Quantification of Higher Mode Effects in the Seismic Response of Two-Dimensional (2D) Reinforced Concrete (RC) Frames Including the Effect of Soil

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Signature of Student

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Abstract

Soil-Structure Interaction (SSI) can significantly influence the dynamic characteristics of the structures and in turn affects its seismic performance. In conventional modeling practice, mostly structures have been analyzed considering the fixed based supports and neglecting the effects of soil-structure interaction (SSI) and the foundation flexibility. Higher mode effects become predominantly significant with increasing height of the building. Accordingly higher modes contribute more significantly to the response of the structure as the height of the structure is increased. The present study focuses on quantification of higher-mode effects in the seismic response of two-dimensional (2D) reinforced concrete (RC) frames located on soil Type D in Pakistan. For this purpose, six two-dimensional (2D) generic RC frames with stories number ranging from three to thirty have been taken and then analyzed for seven different ground motions. These ground motion records are accessed using the NGA-West2 coast of (PEER ground motion database). These time histories of ground motions are adjusted and scaled by spectral matching in "SeismoMatch" software to match with target response spectra, which is developed for a 5% damped design base earthquake (DBE) level for the region of Islamabad capital territory (ICT). These analyses have been done with two different conditions i.e., with taking the effect of soil (flexible base) and without taking the effect of soil (fixed base). For this purpose, a general finite element analysis program named as Response 2D has been developed for the analysis of two-dimensional frames. The main advantage of this program is that the user can also see each mode response and its contribution in the combined response for modal time history analysis which helps in investigating the higher modes effects in different responses in a much better way and with better understanding. Modal properties such as time periods, mode shapes and responses like roof displacement history, story shear, story moment etc. have been compared thoroughly for the two cases of fixed and flexible supports. Along with this, top four modes contribution in the combined response has also been studied for the two cases. It has been observed that the phenomenon of soil-structure interaction (SSI) has significantly affected the higher mode effects in responses like base moment, base shear, story moment, story shear and roof displacement.

Keywords: Higher Mode Effects, Soil-Structure Interaction, Generic Frames, Structural Engineering Software Development, Seismic Response.

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List of Abbreviations

ASCE	American Society of Civil Engineers
BCP	Building Code of Pakistan
CAD	Computer Aided Design
DBE	Design Bases Earthquake (10*% PE in 50 years)*
ELF	Equivalent Lateral Force
FEA	Finite Element Analysis
FEM	Finite Element Method
FEMA	Federal Emergency Management Agency
IBC	International Building Code
ICT	Islamabad Capital Territory
LTHA	Linear Time History Analysis
MCE	Maximum Credible Earthquake (2*% PE in 50 years)*
MDOF	Multi Degree of Freedom
MSZ	Makran Subduction Zone
MTHA	Modal Time History Analysis
NGA	Next Generation Attenuation (*Relationships*)
NIST	National Institute of Standards and Technology
OOP	Object Oriented Programming
PE	Probability of Exceedance
PGA	Peak Ground Acceleration
RC	Reinforced Concrete
RSA	Response Spectrum Analysis
SDOF	Single Degree of Freedom
SLE	Service Level Earthquake *(69% PE in 50 years)*
SRSS	Square Root of Sum of Squares
SSI	Soil-Structure Interaction
UBC	Universal Building Code

List of Notations/Symbols

L	Length of Footing
B	Width of Footing
D	Depth of Footing
L_{col}	Length of Column
B_{col}	Width of Column
G	Shear Modulus of Soil
ν	Poisson’s Ratio
η	Embedment Correction Factor
SBC	Soil Bearing Capacity
fck	Concrete Compressive Strength

Chapter 1

Introduction

1.1 Background

Pakistan lies on the seismic region of three major tectonic plates i.e., Indian, Eurasian, and Arabian. Moreover, Pakistan is in the list of top fifty seismically active countries and in Asia, it is one of the most active seismic areas. In the north of Pakistan, there is great Himalaya which has been formed due to convergence of plate boundaries between Eurasian and Indian plates and in the south, there is subduction zone known as Makran subduction zone (MSZ) due to the subduction of Arabian plate under Eurasian plate. Seismic hazard has become a notable problem for rapid growth of industries, urbanization, infrastructure, and population due to the high rate of seismic activities in Pakistan. In the past, the country has been hit by many earthquakes including the earthquake of Kashmir back in 2005 resulting into a large number of casualties.

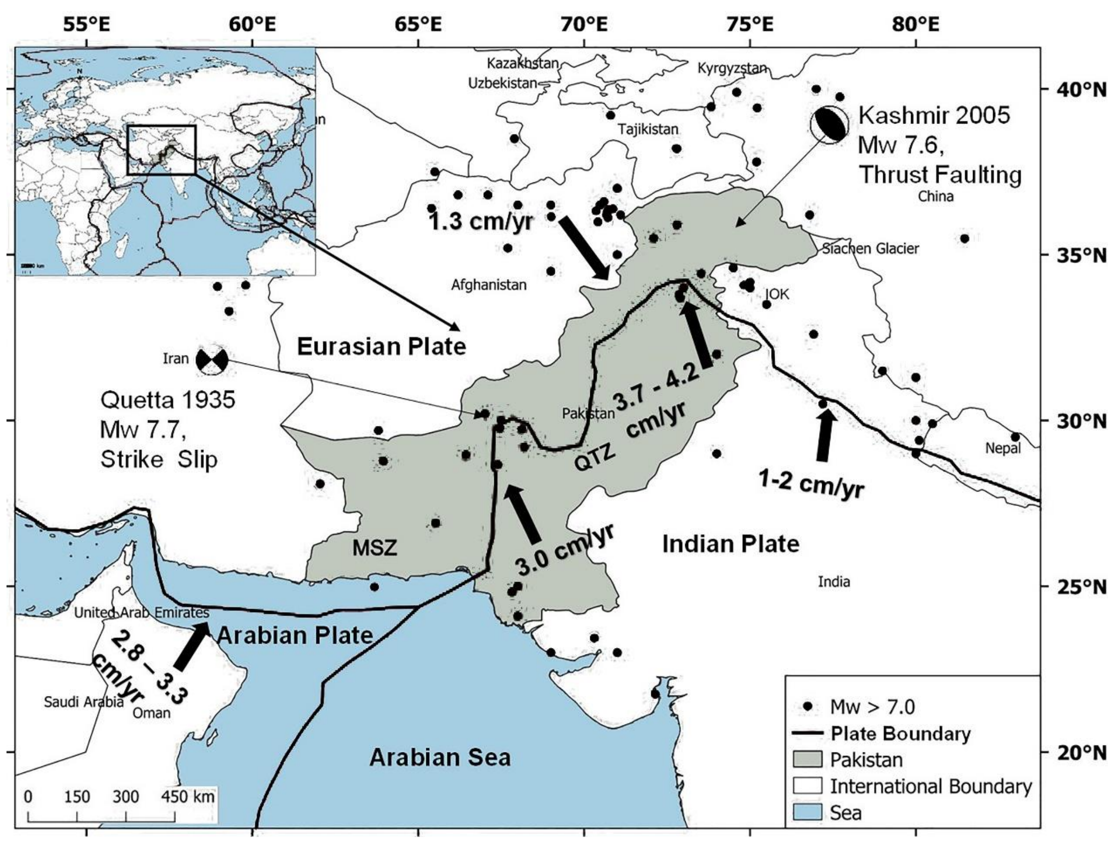


Figure 1-1: Seismo-Tectonic Setting of Pakistan

Table 1-1 Earthquake Events History Detail in Pakistan

Date	Mag. (M_w)	Location	Deaths	Source
24/06/2022	4.2	Khyber Pakhtunkhwa	-	Wikipedia
06/05/2022	5.2	Khuzdar, Balochistan	1	Wikipedia
07/10/2021	5.9	Harnai, Balochistan	27	Wikipedia
06/10/2019	3.6	Mirpur, Azad Kashmir	1	(USGS 2019)
24/09/2019	5.6	Mirpur, Azad Kashmir	38	(USGS 2019)
25/12/2015	6.3	Gilgit-Baltistan	4	-do-
26/10/2015	7.5	Badakhshan, Afghanistan	399	-do-
28/09/2013	6.8	Awaran, Balochistan	400	-do-
24/09/2013	7.4	Awaran, Balochistan	825	-do-
18/01/2011	7.2	Dalbandin, Balochistan	3	-do-
29/10/2008	6.4	Ziarat, Balochistan	215	-do-
08/09/2005	7.6	Balakot, Azad Kashmir	73000	(Durrani 2005)
27/02/1997	7	Balochistan region	57	(ISC 2019)
28/12/1974	6.2	Khyber Pukhtunkhwa	5300	(Utsu 2002)
28/11/1945	8.2	Makran, Balochistan	300-600	-do-
31/05/1935	7.7	Ali jaan, Balochistan	30000-60000	(Bangash 2011)

In Pakistan, most of the times practicing structural engineering ignore the effect of soil while designing the structures. This approach can be conservative and time saving in some design cases however it can never be accepted as a rule. It is well established that the taking the effect of soil affects the modal properties of the structure and in turn modifies the dynamic response. The difference in responses as compared to the responses from fixed base is sometimes of greater magnitude and must be taken into consideration while designing the structure. The term SSI (soil-structure interaction) is used mostly to describe this effect in the literature and this phenomenon is more pronounced in low rise buildings and tends to decrease as we move from low to high rise buildings.

Apart from soil-structure interaction, higher mode effects is another phenomenon which needs much attention while designing the structure. Code based static design approaches which are mostly based on the fundamental mode of vibration are unable to handle the effect of higher modes. When structure is subjected to some ground motion, the response from all the modes should be used to calculate the exact response. However, it has been well established that in low rise buildings, the actual response can be calculated from the first few modes and the contribution of other modes are nearly negligible. But as the height of the building is increased, higher modes participation in the overall response starts increasing and one cannot rely on the code-based approaches for the design process of the structure. Moreover, soil also affects the higher modes effects since it decreases the global stiffness of the structure and accordingly quantification of higher modes is required along with the effect of soil.

1.2 Problem Statement

Buildings are designed with fixed bases while neglecting the effect of soil which can be conservative and time saving in some cases, however it cannot be true for all the cases. Moreover, most of practicing design engineers rely on code based static approaches which are mostly based on the fundamental mode of vibration for the design of new structures and analysis of existing ones. Code based methods are applicable to first mode dominating structure but with increasing height of the building, they become no longer applicable since higher modes tend to participate more dominantly in the response. Accordingly, higher mode effects need to be quantified for various heights of building ranging from low to high rise along with taking the effect of soil. Another problem is that there is no previous computational tool for quantifying the higher modes effects with soil modeling since previous tools display only the combined response and not the modes response.

1.3 Research Objectives

- To evaluate the higher mode effects in the dynamic response of RC frame buildings in Pakistan.
- To compare the modal response participations of RC buildings with fixed base and with the modeling of soil.
- To develop a general-purpose FEA program to perform seismic analysis of 2D frame buildings with soil modeling.

1.4 Methodology

Extensive literature review has been conducted to study the soil-structure interaction phenomenon and the higher mode effects and the effect of soil-structure interaction on higher mode effects. After the literature review, a work plan has been prepared to achieve the research objectives. Most building stock of Pakistan comprises of reinforced concrete (RC) buildings and hence these have been chosen to study the effect of soil on higher mode effects. For achieving the desired objectives, there is a need of computational tool to model the soil and then study the higher modes contribution in various responses. However, there is no previous computation tool available for the modes results and accordingly a general-purpose FEA program named as Response 2D has been developed which has also the additional feature of showing each mode response and its contribution percentage in the combined response. Six two-dimensional generic frames have been taken whose configurations and sections correspond to that of real buildings. These frames have been subjected to seven ground motions which have accessed using NGA-West2 coast of PEER ground motion database and have been scaled and adjusted in "SeismoMatch" software by spectral matching to match with the target spectra developed for a 5% damped design base earthquake (DBE) level for the region of Islamabad capital territory (ICT) .Soil type D which is a stiff and more common soil in Islamabad has been used for capturing the soil-structure interaction effects. Then the modal time history analysis has been performed in Response 2D with fixed and flexible base for all the seven ground motions. The higher modes effects have been compared for the two cases of fixed and flexible bases to quantify the higher modes effect in reinforced concrete (RC) frames along with effect of soil.

1.5 Scope of Research

This research work mainly focuses on the quantification of higher modes effects with including the soil-structure interaction for reinforced concrete (RC) two-dimensional (2D) frames located on soil type D in Pakistan. Heights of frames have been ranging from low to high-rise. These reinforced concrete frames have been subjected to seven ground motions accessed from NGA-West2 coast of PEER ground motion database and have been scaled in "SeismoMatch" software to match with the target spectra which has been developed for the region of Islamabad Capital Territory (ICT) for design base earthquake (DBE). Higher modes effects have been studied thoroughly in various responses with the effect of soil and with neglecting the effect of soil.

1.6 Dissertation Organization

Chapter 1: In the first chapter, a brief overview of the seismicity of Pakistan has been discussed. Along with this, a brief introduction of soil structure interaction and higher mode effects has been presented followed by the problem statement, research objectives, methodology and the scope of research.

