bangla 1997	A	sec	0.7358	1.8038	2.3375	sec	0.7601	1.8639	2.4151	3.656	3.88
	V_B		46.39	113.72	147.37		47.92	117.51	152.26	230.5	244.62
Nagaland	A		0.0093	-	-		0.0085	-	-	3.656	3.88
	V_B		0.59	-	-		0.54	-	-	230.5	244.62
Manipur	A		0.0601	0.0783	0.076		0.0582	0.0772	0.0764	3.656	3.88
	V_B		3.79	4.94	4.79		3.67	4.87	4.82	230.5	244.62
Assam	A		0.0514	0.1021	0.0873		0.047	0.0951	0.081	3.656	3.88
	V_B		3.24	6.44	5.5		2.96	6.0	5.11	230.5	244.62
Phek	A		0.0156	-	-		0.0686	-	-	3.656	3.88
	V_B		0.98	-	-		4.32	-	-	230.5	244.62
Kohima	A		0.2396	0.4194	0.3668		0.047	0.0951	0.081	3.656	3.88
	V_B		15.11	26.44	23.13		13.79	22.9	20.24	230.5	244.62

Table 6.5: Base shear and Spectral acceleration comparison for North region of the country with code based design spectrum

Region		T_n	**Response spectrum**			T_n	**Response spectrum**			**IS: 1893-2002**	
		Program calculated	mean	mean +1 σ	max imum	Empirical equation	mean	mean +1 σ	max imum	static analysis	dynamic analysis
Dharmsala	A	0.455 sec	0.7136	1.4529	1.7061	0.483 sec	0.7122	1.4562	1.7161	3.656	3.88
	V_B		44.99	91.6	107.56		44.9	91.81	108.19	230.5	244.62
Uttarkashi 1991	A		2.6195	5.1532	4.8295		2.4655	4.8884	4.9933	2.437	2.587
	V_B		165.15	324.89	304.48		155.44	308.19	314.81	153.64	163.1
Chamba	A		1.1325	2.43	2.05		1.0894	2.3565	1.9854	3.656	3.88
	V_B		71.4	153.2	129.24		68.68	148.57	125.17	230.5	244.62
Xiz-Ind	A		0.2723	-	-		0.272	-	-	2.437	2.587
	V_B		17.17	-	-		17.15	-	-	153.64	163.1
Chamoli 1999	A		1.9769	3.1418	3.623		1.9	3.3258	3.96	3.656	3.88
	V_B		124.64	198.08	228.42		119.79	209.68	249.66	230.5	244.62
Chamoli 2005	A		0.2491	0.5812	0.4839		0.1788	0.4168	0.3471	3.656	3.88
	V_B		15.7	36.64	30.51		11.27	26.28	21.88	230.5	244.62
Uttarkashi 2007	A		0.0186	0.0247	0.0229		0.0143	0.0158	0.0153	2.437	2.587
	V_B		1.17	1.56	1.44		0.9	1.0	0.96	153.64	163.1
Uttarakhand	A		0.0178	0.0361	0.0388		0.0164	0.0343	0.037	3.656	3.88
	V_B		1.12	2.28	2.45		1.03	2.16	2.33	230.5	244.62

Table 6.6: Base shear and Spectral acceleration comparison for South region of the country with code based design spectrum

Region		T_n	Response spectrum			T_n	Response spectrum			IS: 1893-2002	
		Program calculated	mean	mean +1 σ	max imum	Empirical equation	mean	mean +1 σ	max imum	static analysis	dynamic analysis
Andaman Island 2008	A	0.455 sec	0.6203	-	-	0.483 sec	0.6198	-	-	3.656	3.88
	V_B		39.11	-	-		39.08	-	-	230.5	244.62
Andaman Island 2010	A		0.0839	-	-		0.1055	-	-	3.656	3.88
	V_B		5.29	-	-		6.65	-	-	230.5	244.62

Table 6.7: Base shear and Spectral acceleration comparison for West region of the country with code based design spectrum

Region		T_n	Response spectrum			T_n	Response spectrum			IS: 1893-2002	
		Program calculated	mean	mean +1 σ	max imum	Empirical equation	mean	mean +1 σ	max imum	static analysis	dynamic analysis
Kachchh	A	0.455	1.5756	-	-	0.483	1.8466	-	-	3.656	3.88
	V_B	sec	99.34	-	-	sec	116.42	-	-	230.5	244.62