Chapter 2: The second chapter explains the literature related to the higher mode effects and the soil- structure interaction. Additionally, past research work related to these has also been discussed briefly along with some concepts of structural engineering software development.

Chapter 3: The third chapter is about the methodology adapted to achieve the result objectives. It discusses about the development of Response 2D, selection of generic frames, selection of ground motions and the analysis of these generic frames on Response 2D to quantify the higher modes effects with the effect of soil.

Chapter 4: In this chapter, all the results have been discussed in detail along with the comments on these results.

Chapter 5: Conclusions have been drawn in this chapter followed by the recommendations at the end.

Chapter 2

Literature Review

In this chapter the previous research work related to the higher modes effect and soil structure interaction have been presented. Additionally, a brief overview of soil-structure interaction along with the soil modeling techniques have been also discussed. And at the end, higher modes effects have been described briefly followed by some discussion on structural engineering software development.

2.1 Past Research Work

In 2016, Vivek B [1] has studied the effects of soil on modal properties of a building. He mainly focused on short to midrise buildings resting on ganga sand in India for studying the effects of soil-structure interaction. He ended up with the conclusion that the effect of soil-structure interaction is more prominent in three story frame if compared with the six-story moment frame. Time periods have been increased with the incorporation of soil effect. Also, the displacements have been amplified since the global stiffness has been reduced if compared with the global stiffness of frame with fixed base.

In 2016, Behnoud Gajnavi [2] has studied the influence of higher modes effects on the strength and ductility demands of various MDOF systems and their equivalent SDOF systems along with considering the effect of soil-structure interaction. For this purpose, he has taken more than 6400 linear and non-linear multi degree of freedom (MDOF) systems along with their equivalent single degree of freedom (SDOF) systems. These systems have been subjected to 21 ground motions and have varying number of stories, structure-to-soil stiffness ratio, aspect ratio and inelasticity level. The lateral strength and ductility demand of MDOF soil‐structure systems have been compared to their equivalent-SDOF soil-structure systems. He ended up with the results indicating that the common equivalent-SDOF soil‐structure systems can lead to very un-conservative results for estimating the strength and ductility demands of MDOF structures with taking of effect of soil-structure interaction (SSI). This highlights the significance of higher mode effects for soil‐structure systems as compared to the structures with fixed base, which is more pronounced for elastic and low level of inelastic cases.

In 2013 Charilaos A. Maniatakis [3] has studied the higher mode effects of moment resisting RC frame structures. He focused mainly on the structures with a lateral force resisting system consisting of slender walls for which the higher mode effects are more significant. In Current seismic design practice approaches, the reduction factor has been assumed to be same for all the modes, even though there is strong evidence that higher modes of vibration are affected by inelasticity unequally and in a different manner as that in the elastic range. He measured the accuracy of various response results by comparing Modal Response Spectrum Analysis (MRSA) method and other available methods result with the ones of nonlinear response history analysis results. The results suggested that the story inertial forces and accelerations at all stories and shear forces at higher stories are significantly underestimated by methods based on fundamental mode of vibration, even for first mode dominated structures. Furthermore, he concluded that the contribution of higher modes depends on three main things i.e., ground motion characteristics, the overstrength associated with the mode and the response quantity under examination.

In 2012, **Saad et al.** [4] had taken five, ten, fifteen and twenty-story RC buildings having multiple underground stories. He studied the seismic behavior of these RC buildings for the local seismic conditions of Beirut. The soil was modeled in SAP2000 by multi-linear kinematic plastic link property. Soil class C (very dense soil or soft rock) and soil class D (stiff soil) were considered for the analysis to study the effect of soil on structural responses. Various seismic responses of buildings were studied which are inter-story shears, base shear, and overturning moments. Based on the comparative analysis of results with the fixed base case, he concluded that SSI affects the low-rise buildings very much by increasing story shear and moment demands significantly. Also, he made the conclusion that the effect of soil is more prominent in case of buildings constructed on the soft soils.

In 2010, Tabatabaiefar et al. [5] had taken four reinforced concrete (RC) buildings located in Iran consisting of 3, 5, 7 and 10 stories. Soil class II, soil class III and soil class IV as per Iranian Standards were used to study the SSI effects for this research. He used the direct method of soil modeling as per NIST to perform the analysis in SAP2000. He ended up with the results that the soil structure interaction effects were more prominent for soils having shear wave velocity less than 600 m/s. He also recommended that by considering soil-structure interaction (SSI) effects, safer and economical structures can be designed and built for this category of buildings and soils.

2.2 Brief Overview of Soil-Structure Interaction

According to FEMA P-750 (2009 Edition) [6], three interconnected systems affect the response of a structure when subjected to earthquake shaking: the structure, the foundation, and the soil beneath and surrounding the foundation. The term SSI (Soil-Structure Interaction) has been used mostly to describe this effect in the literature. Seismic SSI evaluates the response of the structure and foundation to a specific ground motion in the free field (NIST) [7]. The term free field motion can be simply defined as the motions which are not disturbed by the structural vibrations.

The effects of Soil-Structure Interaction (SSI) have been divided into two categories. Inertial interaction effects refer to displacements and rotations at the foundation level of the structure resulting from inertial forces such as base shear and moment. Inertial displacements and rotations can be a significant source of flexibility and energy dissipation in the soil structure system. Kinematic interaction effects are the result of the presence of rigid foundation elements on or in the soil, causing the motions in the foundation to deviate from those in the free field.

These two effects are related to the following:

- Foundation stiffness and damping
- Variation between input foundation motions and free field ground motions.
- Foundation Deformations.

There are few more terms which have been used while dealing with the Soil-Structure Interaction.

Rigid base, in which soil stiffness has been assumed as infinite.

Rigid foundation, in which foundation stiffness has been assumed as infinite.

Fixed base, a combination of rigid foundation and rigid base.

Flexible base, in which both the stiffness of soil and foundation are finite, and the deformability has been captured for both.

In some situations, SSI can significantly affect the behavior of buildings during earthquake shaking and in the design forces. Some of their examples are as follows:

- Large floor plan area of the building
- Substantial embedment of foundations
- High ratio of soil and structure stiffness
- Foundation Rocking

The concept of soil effect on the response of structure can be realized by a simple example. Consider a single degree of freedom (SDOF) system with mass m and stiffness k. Now the deflection can be given by the following relation.

$$\Delta = \frac{F}{k} \tag{2.1}$$

The undamped natural vibration frequency, ω, and period, *T*, of the structure are given by Clough and Penzien (1993) as:

$$\omega = \sqrt{\frac{k}{m}}, \frac{2\pi}{T} = \sqrt{\frac{k}{m}}, T = 2\pi\sqrt{\frac{m}{k}} \tag{2.2}$$

By substituting the value of k from equation 2.1, we get

$$T = 2\pi\sqrt{\frac{m\Delta}{F}}, T^2 = 4\pi^2\frac{m\Delta}{F} \tag{2.3}$$

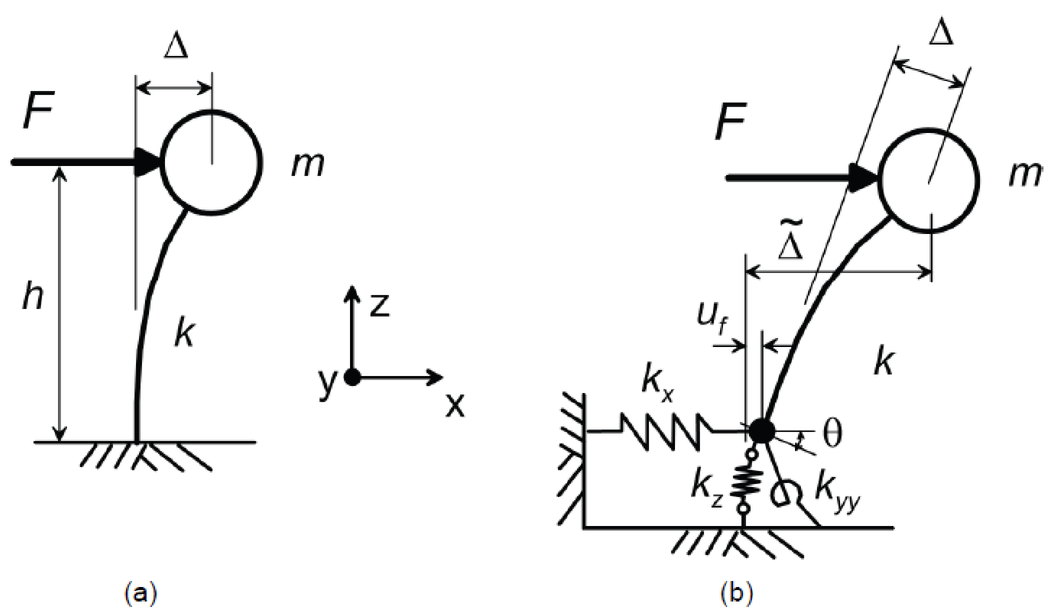


Figure 2-1: Illustration of deflection caused by force applied to: (a) fixed based structure (b) flexible base structure with all three springs

Now consider the same structure with vertical, horizontal, and rotational springs at its base, representing the effects of soil flexibility against a rigid foundation, as shown in Figure 2-1. The vertical, horizontal, and rotational stiffness of the spring have been denoted by k_z, k_x and k_{yy} respectively. When a force F acts on the mass in the x direction, the structure deflects just as in the fixed base system, but the base shear (F) deflects the horizontal spring by amount u_f, and the base moment (Fh) deflects the rotational spring by amount θ. Accordingly, the total deflection has been given by the following formula:

$$\Delta = \frac{F}{k} + u_f + \theta . h \tag{2.4}$$

$$\Delta = \frac{F}{k} + \frac{F}{k_x} + \left(\frac{F.h}{k_{yy}}\right) h \tag{2.5}$$

$$T^2 = 4\pi^2 \frac{m\Delta}{F} \tag{2.6}$$

$$T^2 = 4\pi^2 m \left(\frac{1}{k} + \frac{1}{k_x} + \frac{h^2}{k_{yy}}\right) \tag{2.7}$$

$$\left(\frac{T}{T}\right)^2 = k \left(\frac{1}{k} + \frac{1}{k_x} + \frac{h^2}{k_{yy}}\right) \tag{2.8}$$

$$\frac{T}{T} = \sqrt{1 + \frac{k}{k_x} + \frac{kh^2}{k_{yy}}} \tag{2.9}$$

In Equation 2-7, h is denoted as the centroid height (the height of the center of mass) for the first mode shape. This is commonly known as the effective modal height and is taken as two-thirds of the total height of the structure. It can therefore be stated that with the introduction of soil effects, the time-period has become elongated, and this phenomenon is known as time-period lengthening. The increase in the time-period also affects the mode shapes of the structure and subsequently also modifies the dynamic responses.

2.3 Soil Modeling Techniques

According to FEMA P-2091 (A Practical Guide to Soil-Structure Interaction) [8], there are two common approaches to model the interactions between a structure, its foundation, and the soil that supports it including soil flexibility and damping.

- **Substructure Approach**, where the soil is represented with springs (Fig 2-2). There are approaches to calculate the soil stiffness and the calculated stiffness is assigned to springs to capture the soil effect.
- **Direct Analysis Approach** refers to the approach in which the soil and the structure both have been modeled using finite elements (Fig 2-3). In NIST (National Institute of Standards and Technology) GCR 12-917-21 (Soil-Structure Interaction for Building Structures) the term continuum has been used for finite elements.