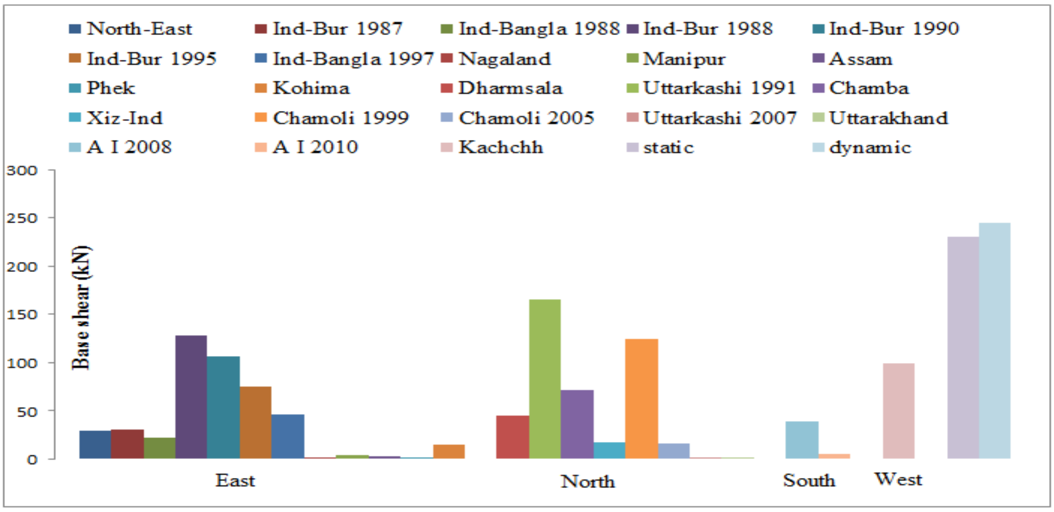


Figure 6.3: Base shear comparison for mean response spectrum (Dynamic analysis)

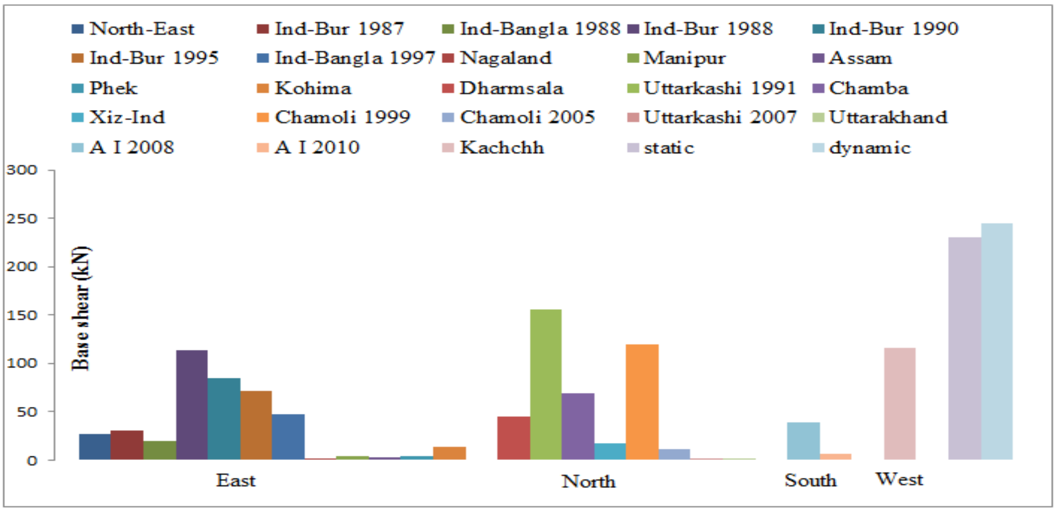


Figure 6.4: Base shear comparison for mean response spectrum (Static analysis)

Observations

- Mean response spectrum is conservative for this structure as compared to code design spectrum for all regions of the country.
- Base shear for south region is very less than obtained from codal provisions.

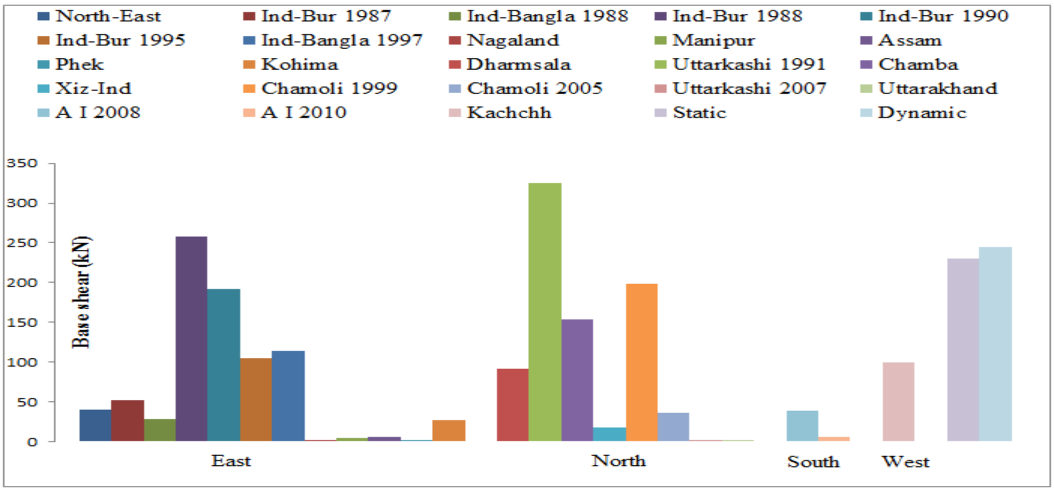


Figure 6.5: Base shear comparison for mean +1 σ response spectrum (Dynamic analysis)