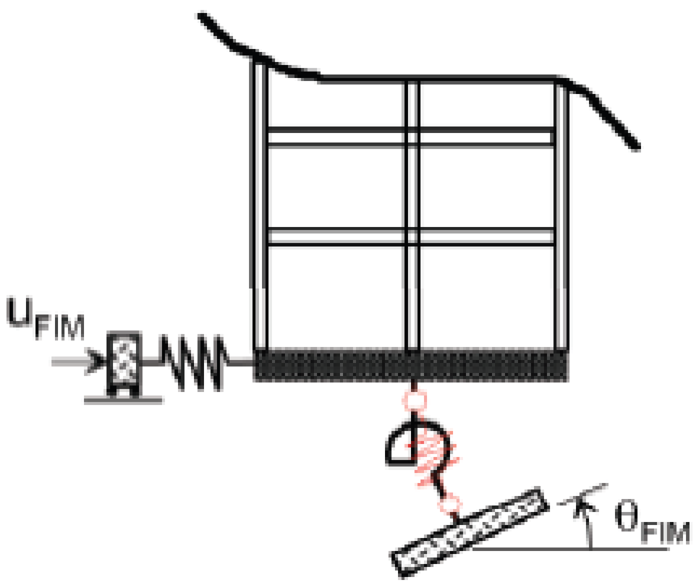


Figure 2-2: Illustration of Sub-Structure Approach

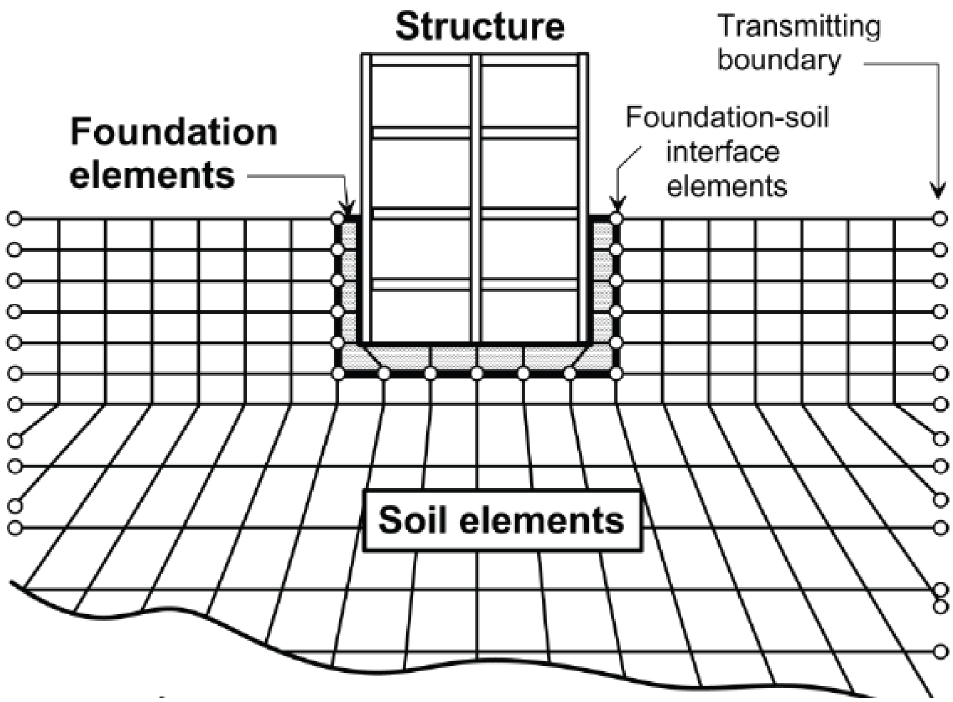


Figure 2-3: Illustration of Direct Analysis Approach

In substructure approach, soil flexibility is generally taken into account in the analytical building model by modeling the connection of structural elements to fixed supports with spring elements. They can be point springs, modeling each degree of freedom on a single spring basis, or they can be distributed springs representing the soil support as a discretized continuous medium, known as a Winkler foundation. (FEMA P-751)

Vertical and rotational springs can model similar behavior. They affect the rocking of the structure at the base due to the elastic vertical compression of the soil. Horizontal springs model foundation displacement relative to free-field soil displacement or soil resistance against basement walls or other vertical surfaces. (FEMA P-751)

The soil stiffness is calculated mostly by two techniques. One developed by Pais and Kausel in 1988 and one by Gazetas in 1991 which is expanded by Mylonakis et al. in 2006. Pais and Kausel equations are mostly used for calculating the soil spring stiffnesses. These equations are as follows:

$$K_{x,sur} = \frac{GB}{2-\nu}\left[6.8\left(\frac{L}{B}\right)^{0.65} + 2.4\right] \quad 2.10$$

$$K_{z,sur} = \frac{GB}{1-\nu}\left[3.1\left(\frac{L}{B}\right)^{0.75} + 1.6\right] \quad 2.11$$

$$K_{yy,sur} = \frac{GB^3}{1-\nu}\left[3.73\left(\frac{L}{B}\right)^{2.4} + 0.27\right] \quad 2.12$$

The reduction of ground motions with depth is referred to as the embedment effects, which are kinematic interaction effects. Pais and Kausel (1988) also provide correction factors to increase the values of stiffness for each degree of freedom to incorporate the embedment effects. These are presented below:

$$\eta_x = \left[1.0 + \left(0.33 + \frac{1.34}{1+L/B}\right)\left(\frac{D}{B}\right)^{0.8}\right] \quad 2.13$$

$$\eta_z = \left[1.0 + \left(0.25 + \frac{0.25}{L/B}\right)\left(\frac{D}{B}\right)^{0.8}\right] \quad 2.14$$

$$\eta_{yy} = \left[1.0 + \frac{D}{B} + \left(\frac{1.6}{0.35+(L/B)^4}\right)\left(\frac{D}{B}\right)^{2}\right] \quad 2.15$$

B=Foundation half width

L=Foundation half length

D=Depth from the ground surface to the bottom of the foundation

G=Shear modulus of soil

ν=Poisson's ratio of soil

2.4 Brief Overview of Higher Mode Effects

In general, low-rise buildings are designed and analyzed by the equilateral lateral force methods (linear static analysis), considering only the fundamental mode of vibration. But as the height of the building increases, the higher modes tend to contribute more to the dynamic response of the building. Thus, code-based approaches or the ELF procedure cannot solve this problem. The number of modes needed in a dynamic analysis depends on the higher mode contribution to the structural response, which is called the higher mode effect [9]. The higher mode effects in the dynamic response contribution are of great importance for the design of new structures and the evaluation of existing ones, because without considering the effect of the higher mode contribution in the response, the results are no longer accurate from the simplified methods that are based on the fundamental mode mostly. According to Anil K. Chopra, if an accurate value of the structural response to earthquake excitation is desired, the response contributions of all natural modes of vibration must be included, but the first few modes can usually provide sufficiently accurate results [10]. Higher mode response can be defined as the combined response due to all modes higher than the first mode. For fixed ρ (column to beam stiffness ratio), higher mode response is more significant for buildings with large time-period, and for fixed T_1, higher mode response is more significant for frames with smaller ρ. As the height of the building increases, the fundamental time-period also increases, and therefore due to the long-period nature of the structures, a significant contribution to the seismic responses for higher vibration modes is expected.

Due to higher-mode vibrations, two main phenomena occur which have challenged the interest of engineers in recent decades. The first phenomenon is the amplification of shear demands due to higher modes effects and is known as shear amplification. The amplification of shear forces has been found to increase with increasing fundamental period and ductility. The second phenomenon shows unexpectedly large accelerations induced by earthquakes observed during the seismic events or evaluated based on the analytical model and is referred to as floor acceleration magnification. The phenomenon of floor acceleration magnification is strongly associated with inertial forces, as the ratio of floor inertial force to its mass is equal to floor acceleration in the lumped mass approach for multi degree of freedom (MDOF) structures; thus, an inaccurate evaluation of story acceleration leads to an inaccurate estimate of story inertial forces. Several indications suggest that many failures and the building collapses during past earthquake ground shakings were caused by large floor accelerations that were not planned for.

The response spectrum (RSA) procedure, which accounts for multimode effects, is commonly used in the seismic design of structures. However, recent studies clearly show that the RSA procedure greatly underestimates the seismic demands along the entire height of the structure [11].One critical assumption in the RSA procedure is that the elastic responses of each vibration mode can be proportionally reduced in inelastic seismic demands by the same response modification factor "R". Thus, modal time-history analysis provides better results compared to the RSA procedure because it is more calculation-based than assumption-based, although it is laborious.

2.5 Limitations in Current FEM Software and the development of Response 2D

There are huge numbers of general-purpose FEM Software which can be used to fulfill structure design and analysis goals. Some of these have been listed below:

- Abaqus
- Advance Design
- Autodesk Revit
- ETABS
- Realsoft 3D
- RFEM
- RISA
- SAP2000
- SketchUp
- STAAD Pro
- Tekla Structures

The above listed software can also be used to model the soil and its effect can be studied on various responses of the structure. However, to study the higher modes effects with the inclusion of soil-structure interaction, there is need to get each mode response in the modal time history analysis method which has been summed to get the combined response. And the current software do not have this feature of showing each mode response. However, there are some indirect methods which can be used to get the response of each mode and its contribution in the combined response. But these are very time taking and effortful methods. Along with this, there is not much development in the field of structural engineering software development in Pakistan and accordingly there is dire need of advancement in this field, and someone needs to step up. Accordingly, a general-purpose FEM program named as Response 2D has been developed with the additional feature of showing each modes response and with better options of automated results to perform the parametric studies faster.

2.6 Selection of Programming Language for the Development of Response 2D

In the engineering world, languages like Python, C++, C#, and VBA (Excel) are a great place to start and have lots of real-life applications both in web and desktop platforms. If someone is interested in online web development, JavaScript is a great code to learn first, with great front-end and back-end features. There are numerous programming languages which can be used to develop a desktop application. The most common of these are Java, MATLAB, python, VB.net etc. However, there are few things which should be kept in mind before selecting the right language, which are as follows:

- Ease to learn
- Open-source libraries
- Ease in developing interfaces, forms etc.
- Readability of syntax
- Language paradigm
- Language community

With respect to beginner friendliness and language community, python and MATLAB are better choices than VB.net and Java. However, developing interfaces and forms creation are much easier in VB.net. Moreover, its syntax is more readable than the others and with the support of .NET framework, there is also no need of compiler to run the program. Accordingly, Vb.net has been chosen for the development of Response 2D to achieve the research objectives.

Chapter 3

Methodology

There are two main phases involved in achieving the research objectives. Their detail can be seen with the help of a flow chart in Fig 3-1.

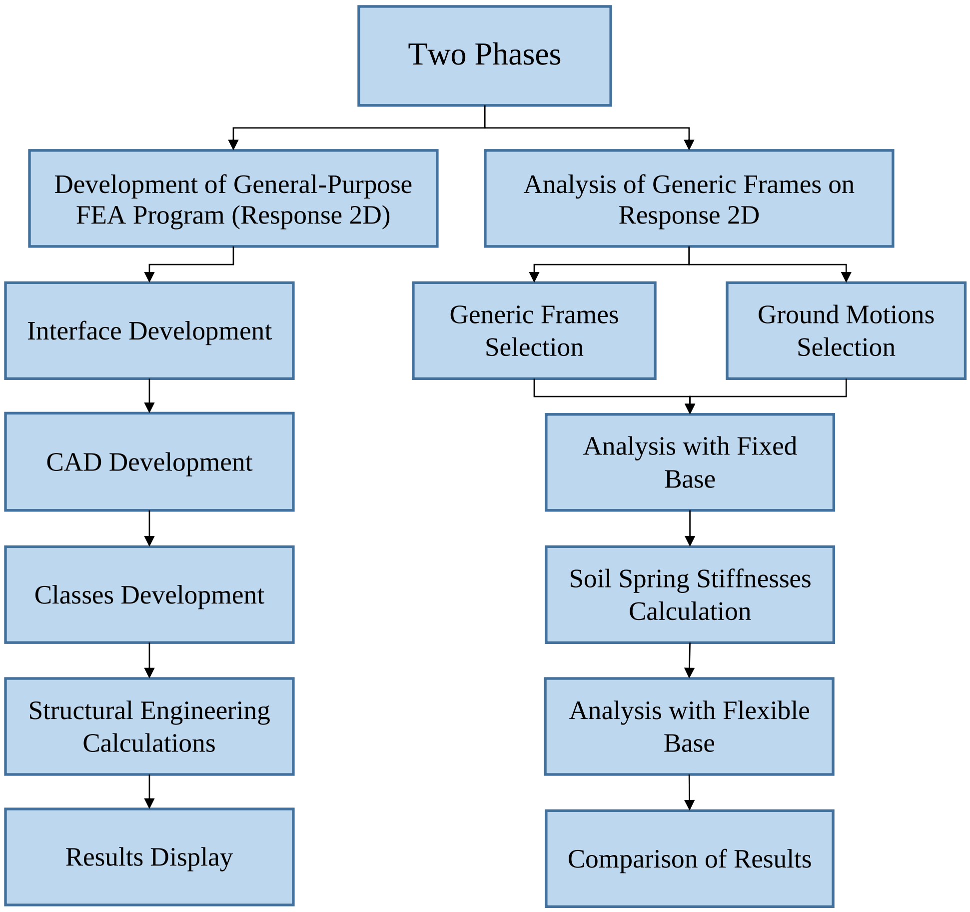


Figure 3-1: Schematic flow chart for methodology

3.1 Development of Response 2D

A general-purpose FEA program named as Response 2D has been developed in object-oriented programming (OOP) paradigm and using the programming language Microsoft VB.net. The main objective of development of Response 2D is that the previous programs do not show each modes response in modal time history analysis and show only the combined response whereas in Response 2D combined response along with the modes response and modes contribution percentage can be seen in the results. The various steps involved in the development of Response 2D have been discussed in detail below.

3.1.1 Interface Development

The interface of Response 2D has been kept very simple and self-explanatory as most of the structural analysis software. There are three tool bars i.e., one left and two top bars. The left tool bar has options of drawing, selecting, and deleting members and restraints can also be assigned from the left bar. The topmost tool bar (known as menu bar in VB.net) has several options like file, edit, define, draw etc. which are self-explanatory. Bottom tool bar has most of the buttons same as that of the topmost tool bar. However, in this toolbar, there are some other buttons too like zoom in, zoom out, zoom fit etc. Apart from the tool bars, there are two tables in the right of interface. In the top table, the user can see the defined materials, sections, load patterns and load cases. However, these can be defined only from the tool bars. In the bottom table, the user can see the defined response spectrum and time history functions. In the extreme right bottom, there is a units dropdown option from where the units can be changed. This option is also available in the edit option in topmost tool bar. In the center of the interface, there is CAD environment where user can interact with the model and make changes to the drawn members or draw new members.