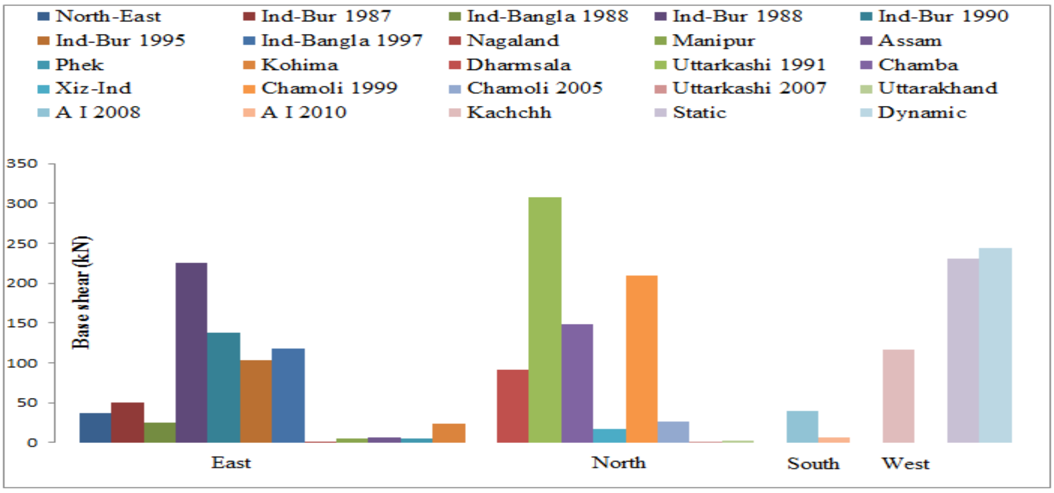


Figure 6.6: Base shear comparison for mean +1 σ response spectrum (Static analysis)

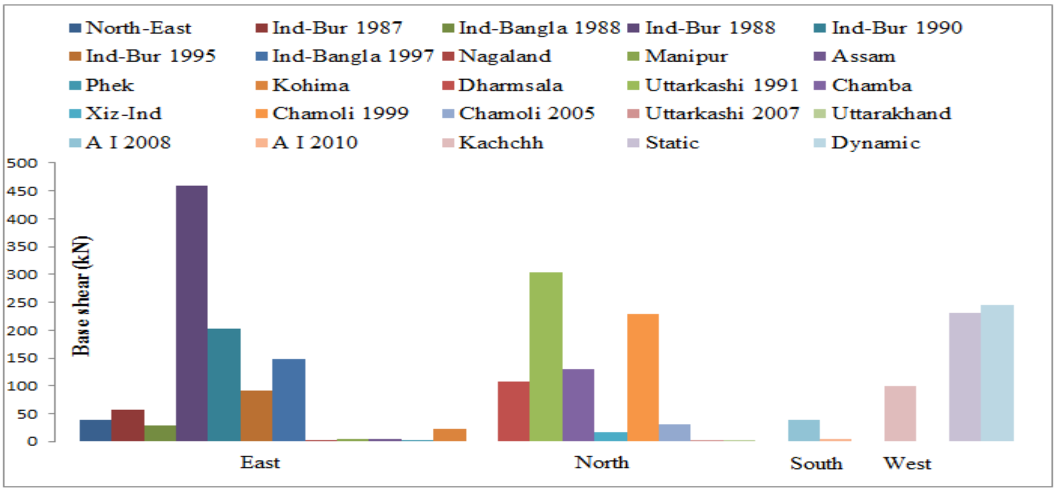


Figure 6.7: Base shear comparison for maximum response spectrum (Dynamic analysis)

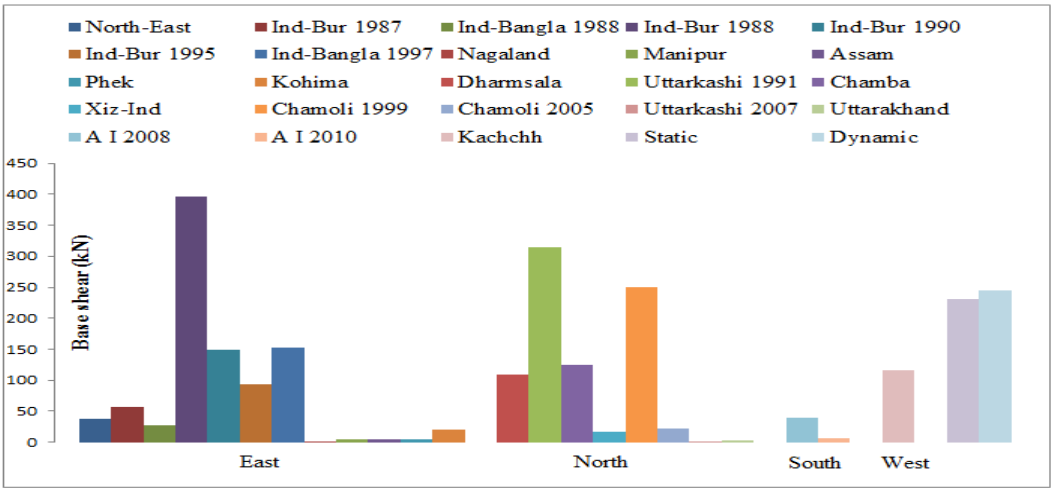


Figure 6.8: Base shear comparison for maximum response spectrum (Static analysis)

6.5 Observations

From the Table 6.4 - 6.7 it is observed that there are variations for spectral acceleration coefficient values in each region of the country. In only some of the cases Sa values of IS:1893 (Part-I)-2002 are governing while in most cases Sa values of proposed response spectrum are governing. From the Figure 6.3 - 6.8, it is observed that base shear is higher most cases by considering representative response spectrum of the country.

- From the Figure 6.3 it is observed that seismic force obtained from mean response spectrum for south region of the country are lower than that required by the code based design spectrum.
- Response from all representative response spectrum for south region are well below code based design spectrum for this building.
- Significant amplification of seismic force is observed for India-Burma 1988 earthquake event for maximum response spectrum.
- Mean plus one standard deviation and maximum response spectrum shows huge increase in response for Uttarkashi 1991 earthquake event.
- Structure is least affected in South region of the country as response is very less.
- Response of a structure is close in agreement with code base design spectrum in West region.
- Response using dynamic and static analysis are nearly same for all strong ground motions.

6.6 Summary

For the present work, Response spectrum method is used to obtain the response of a structure. In this approach, multiple modes of response of a building to an earthquake are taken into account. For each mode, response is read from the representative response spectrum, based on the modal frequency and modal mass. The responses of different modes are combined to provide an estimate of total response of the structure using modal combination (SRSS) method.

Response quantities such as acceleration and base shear are estimated for this building by considering fundamental time period from the representative response spectrum for all recorded earthquake events. Comparison is done for response quantities such as acceleration and seismic force among all regions with response quantity obtained using static and dynamic seismic force.

Chapter 7

Response Spectrum for Multi-Degree of Freedom System

7.1 General

The SDOF approach is not applicable for complex structures such as multilevel framed structure. To predict the response of such a complex structure, the structure is discretized with several members of lumped masses. As the number of lumped masses increases, the number of displacements required to define the displaced positions of all masses also increases. The response of MDOF system is discussed in this chapter. Comparison of response of MDOF system is done between hand calculation results with equivalents obtained from ETABS analysis.

7.2 Equation of Motion for MDOF system

The equation of motion of MDOF system is similar to the SDOF system, but the stiffness k, mass m, and damping c are matrices. The equation of motion for MDOF system under ground excitation is written as

$$m\ddot{u}(t) + c\dot{u}(t) + ku(t) = -m\iota\ddot{u}_g(t) \tag{7.1}$$

where,

$\ddot{u}_g(t)$ is the ground acceleration

m, c, and k are the mass, damping and stiffness matrix respectively.

n is number of degrees of freedom

For building with n degree of freedom, the size of matrix [m], [c], and [k] is $n \times n$.

l is the influence vector of size n $\times$ r.

r is number of components of input ground motion

u is displacement vector which is composed of n lateral storey displacements (degrees of freedom)

$\dot{u}$ is relative velocity vector of size n $\times$ 1

$\ddot{u}$ is absolute acceleration vector of size n $\times$ 1

7.3 Solution of Equation of Motion

MDOF systems are usually analyzed using modal superposition analysis. A typical MDOF system with N degrees of freedom is subjected to ground motion, hence it undergoes deformations in number of possible ways. These deformed shapes are known as modes of vibration or mode shapes. Each shape is vibrating with a particular natural frequency. Total unique modes for each MDOF system are equal to the possible degrees of freedom of system.

The eigen values and eigen vectors of MDOF system are obtained from characteristic equation used in chapter 6.

Let the displacement response of MDOF system expressed as

$$u(t) = [\phi_n] y(t) \tag{7.2}$$

where y(t) represents modal displacement vector and $[\phi_n]$ is mode shape matrix

Substituting Equation 7.2 in Equation 7.1 and pre-multiply by $[\phi_n]^T$, we get

$$[\phi_n]^T[m][\phi_n]\ddot{y}(t) + [\phi_n]^T[c][\phi_n]\dot{y}(t) + [\phi_n]^T[k][\phi_n]y(t) = -[\phi_n]^T[m]\iota\ddot{y}_g(t) \tag{7.3}$$

The above equation is reduced to

$$[M_n]\ddot{y}(t) + [C_n]\dot{y}(t) + [K_n]y(t) = -L_n\ddot{y}_g(t) \tag{7.4}$$

where,
$[\phi_n]^T$[m]$[\phi_n]$=[M] = generalized mass matrix
$[\phi_n]^T$[c]$[\phi_n]$=[C] = generalized damping matrix
$[\phi_n]^T$[k]$[\phi_n]$=[K] = generalized stiffness matrix
L_n=ϕ_n^Tmι