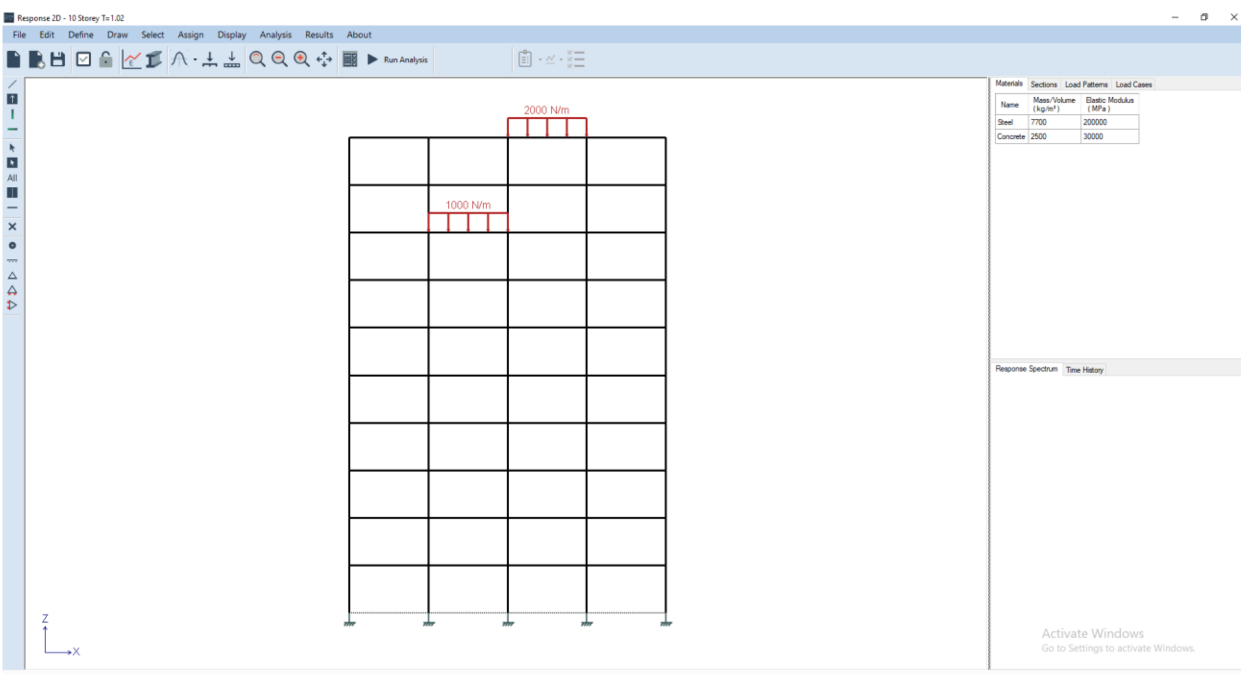


Figure 3-2: Main interface of Response 2D

3.1.2 CAD Development

CAD or Computer Aided Design simply refers to the modeling environment in which the user can interact with the drawing to modify or create it. For Response 2D, after the development of user interface, the most important step is to create such environment for the drawing and modification of 2D frames. For this purpose, grids have been used on which the user can draw members. The user can only interact with the grids and drawn members on the modeling environment. So, the very first step is to define the grids by providing the story numbers, bays numbers and their dimensions as seen in the Fig 3-3.

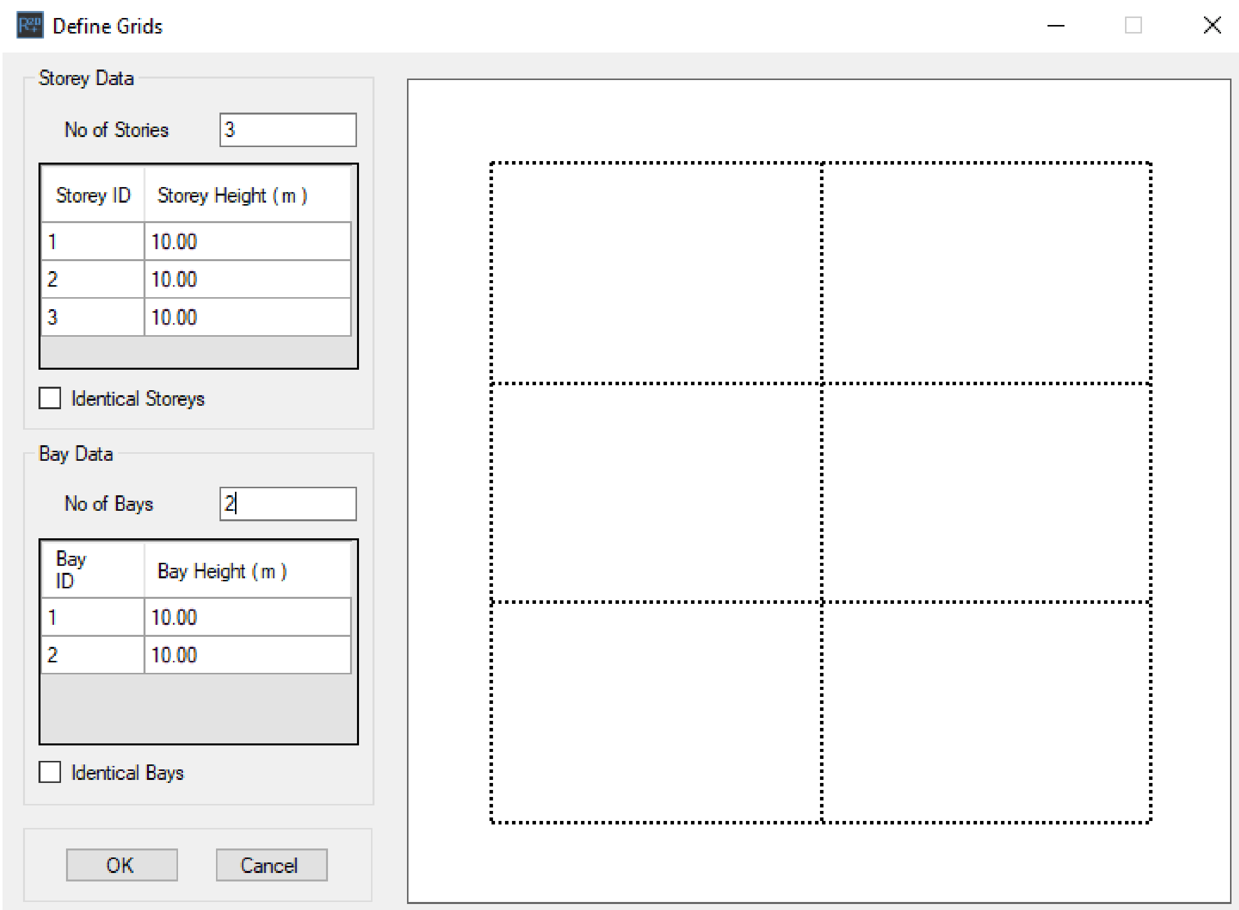


Figure 3-3: Grids form in Response 2D

After defining grids, these can be seen in the main interface modeling environment and then the user can draw members or perform the other actions like assigning sections to members, deleting members, assigning restraints to joints etc. No third-party libraries have been used in the development of this modeling environment. All codes have been written from scratch for the CAD development.

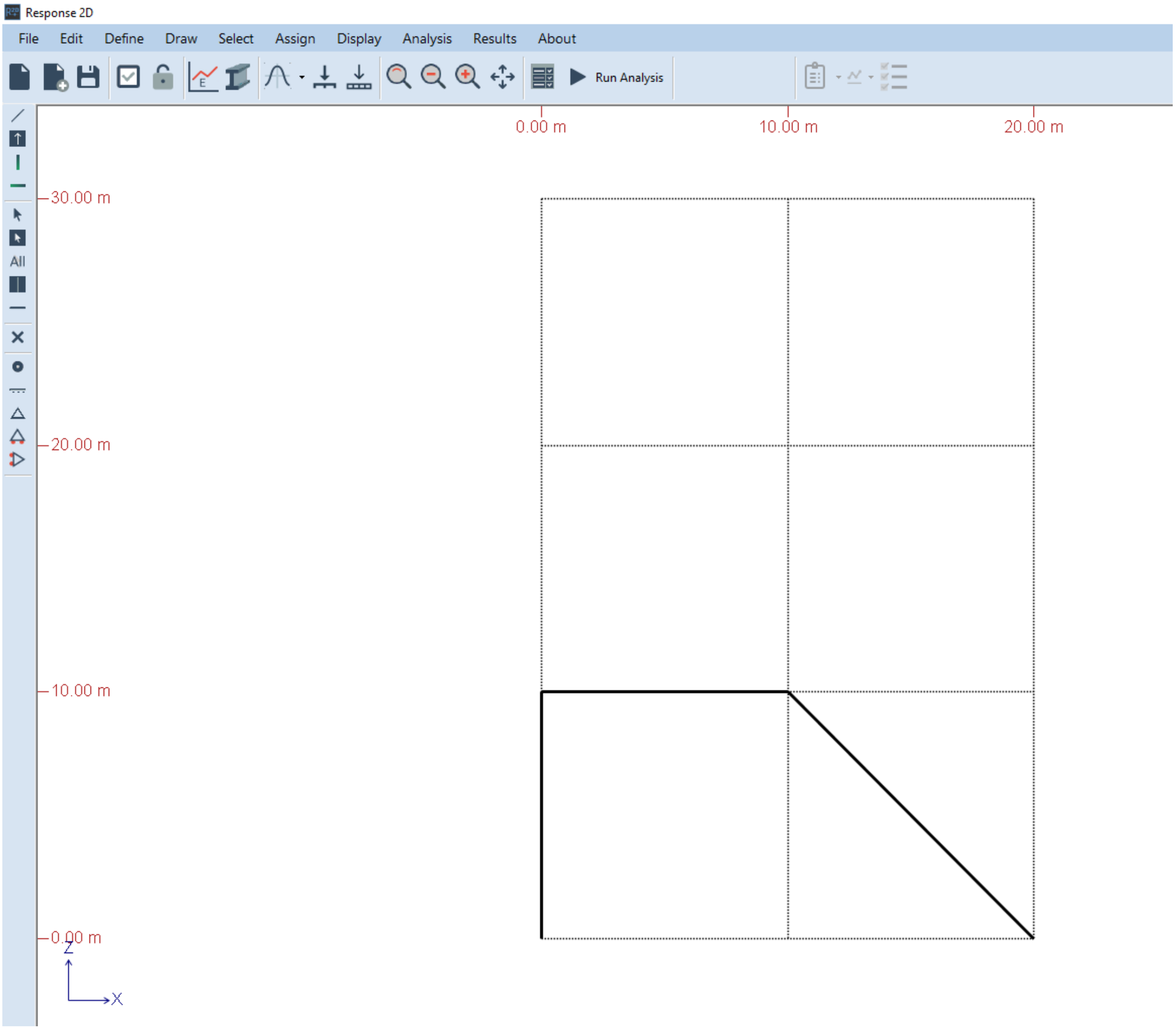


Figure 3-4: Grids and modeled frame display in Response 2D

3.1.3 Classes Development

After the development of modeling environment, there is need of various classes to store the information provided by the user. For example, when a user draws a member on the modeling environment, it ends up with some important information which has been provided by the user that are joints coordinates, angle of member from some reference axis etc. After that the user can also assign restraint to the joint which is also an information provided by the user. Accordingly, classes have been developed to store the information and each class has properties and methods to perform the calculations. The list of classes can be seen below.

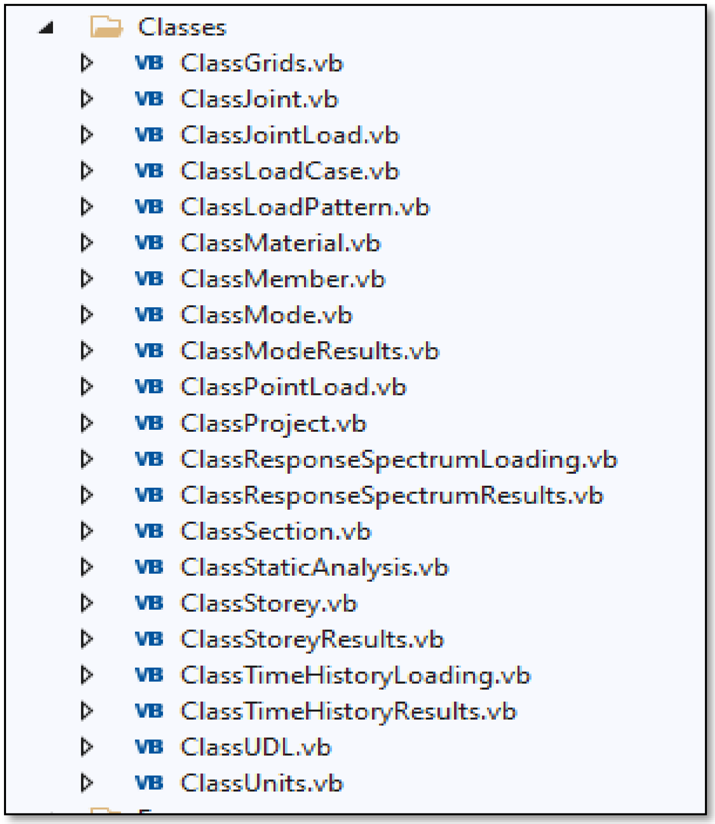


Figure 3-5: Different classes in Response 2D development

Each of these classes will have some properties and methods, for example Class Joint has properties like x coordinate, y coordinate, restraint type etc. and methods like calculate lumped masses etc. Similarly, Class Load Case will have properties like name, load case type (i.e., linear static, response spectrum, time history), action type (if this load case will be analyzed or not) etc.