For classically damped system, Equation 7.4 is reduced to following equation

$$\ddot{y}_n(t) + 2\zeta_n\omega_n\dot{y}_n(t) + \omega_n^2 y_n(t) = -\frac{L_n}{M_n}\ddot{y}_g(t) \tag{7.5}$$

Equation 7.5 is valid for all modes, n=1,2,3...N.

where,
y_n(t) is modal displacement response in n^{th} mode,
ζ_n is modal damping ratio in n^{th} mode,
Γ_n is modal participation factor for n^{th} mode and is expressed by

$$\Gamma_n = \frac{{\phi_n}^T[m]\iota}{{\phi_n}^T[m]\phi_n} = -\frac{L_n}{M_n} \tag{7.6}$$

Equation (7.3 - 7.5) describes the modal superposition procedure where the system of N-coupled equations of motion of MDOf system in Equation 7.1 is replaced with N-uncoupled equations of motion of equivalent SDOF systems in Equation 7.5.

From the equation of motion of SDOF system under ground excitation (Equation 3.11), the only difference between Equation 7.5 and Equation 3.11 is the $\frac{L_n}{M_n}$ term applied to the ground excitation. Therefore the solution procedure developed for SDOF systems under earthquake excitation in Chapter 3 is also valid for solving Equation 7.5.

Modal Superposition Analysis Procedure

Modal superposition analysis procedure for solving Equation7.1 is given below.

1. Carry out eigenvalue analysis and determine the modal properties (ϕ_n, ω_n) for n=1,2....N.

2. Construct Equation 7.5 for each mode n.

3. Solve Equation 7.5 by using the method developed for SDOF system (Newmark Beta) in Chapter 3 ($\ddot{u}_g$ is scaled by $\frac{L_n}{M_n}$), and determine y_n(t) for n=1,2...N.

4. Transform from modal to physical coordinates by using Equation 7.2.

The same 4 storey building mentioned in Chapter 6, is used for calculation of response quantity (displacement).

Frequencies and mode-shapes of the structure are presented in Section 6.3.

Modal masses and modal excitation factors are determined.

M_1=ϕ_1^Tmϕ_1=180796.55kg M_2=205080kg M_3=293720kg M_4=803000kg

L_1=ϕ_1^Tm=226023.635kg L_2=-72790kg L_3=41250kg L_4=-28710kg

$\frac{L_1}{M_1}$=1.23 $\frac{L_2}{M_2}$=-0.355 $\frac{L_3}{M_3}$=0.1404 $\frac{L_4}{M_4}$=-0.0357

The response in each mode of vibration is computed by solving the Equation 7.5 for the system subjected to El Centro ground excitation through MATLAB.

$$q_1{=}\frac{L_1}{M_1}S_d{=}1.25{\times}0.04248{=}0.0531\text{m}\ q_2{=}0.0017\text{m}\ q_3{=}0.0002785\text{m}\ q_4{=}0.0000384\text{m}.$$

Transform from modal to physical cordinates.

$$u_1{=}\phi_1 q_1{=}\begin{bmatrix} 0.019063 \\ 0.03563 \\ 0.0475 \\ 0.0531 \end{bmatrix}\text{m}\ u_2{=}\begin{bmatrix} -0.0016 \\ -0.00148 \\ 2.35\times10^{-4} \\ 0.0017 \end{bmatrix}\text{m}\ u_3{=}\begin{bmatrix} 3.4\times10^{-4} \\ -1.55\times10^{-4} \\ -2.7\times10^{-4} \\ 2.78\times10^{-4} \end{bmatrix}\text{m}$$

$$u_4{=}\begin{bmatrix} -4.76\times10^{-5} \\ 7.53\times10^{-5} \\ -7.2\times10^{-5} \\ 3.84\times10^{-5} \end{bmatrix}\text{m}$$

Now the modal displacements are combined with the SRSS rule for obtaining the maximum storey displacement.

u_1=0.01914m, u_2=0.0367m, u_3=0.0475m u_4=0.05313m

The same building is modelled in E-TABS. El Centro time history is given as an input parameter to determine the response at each floor. The mode superposition method of response analysis is used by E-TABS to solve the dynamic equation of motion for the structure.

Table 7.1 shows comparison of displacement response between hand calculation results and with E-TABS analysis.

Table 7.1: Displacement response calculation

T_n=0.4547s	u_1m	u_2m	u_3m	u_4m
Mode superposition	0.01914	0.0367	0.0475	0.05313
E-TABS	0.01949	0.04623	0.06652	0.07992

7.4 Study of Response spectrum generation

Response spectrum is generated by calculating peak response of all SDOF systems subjected to ground motion. For MDOF systems subjected to ground excitation, it undergoes deformations in number of possible ways. These deformed shapes are mode shapes and each shape is vibrating with a particular natural frequency. Peak response is obtained at each storey by mode superposition analysis and response spectrum is generated for peak response of all storey.

Consider several four storey building systems having different time period. All systems are subjected to El Centro earthquake excitation. Peak deformation at each storey is obtained through E-TABS and is presented in Table 7.2.

Table 7.2: Displacement response at each storey

Time period	**u1** 1^{st} **floor**	**u2** 2^{nd} **floor**	**u3** 3^{rd} **floor**	**u4 roof**
0.05	0.00235	0.007897	0.015	0.02246
0.1	0.00591	0.01896	0.03422	0.04885
0.15	0.01049	0.0321	0.055	0.07465
0.2	0.01388	0.03941	0.06541	0.08542
0.3	0.01714	0.04648	0.07284	0.09042
0.5	0.01975	0.046	0.06814	0.08023
1.0	0.03485	0.06715	0.09664	0.1175
1.3	0.03666	0.07273	0.09974	0.1172
1.6	0.03962	0.08249	0.12	0.142
1.9	0.05279	0.09626	0.1271	0.1472
2.2	0.07052	0.1283	0.1561	0.1707
2.5	0.1055	0.1889	0.2348	0.2477
2.8	0.1317	0.2388	0.3058	0.3548
3.1	0.1381	0.2544	0.3269	0.3753
3.4	0.1329	0.235	0.2972	0.351
3.7	0.1256	0.2183	0.3073	0.3723
4.0	0.1309	0.2288	0.3141	0.3722

Response spectrum is generated by plotting peak displacement response for all systems vs natural time period as shown in Figure 7.1.

Building mode shape coefficients obtained from E-TABS are shown in Table 7.4.

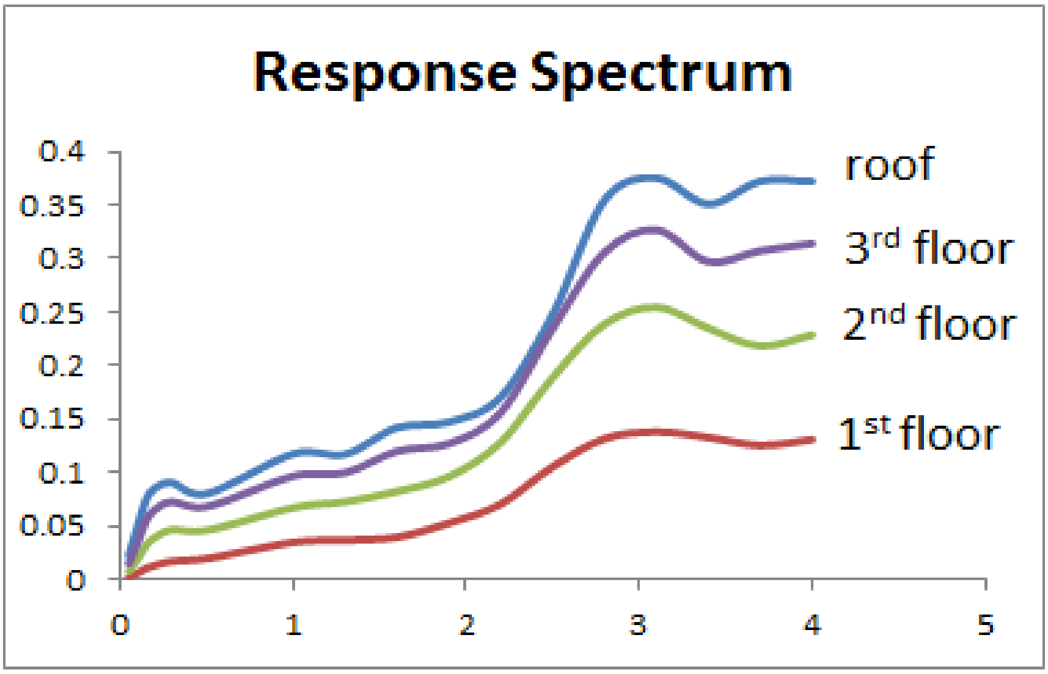


Figure 7.1: Response spectrum subjected to El Centro ground motion for all storey

Mode shapes and Displacement are then normalized to 1 for top storey. Considering two stiff systems (Tn=0.3s, Tn=0.5s) and two flexible systems (Tn=1.3s, Tn=1.6s), building mode shapes and peak displacement are normalized to 1 for top storey. It is found that building mode shape coefficient and peak displacement are almost identical for all storey as shown in Table 7.3.