3.1.4 Structural Engineering Calculations

After the creation of classes, the next step is to perform the structural engineering calculations. Stiffness method has been used to perform the linear static analysis. The most important calculation is the determination of global stiffness matrix and the mass matrix. Global stiffness matrix has been assembled from the members global stiffness matrix, which in turn has been calculated from the member local stiffness matrix using the equation 3.1.

$$K_{Global} = T' \times K_{Local} \times T \qquad 3.1$$

Here T is the transformation matrix which is used to transform the local matrix to global matrix. The calculations of K_{Local}, T and K_{Global} can be seen in the Fig 3-6, 3-7 and 3-8 respectively. In the calculation of local stiffness matrix, length of the member will be calculated from the joint coordinates, while section area and material elastic modulus will be provided by the user.

```
Public ReadOnly Property KLocal As MathNet.Numerics.LinearAlgebra.Matrix(Of Double)
    Get
        Try
            Dim matK As MathNet.Numerics.LinearAlgebra.Matrix(Of Double)
            matK = MathNet.Numerics.LinearAlgebra.CreateMatrix.Sparse(Of Double)(6, 6)
            With Section
                matK(0, 0) = .Area * .Material.ElasticModulus / Length
                matK(0, 3) = - .Area * .Material.ElasticModulus / Length
                matK(1, 1) = 12 * .Material.ElasticModulus * .Ix / Length ^ 3
                matK(1, 2) = 6 * .Material.ElasticModulus * .Ix / Length ^ 2
                matK(1, 4) = -12 * .Material.ElasticModulus * .Ix / Length ^ 3
                matK(1, 5) = 6 * .Material.ElasticModulus * .Ix / Length ^ 2
                matK(2, 1) = 6 * .Material.ElasticModulus * .Ix / Length ^ 2
                matK(2, 2) = 4 * .Material.ElasticModulus * .Ix / Length
                matK(2, 4) = -6 * .Material.ElasticModulus * .Ix / Length ^ 2
                matK(2, 5) = 2 * .Material.ElasticModulus * .Ix / Length
                matK(3, 0) = - .Area * .Material.ElasticModulus / Length
                matK(3, 3) = .Area * .Material.ElasticModulus / Length
                matK(4, 1) = -12 * .Material.ElasticModulus * .Ix / Length ^ 3
                matK(4, 2) = -6 * .Material.ElasticModulus * .Ix / Length ^ 2
                matK(4, 4) = 12 * .Material.ElasticModulus * .Ix / Length ^ 3
                matK(4, 5) = -6 * .Material.ElasticModulus * .Ix / Length ^ 2
                matK(5, 1) = 6 * .Material.ElasticModulus * .Ix / Length ^ 2
                matK(5, 2) = 2 * .Material.ElasticModulus * .Ix / Length
                matK(5, 4) = -6 * .Material.ElasticModulus * .Ix / Length ^ 2
                matK(5, 5) = 4 * .Material.ElasticModulus * .Ix / Length
            End With
            Return matK
        Catch ex As Exception
            MsgBox(ex.Message)
            Return Nothing
        End Try
    End Get
```

Figure 3-6: Calculation of member local stiffness matrix in Response 2D

```
Public ReadOnly Property T As MathNet.Numerics.LinearAlgebra.Matrix(Of Double)
    Get
        Try
            Dim matT As MathNet.Numerics.LinearAlgebra.Matrix(Of Double)
            matT = MathNet.Numerics.LinearAlgebra.CreateMatrix.Sparse(Of Double)(6, 6)
            matT(0, 0) = l
            matT(0, 1) = m
            matT(1, 0) = -m
            matT(1, 1) = l
            matT(2, 2) = 1
            matT(3, 3) = l
            matT(3, 4) = m
            matT(4, 3) = -m
            matT(4, 4) = l
            matT(5, 5) = 1
            Return matT
        Catch ex As Exception
            MsgBox(ex.Message)
            Return Nothing
        End Try
    End Get
End Property
```

Figure 3-7: Calculation of member transformation matrix in Response 2D

```
Public ReadOnly Property KGlobal As MathNet.Numerics.LinearAlgebra.Matrix(Of Double)
    Get
        Try
            Dim KG As MathNet.Numerics.LinearAlgebra.Matrix(Of Double)
            KG = MathNet.Numerics.LinearAlgebra.CreateMatrix.Sparse(Of Double)(6, 6)
            KG = T.Transpose * KLocal * T
            Return KG
        Catch ex As Exception
            MsgBox(ex.Message)
            Return Nothing
        End Try
    End Get
End Property
```

Figure 3-8: Calculation of member global stiffness matrix in Response 2D

After the formation of global stiffness matrix, force matrix has been created to perform the linear static analysis for the determination of joint displacements using the equation 3.2.

$$F = K_{Global}.u \qquad 3.2$$

Afterwards, reactions and member forces can be calculated from the joint displacements.

For response spectrum analysis, for the time being ASCE 7-16 response spectrum function has been available only (Fig 3-9). SRSS and CQC modal combination methods are available for combining the response. Before creating the load case for response spectrum analysis, there must be some defined load function because the load function is an input parameter of response spectrum load case.

For time history analysis, two solution methods are available i.e., Direct Integration and Modal. Further in direct integration time history analysis, two methods have been implemented in Response 2D, i.e., Newmark linear acceleration and Newmark average acceleration method. The user can provide the time history load function in two different ways either from excel file or by inputting each value.

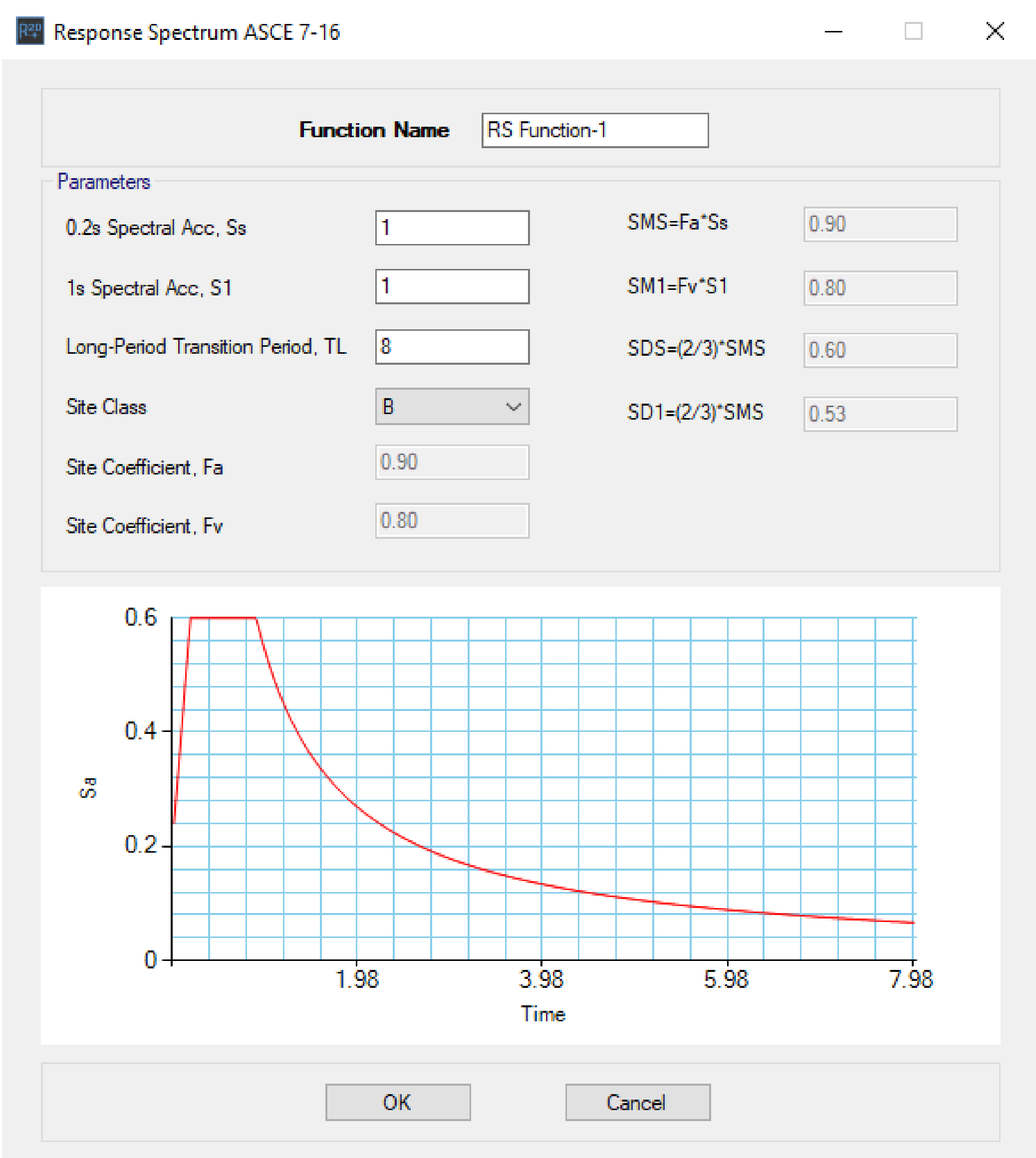


Figure 3-9: Response spectrum ASCE 7-16 function form in Response 2D

3.1.5 Results Display

In tabular form, Response 2D can show joint displacements, joint reactions, modal analysis results and member forces (both local and global). In graphical form, it can show deformed shape, reactions, axial force, shear force and bending moment diagrams. The animations of deformed shape and modes shape are also available in Response 2D. In Fig 3-10, mode shape display of second mode can be seen for 15 story-frame in Response 2D. Above all, the most important feature of Response 2D is the display of each mode response and its contribution percentage in the combined response in modal time history analysis (MTHA). For example, in Fig 3-11, first mode response has been shown for the joint displacements for maximum and minimum values. Apart from the extreme values, joint displacements can also be seen for all the time steps of load case. Moreover, instead of all joints, the joint displacement results can also be seen for only the top joint for extreme or all time step values.

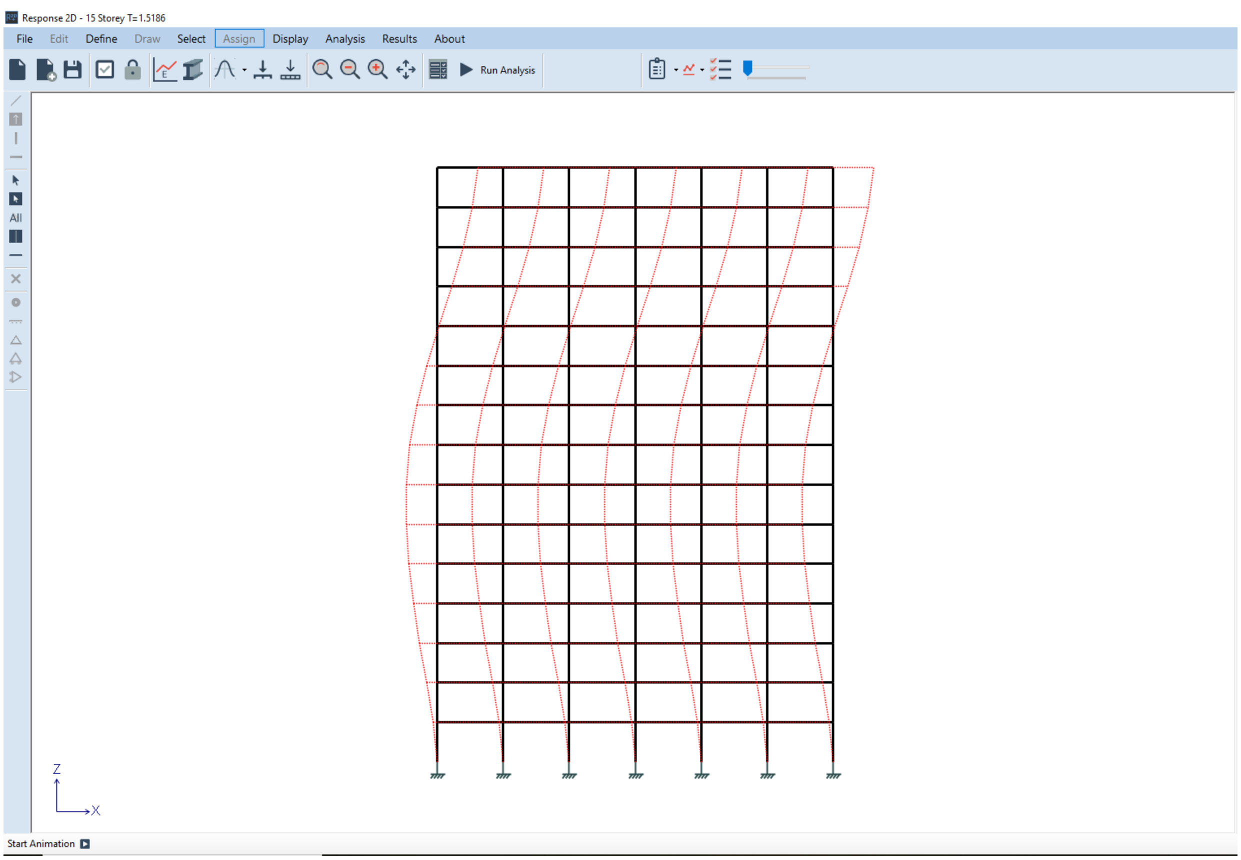
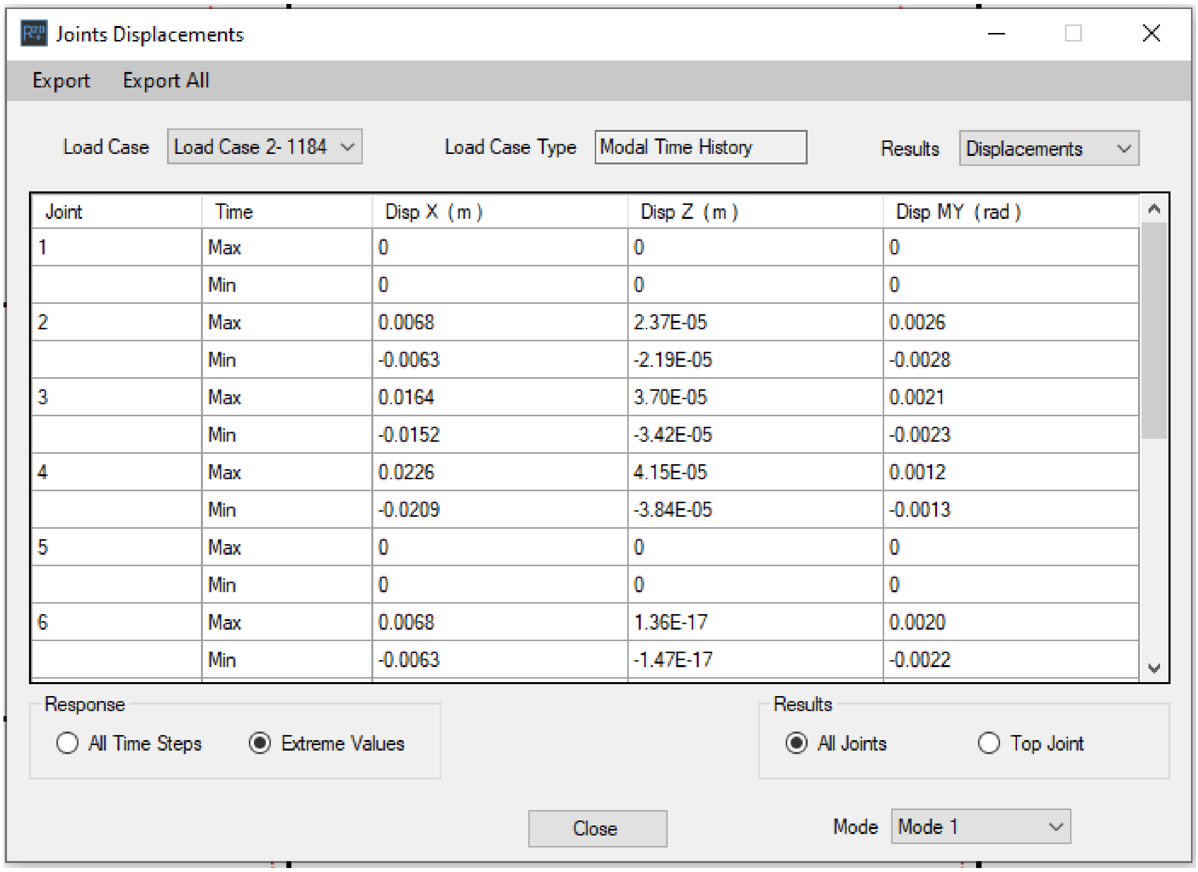


Figure 3-10: Modes shapes display in Response 2D



Joint	Time	Disp X (m)	Disp Z (m)	Disp MY (rad)
1	Max	0	0	0
	Min	0	0	0
2	Max	0.0068	2.37E-05	0.0026
	Min	-0.0063	-2.19E-05	-0.0028
3	Max	0.0164	3.70E-05	0.0021
	Min	-0.0152	-3.42E-05	-0.0023
4	Max	0.0226	4.15E-05	0.0012
	Min	-0.0209	-3.84E-05	-0.0013
5	Max	0	0	0
	Min	0	0	0
6	Max	0.0068	1.36E-17	0.0020
	Min	-0.0063	-1.47E-17	-0.0022

Figure 3-11: First mode response display for joint displacement in Response 2D

3.1.6 Analysis Procedure in Response 2D

The procedure of performing the analysis in Response 2D is as simple as in most of the FEA programs. First, grids will be defined followed by the creation of the geometry of frame. At the same time, material will be defined separately and then sections will be created. I/Wide Flange, Channel, Box, Round, Pipe, Tube, and Angle sections are available in Response 2D (Fig 3-12). Generalized section option is also available in which section can be defined by directly providing the area and moment of inertias. Afterwards the sections will be assigned to the members. Next step is to create the load cases for which analysis needs to be run. For time history and response spectrum analysis, load functions need to be defined before the creation of load cases. Then analysis can be run for getting the results. The analysis can be run for as many load cases as per the user's requirement. Different steps involved in the analysis procedure have also been shown in the flow chart in Fig 3-13.

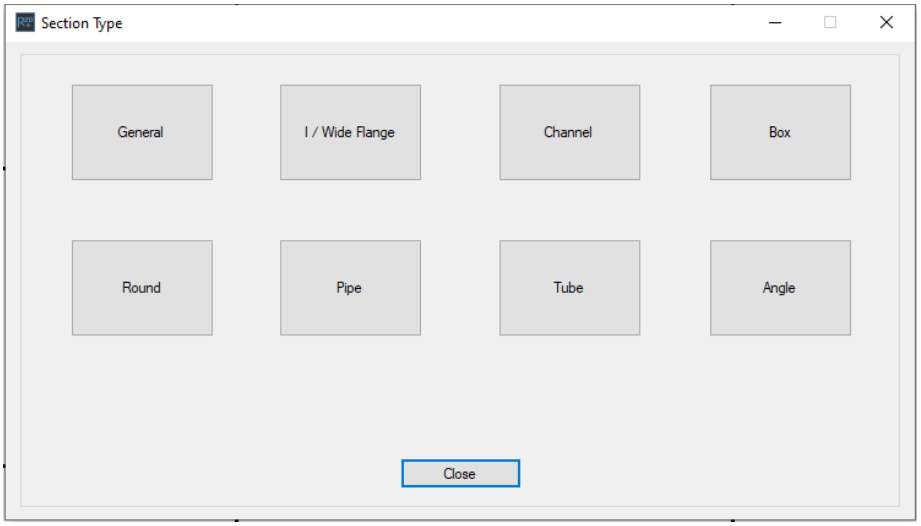


Figure 3-12: Different section types available in Response 2D

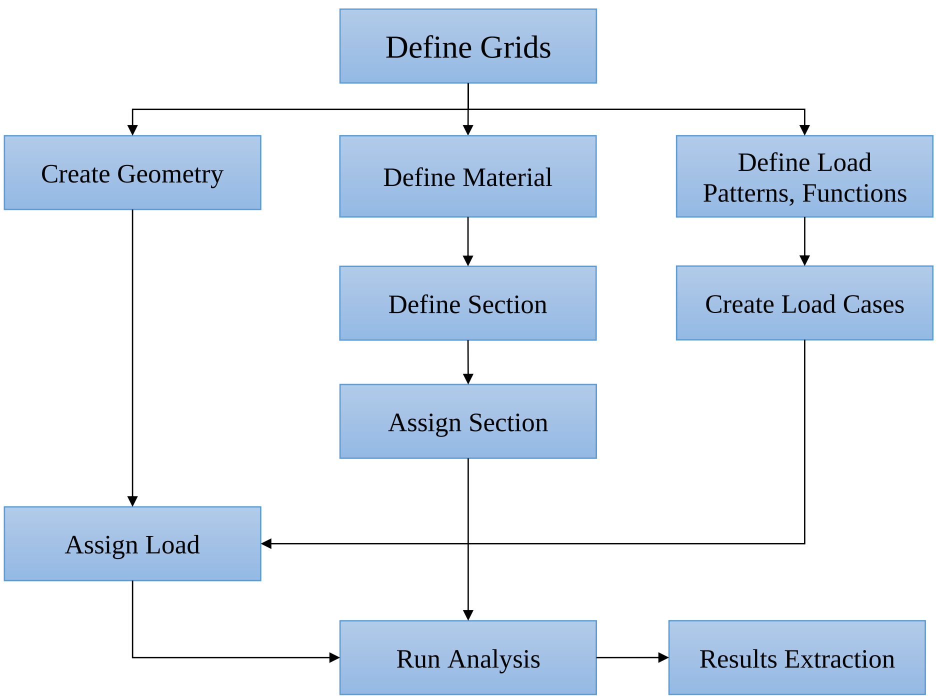


Figure 3-13: Schematic flowchart of analysis procedure in Response 2D

3.1.7 Testing of Response 2D

Response 2D has been extensively tested for the accuracy of its results. For this purpose, various examples of linear static, response spectrum, direct integration time history and modal time history load cases have been taken and the analysis have been performed in Response 2D. The results have been compared with either the manual calculations or from the results of SAP2000.

One example from linear static load case has been presented here for the testing of results. The two-dimensional frame consists of two frames subjected to point loads and uniformly distributed loads as depicted in Fig 3-14. Modulus of elasticity of material for both the sections is 200GPa while the sections are generalized section with I=60(10^6) mm^4 and A=600 mm^2. The model has been drawn in Response 2D as shown in the Fig 3-15.

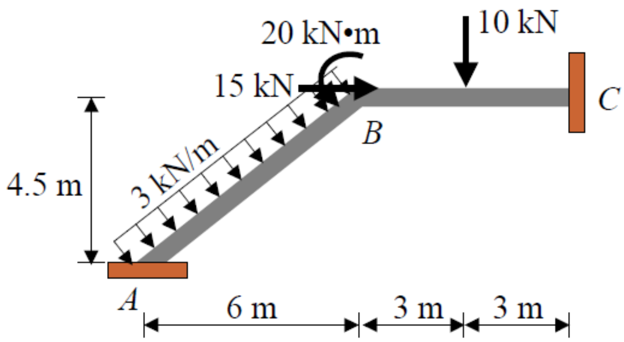


Figure 3-14: Linear static test example

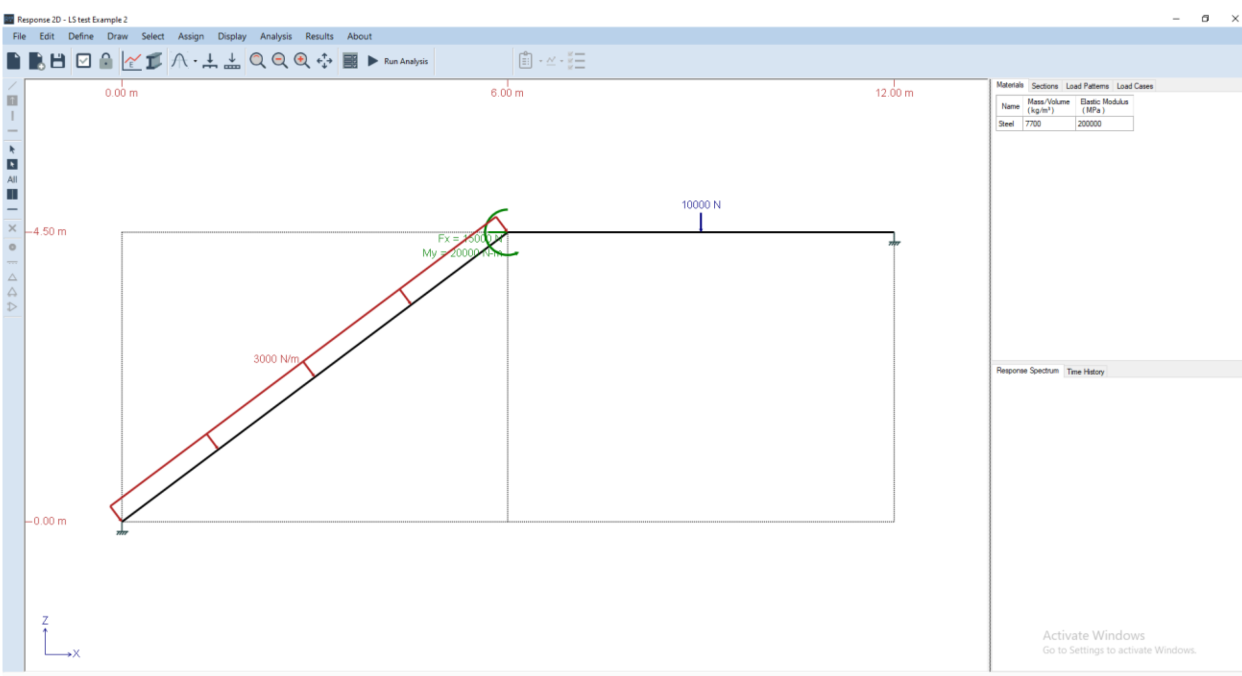
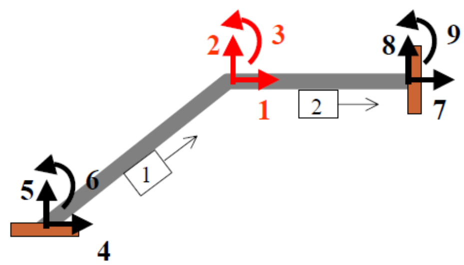


Figure 3-15: Test example modeled in Response 2D

Since the frame has only three unrestrained degrees of freedom i.e., 1,2 and 3 (Fig 3-16), hence there will be three displacements whose values can also be seen from the Fig 3-16. These displacement values have been calculated manually.



$$\begin{pmatrix} D_1 \\ D_2 \\ D_3 \end{pmatrix} = \begin{pmatrix} 1.751(10^{-3})\ \text{m} \\ -4.388(10^{-3})\ \text{m} \\ 2.049(10^{-3})\ \text{rad} \end{pmatrix}$$

Figure 3-16: Test example results from manual calculations

Now these joints displacements results can also be seen in Fig 3-17 which has been calculated in Response 2D. The accuracy of results has been calculated (Table 3-1) and the results are in compliance with the manual calculations.