Table 7.3: Building mode shapes and Peak displacement normalized to 1

Storey	**Normalized Building mode shapes**				**Normalized Peak displacement**			
	Time period				**Time period**			
	0.3s	**0.5s**	**1.3s**	**1.6s**	**0.3s**	**0.5s**	**1.3s**	**1.6s**
4^{th} floor	1	1	1	1	1	1	1	1
3^{rd} floor	0.816	0.845	0.864	0.867	0.806	0.849	0.851	0.845
2^{nd} floor	0.53	0.58	0.614	0.62	0.514	0.573	0.62	0.581
1^{st} floor	0.2	0.241	0.285	0.29	0.19	0.246	0.313	0.28

Table 7.4: Building mode shapes for several systems

Time period	ϕ_1 1^{st} **floor**	ϕ_2 2^{nd} **floor**	ϕ_3 3^{rd} **floor**	ϕ_4 **roof**
0.05	0.0085	0.0285	0.0543	0.0816
0.1	0.0103	0.0332	0.0601	0.0858
0.15	0.0123	0.0372	0.0635	0.0858
0.2	0.014	0.0403	0.0658	0.085
0.3	0.0167	0.0444	0.0683	0.0837
0.5	0.02	0.048	0.0702	0.0828
1.0	0.0218	0.0498	0.0716	0.0836
1.3	0.0237	0.0511	0.0719	0.0832
1.6	0.0242	0.0515	0.0722	0.0833
1.9	0.0245	0.0517	0.0723	0.0834
2.2	0.0248	0.0519	0.0724	0.0834
2.5	0.025	0.0521	0.0725	0.0835
2.8	0.0252	0.0522	0.0726	0.0835
3.1	0.0254	0.0523	0.0726	0.0835
3.4	0.0255	0.0525	0.0727	0.0836
3.7	0.0257	0.0525	0.0727	0.0836
4.0	0.0258	0.0526	0.0728	0.0836

It is observed that response quantity (Displacement) is related with the mode shapes of the system directly. Hence, Response spectrum for MDOF system can be developed for each mass level from relationship of mode shape and displacement.

7.5 Summary

The chapter deals with the generation of Response Spectrum for MDOF system subjected to El Centro ground excitation. Mode superposition analysis procedure is used to solve equation of motion for MDOF system. Peak displacement response is obtained for several systems and displacement response spectrum is generated.

Chapter 8

Summary and Conclusions

8.1 Summary

This thesis is an attempt to understand the concept of Response spectrum and its application in seismic analysis and design of structures. The main objective of the work is to generate response spectrum for various regions of Indian subcontinent and compare it with design spectrum given in code. Numerical algorithm (Newmark Beta method) is used for solving equation of motion of SDOF system and response quantities such as displacement, velocity and acceleration are obtained. Based on this method, MATLAB Code is generated for development of response spectrum.

From different regions of the country, 184 earthquake ground motions from 23 recording stations of India are collected. As all ground motions are not devastating, parameters are defined for selecting strong ground motions. Duration, RMS acceleration and PGA parameters are used for qualifying earthquake ground motions to strong ground motions. 67 strong ground motions are qualified and Response spectrum for all 67 strong ground motions are generated. Plots are developed for 3000 SDOF systems having damping ratio 5%. Statistical approach is carried out to generate smooth representative response spectrum. Three representative response spectrum mean, mean plus one standard deviation and maximum response spectrum are developed. These representative response spectrum are compared with code based design spectrum for all regions of the country.

The effect of response spectrum on design of four storey RC framed structure is considered. Dynamic analysis of this building is carried out considering response spectrum method of IS: 1893 (Part-I)- 2002. Response quantities such as spectral acceleration and seismic force is estimated for this building from proposed response spectrum for different regions of the country. These response quantities are compared with design response spectrum of IS: 1893 (Part-I)-2002. Response Spectrum is also generated for Multi Degree of Freedom (MDOF) system subjected to earthquake ground excitation. It is found that, single response spectrum is enough to estimate lateral force on the building system since response spectrum generated for each mass level of MDOF system is related to their dynamic mode shapes.

8.2 Conclusions

Based on the work carried out following conclusions are made.
In order to understand the characteristics of response spectrum, following points are kept in mind.

- Response spectrum is very useful in design because the designer can easily assess how structures of different natural periods will respond to a specific earthquake.
- Amount of damping directly influence the response of structure.
- Absolute acceleration and pseudo-acceleration are identical for zero damping and differ in small amount for rest of damping curves.

Based on the response spectrum generated for Indian subcontinent, following points are concluded.

- Based on the statistical studies of a number earthquake records, it is shown that mean plus one standard deviation response spectrum derived for all regions is good to represent strong ground motions.

- Negligible difference in response is observed for Mean +1 s and Maximum response spectrum for all regions of the country.
- It is difficult to predict the deficient and conservative part of the design spectrum for south and north region (zone 4) given in IS:1893-2002.
- Based on limited number of records, it is observed that G+3 storey building is least affected in south region of the country.

8.3 Future Scope of the Work

The present work can be extended as follows.

- Parameters used for selecting strong ground motions can be increased.
- Exact Design Spectrum can be developed as per the approach proposed by Newmark and Hall.
- Collecting artificial ground motion records from Indian Seismological Research Center (ISR) and Indian Meteorological Department (IMD), Response Spectrum can be compared.