Apart from the joint displacements results, other results like joint reactions, member forces, modes time periods and shapes have also been compared with the manual calculation results or from SAP2000 results, and they are more than 99% accurate (on average).

Examples from other load cases i.e., Response spectrum, direct integration time history and modal time history have been also tested with the results from SAP2000 and the results are also found to be accurate in accordance with the SAP2000 results.

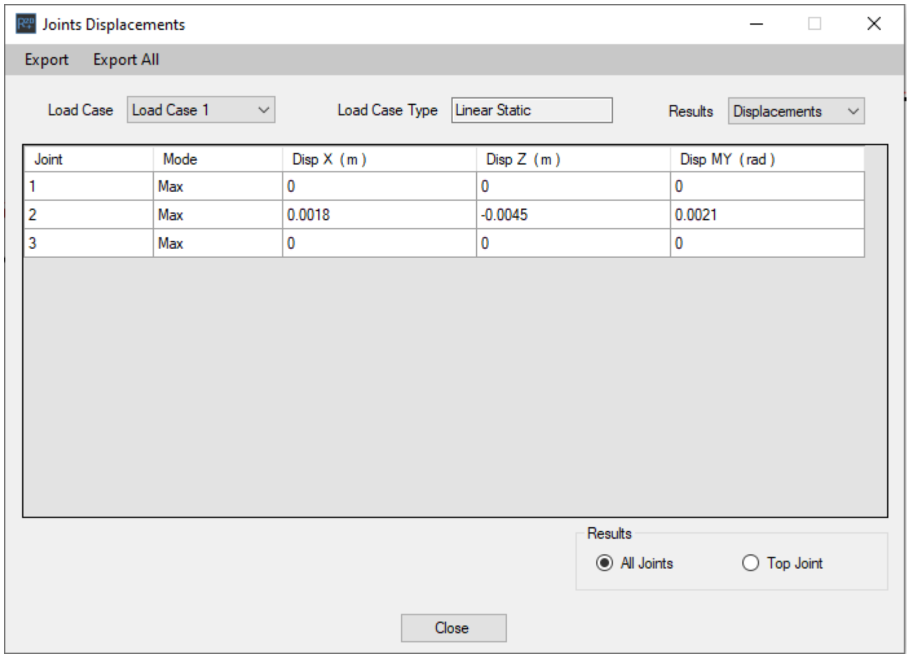


Figure 3-17: Test example results in Response 2D

Table 3-1: Comparison of results from manual calculations and Response 2D

Joint Degree of Freedom	Value from Manual Calculations (mm)	Value from Response 2D (mm)	Percentage Difference (%)
1	1.751000	1.766594	0.85
2	-4.388000	-4.452398	1.44
3	0.002049	0.002052	0.14

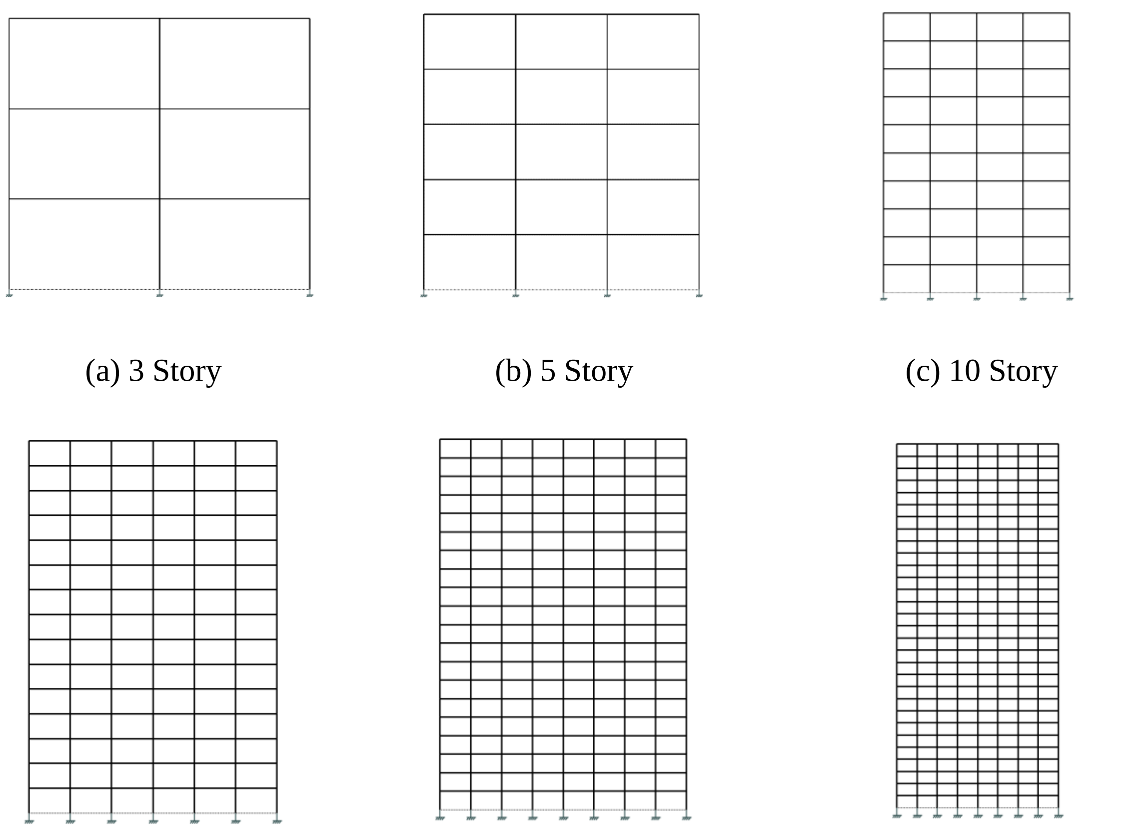
3.2 Analysis of Generic Frames on Response 2D

3.2.1 Selection of Generic Frames

Six generic two-dimensional RC frames have been taken such that their properties correspond to the frames in the real buildings. Concrete mass per volume has been taken as 2500kg/m^3 whereas young's modulus is kept as 30000 MPa. All columns and beams sections in these six frames are rectangular sections. Each story height has been taken as 3m while bay size is 5m. The generic frames elevation view (from modelling in Response 2D) has been shown in Fig 3-18. The first mode time periods (with fixed supports), stories number, bays number, heights in these six 2D frames can be seen in table 3-2.

Table 3-2: Generic frames detail

Frame	Stories	Bays	Total Height (m)	First Mode Time Period (sec)
3-Storey	3	2	9	0.3425
5-Storey	5	3	15	0.5312
10-Storey	10	4	30	1.0200
15-Storey	15	6	45	1.5186
20-Storey	20	8	60	2.0266
30-Storey	30	8	90	3.0440



(a) 3 Story (b) 5 Story (c) 10 Story

(d) 15 Story (e) 20 Story (f) 30 Story

Figure 3-18: Generic frames (modeled in Response 2D) elevation view

3.2.2 Selection of Ground Motions

Islamabad capital territory (ICT) has been idealized for the detailed analysis for studying the effect of soil-structure interaction on the higher mode effects. The two faults that mainly contribute to the seismicity of ICT are main boundary thrust and strike-slip faults [12]. A suite of seven ground motion histories for these two fault types with magnitudes of $M6.3 - 7.8$ are employed to carry out the detailed modal time history analysis (MTHA) procedure. These ground motion records are accessed using the NGA-West2 coast of (PEER ground motion database) and are most likely to occur within 10-50kms of source to site distance with site undergoing 490-620m/s velocity of shear waves. (Table 3-3) These time histories of ground motions are adjusted and scaled by spectral matching in "SeismoMatch" software to match with target response spectra of UBC-97. This target spectrum is developed for a 5% damped design base earthquake (DBE) level for the region of Islamabad capital territory (ICT) with Cv=0.32g and Ca=0.24g (UBC-97) (Fig 3-19).

Table 3-3: Details of seven ground motions selected

Event	Year	Station	Magnitude	Mechanism	R_{rup} (km)	V_{s30} (m/s)
Chichi Taiwan	1999	CHY010	7.62	Reverse oblique	19.96	538.69
Chichi Taiwan	1999	CHY029	7.62	Reverse oblique	12.65	573.04
Chichi Taiwan	1999	CHY087	7.62	Reverse oblique	28.91	505.2
Chichi Taiwan	1999	TCU042	7.62	Reverse oblique	26.31	578.98
Chichi Taiwan_06	1999	CHY029	6.3	Reverse	41.36	544.74
Chuetsu	2007	Nadachiku Joetsu City	6.8	Reverse	35.93	570.62
Iwate Japan	2008	Misato Akita City	6.9	Reverse	41.72	552.38

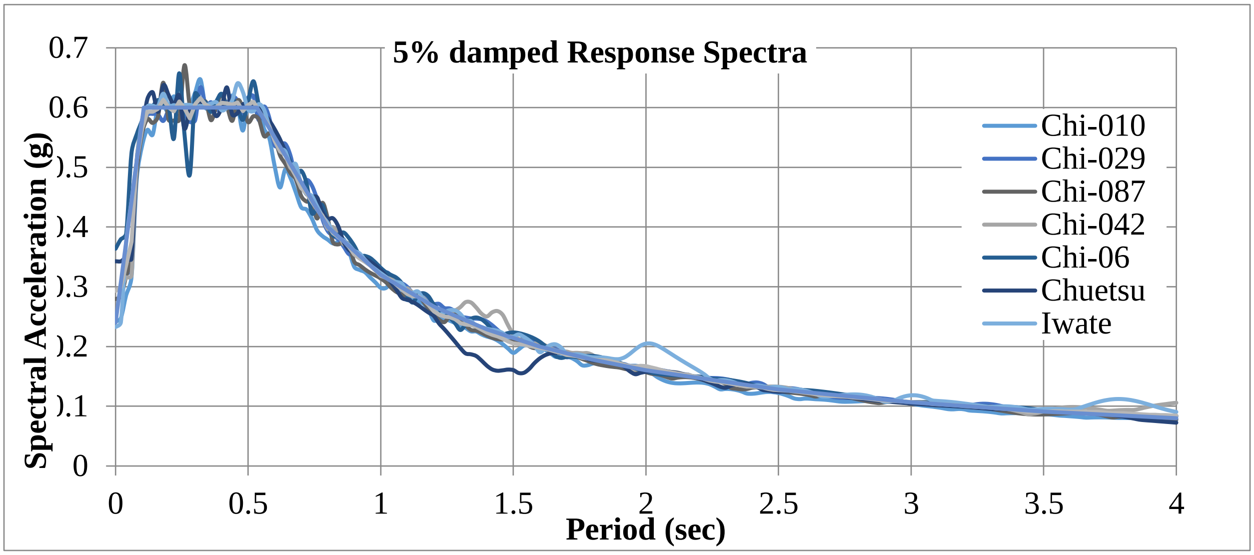


Figure 3-19 The acceleration response spectra of selected ground motions matched with target spectra

3.2.3 Analysis with Fixed Base

Analysis have been run on Response 2D for all the six frames with fixed base condition for all the seven ground motions.