Appendix A

MATLAB Code

A) MATLAB Code for Response spectrum construction using Newmark-Beta Method

% Development of Response Spectrum for El Centro earthquake excitation using Newmark-Beta method clear,clc;
close all
m=14149.337; % mass of SDOF system
zeta=0.05; % damping ratio
fid=fopen('.txt file of El Centro Acceleration Data');
acc = fscanf(fid,'%g');
acc=[0 ; acc];
acc=acc.*9.81; % in m/sec^2
pga=max(abs(acc)) % peak ground acceleration
p0=-m*acc;
% integration parameter for constant acceleration method
beta0=1/4;
gamma0=0.5;
dt=0.02; % increment in time
u0=0; % initial displacement
v0=0; % initial velocity

```
for Tn=0:0.001:3 % 3000 possible SDOF systems
k(i)=m*(2*pi/Tn)^2; % stiffness of system
c(i)=zeta*4*m*pi/Tn; % damping of system
a0=inv(m)*(p0(1,:)-c(i)*v0'-k(i)*u0);
k1=k(i)+(gamma0/(beta0*dt)*c(i))+(1/(beta0*dt*dt))*m;
aa(i)=(1/(beta0*dt)*m)+(gamma0/beta0)*c(i);
bb(i)=(1/(2*beta0)*m)+dt*(gamma0/(2*beta0)-1)*c(i);
u(1,:)=u0;
v(1,:)=v0;
a(1,:)=a0(i);
for j=2:1560 % number of data points
dp(j-1,:)=p0(j,:)-p0(j-1,:)+aa(i)*v(j-1,:)+a(j-1,:)*bb(i);
du(j-1,:)=dp(j-1,:)/k1(i);
dv(j-1,:)=(gamma0/(beta0*dt)*du(j-1,:))-(gamma0/beta0)*v(j-1,:)+
dt*(1-gamma0/(2*beta0)))*a(j-1,:);
da(j-1,:)=(1/(beta0*dt^2))*du(j-1,:)-(1/(beta0*dt))*v(j-1,:)-(1/(2*beta0))*a(j-1,:);
u(j,:)=u(j-1,:)+du(j-1,:);
v(j,:)=v(j-1,:)+dv(j-1,:);
a(j,:)=a(j-1,:)+da(j-1,:);
end
u1(i)=max(abs(u)); % relative displacement
v1(i)=max(abs(v)); % relative velocity
a1(i)=max(abs(a)); % relative acceleration
V(i)=((2*pi)/Tn)*u1(i); % pseudo-velocity
A(i) = (((2*pi)/Tn)^2)*u1(i); % pseudo-acceleration
i=i+1;
end
u1
v1
a1
```

```
V
A
Tn=0:0.001:3;
subplot(3,1,1)
plot(Tn,u1,'g');
xlabel('Time (sec)','Fontsize',8);
ylabel('Disp.(m)','Fontsize',8);
title('Deformation response spectrum','Fontsize',8);
subplot(3,1,2)
plot(Tn,V,'r');
xlabel('Time (sec)','Fontsize',8);
ylabel('V (m/sec)','Fontsize',8);
title(' Pseudo-velocity response spectrum','Fontsize',8);
subplot(3,1,3)
plot(Tn,A,'b');
xlabel('Time (sec)','Fontsize',8);
ylabel('A(m/sec^2)','Fontsize',8);
title('Pseudo-acceleration response spectrum','Fontsize',8);
```

B) MATLAB Code for RMS acceleration calculation

```
% RMS acceleration calculation of Port blair ground motion using Simpson's 1/3rd rule
clear,clc
fid=fopen('.txt file of Port blair 2010 Acceleration Data');
acc = fscanf(fid,'%g');
l=length(acc); % acceleration data points
a = acc^2;
i=1;
```

```
j=1;
k=2;
for i=1:l/2
odd(i)=a(k);
even(i)=a(j);
k=k+2;
j=j+2;
end
oddterms=4*sum(odd)
eventerms=2*(sum(even)-a(1))
firstlast=a(1)+a(l)
t=0:0.02:181.435 % recorded time (sec)
n=length(t)
h=(t(n)-t(1))/(n-1)
sim=(h/3)*(oddterms+eventerms+firstlast) % Simpson's 1/3rd rule
arms=sqrt(sim/t(n))
arms
```

C) MATLAB Code for Strong Motion Duration calculation

```
% Strong motion duration calculation for Port blair 2010 ground motion
clear,clc
fid=fopen('.txt file of Port blair 2010 Acceleration Data');
acc = fscanf(fid,'%g');
acceleration=0.01*acc;
t=linspace(0.005,181.435,36288);
plot(t,acceleration) % port blair 2010 time history
pga=max(abs(acceleration));
l=length(acceleration);
b=abs(acceleration);
```

```
for i=1:l
if b(i)>0.00496 % rms value
a(i)=1;
end
end
datapoints=find(a)
points=length(datapoints)
firstp=datapoints(1)
lastp=datapoints(points)
timefirst=t(firstp)
timelast=t(lastp)
duration=(timelast-timefirst)
timefirst
timelast
duration
```

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