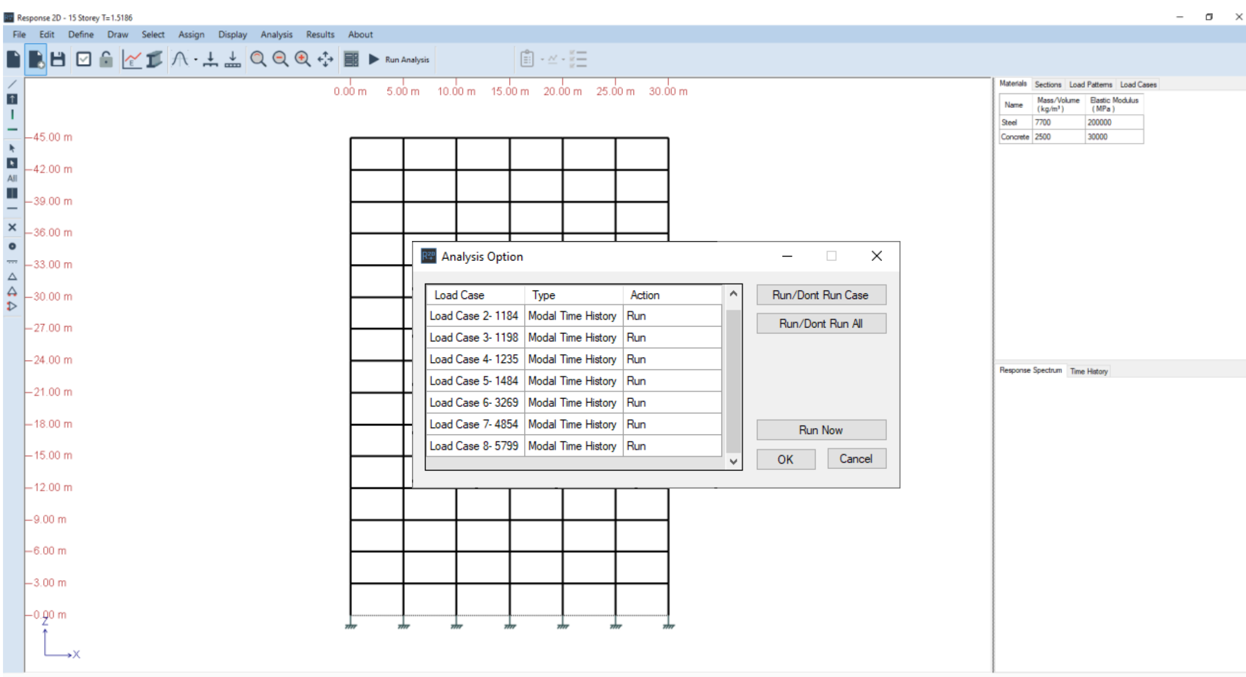


Figure 3-20: Analysis performed with fixed base in Response 2D

These ground motions have been applied in translational x direction. Modal damping has been taken as 5% as seen in the Fig 3-21. Reactions result have been taken from Response 2D which will be used further to design the footings.

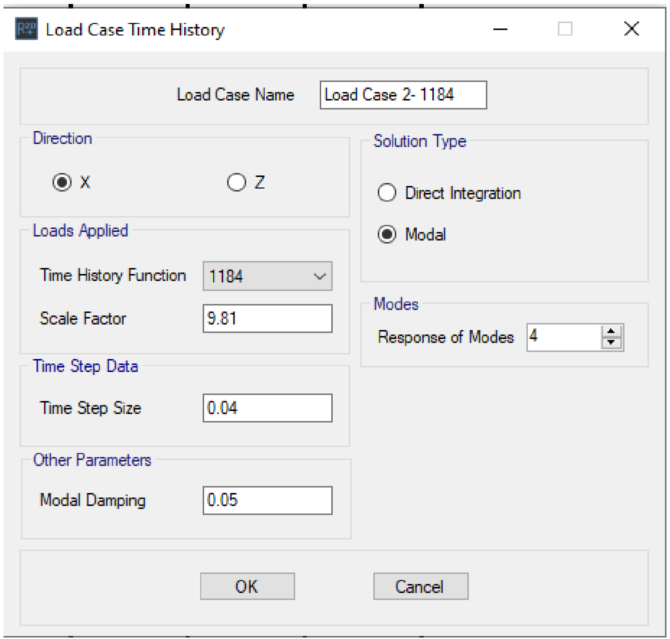


Figure 3-21: Modal time history load case creation in Response 2D

3.2.4 Calculation of Soil Stiffnesses

Footings have been designed for the six generic frames. Square footings have been idealized for all these frames. Soil spring properties have been calculated for the soil type D (UBC 97) which is mostly used for Islamabad Capital Territory (ICT) using Pais and Kausel (1988) equations. The calculate values of stiffnesses can be seen in Table 3-4.

Table 3-4: Calculated soil stiffness values for six frames

No. of Stories	**K_x (10^6 N/m)**	**K_z (10^6 N/m)**	**K_{yy} (10^6 Nm/rad)**
3	0.7737	0.9805	0.0857
5	1.1654	1.4723	0.2890
10	1.8606	2.3383	1.1505
15	2.7790	3.4712	3.7384
20	3.1092	3.8787	5.2083
30	4.1350	5.1524	12.1934

3.2.5 Analysis with Flexible Base

Now, analysis have been run with the flexible base. The stiffnesses value calculated in the above table has been provided to the springs assigned to the base. Afterwards, modal time history analysis has been performed in Response 2D for all the seven ground motions and the results have been compared with that of fixed base results.

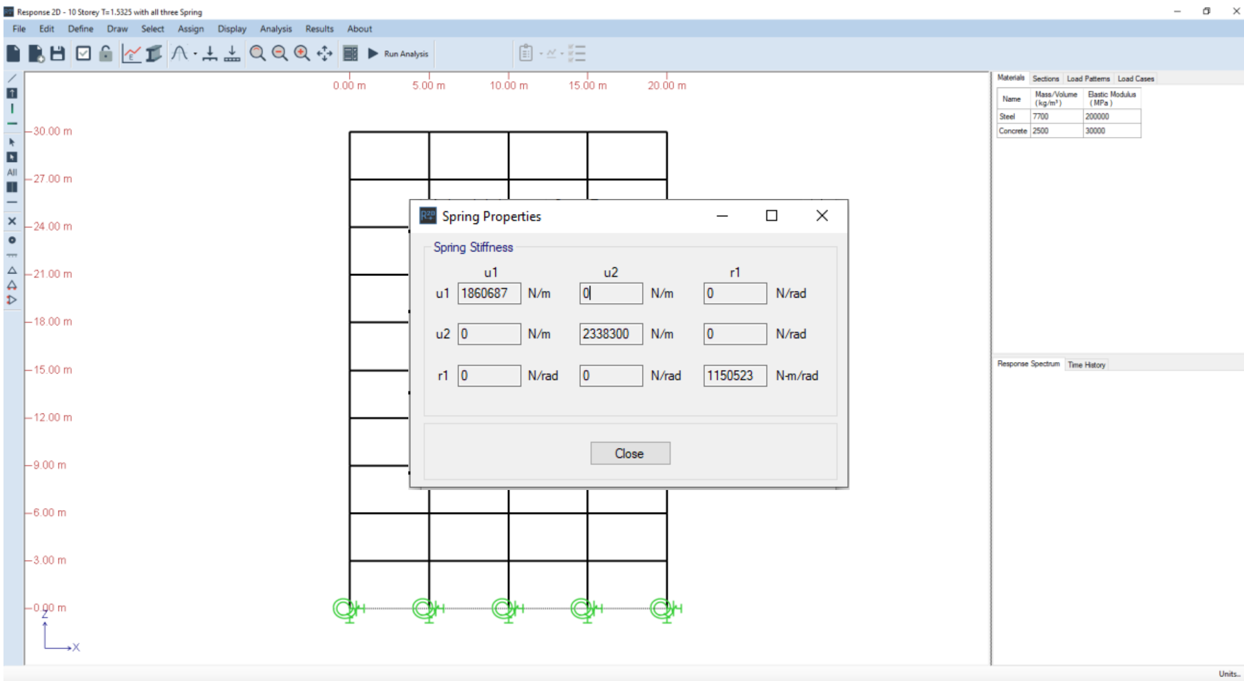


Figure 3-22: Analysis performed with flexible base in Response 2D

Chapter 4

Results and Discussion

Dynamic responses like base shear, base moment, roof displacement, story shear, story moment, mode shapes and time periods have been thoroughly investigated and compared for the cases of fixed and flexible base. Higher modes effects have also been studied for these two support conditions cases to quantify their effects.

4.1 Time Period

The introduction of flexible base with the incorporation of soil effect has resulted in the reduction of global stiffness of the frame structure. Accordingly, this will result in the variation of time periods of the frames. The comparison of first four mode time periods for the fixed and flexible base case can be seen in Fig 4-1.

Story No.	Height (m)	T_1(sec) Flexible Base	T_1(sec) Fixed Base	Percentage Increase (%)
3	9	0.7433	0.3425	117.0218
5	15	0.9638	0.5312	81.4382
10	30	1.5325	1.0279	49.0903
15	45	2.0961	1.5186	38.0284
20	60	2.5586	2.0266	26.2508
30	90	3.88	3.076	26.1378

First Mode Time Periods Comparison

Story No.	Height (m)	T_2(sec) Flexible Base	T_2(sec) Fixed Base	Percentage Increase (%)
3	9	0.2883	0.1037	178.0135
5	15	0.3373	0.1742	93.6280
10	30	0.4694	0.3709	26.5570
15	45	0.6625	0.5552	19.3263
20	60	0.8454	0.7409	14.1044
30	90	1.2813	1.1799	8.5939

Second Mode Time Periods Comparison

Story No.	Height (m)	T_3(sec) Flexible Base	T_3(sec) Fixed Base	Percentage Increase (%)
3	9	0.2455	0.0575	326.9565
5	15	0.3049	0.0947	221.9640
10	30	0.4221	0.2079	103.0303
15	45	0.5061	0.3354	50.8944
20	60	0.5343	0.4649	14.9279
30	90	0.8126	0.7469	8.7963

Third Mode Time Periods Comparison

Story No.	Height (m)	T_4(sec) Flexible Base	T_4(sec) Fixed Base	Percentage Increase (%)
3	9	0.1411	0.0165	755.1515
5	15	0.2138	0.0613	248.7765
10	30	0.3168	0.1383	129.0672
15	45	0.4065	0.2105	93.1116
20	60	0.5177	0.312	65.9294
30	90	0.6154	0.5064	21.5244

Fourth Mode Time Periods Comparison

Figure 4-1: Time Periods Comparison for First Four Modes with Fixed and Flexible Base

It can be clearly seen that the with increasing height of the frame structure, percentage difference between the T_1 of fixed and flexible base tends to decrease. For three-story frame, the difference is more than two times however with increasing story numbers, the difference is decreasing. So, it can be predictable that the introduction of base flexibility will affect the first mode time-period more for short story frames and this trend will go on decreasing with increasing height of the frame. For tall frames the effect of soil on the first mode time-period is nearly negligible. The same trend can be seen for the higher modes time periods, however from the second, third and fourth time periods percentage difference, the difference is much higher for the fourth mode if compared with the corresponding second and third mode time-period of the same height frame. So, it can be inferred that the introduction of soil will affect the time periods of higher order modes more as compared to the lower order modes. But this trend remains true for short story frame and will become no longer true going from mid to high rise frames as we can see in the thirty-story frame that the percentage differences for first four modes are 26.13, 8.59, 8.79 and 21.52. The comparison between time periods of first four modes can also be seen with the help of graph in Figure 4-2 to Figure 4-5.

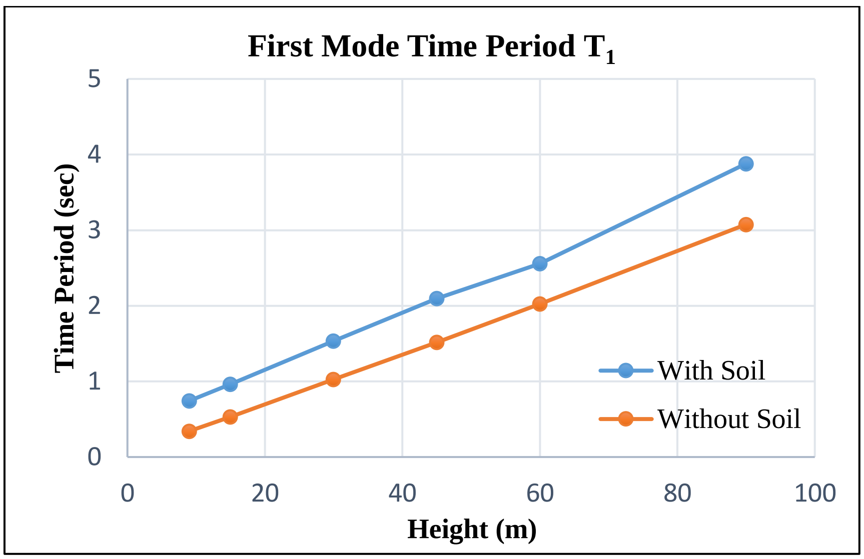


Figure 4-2: First mode time-periods comparison of fixed and flexible base

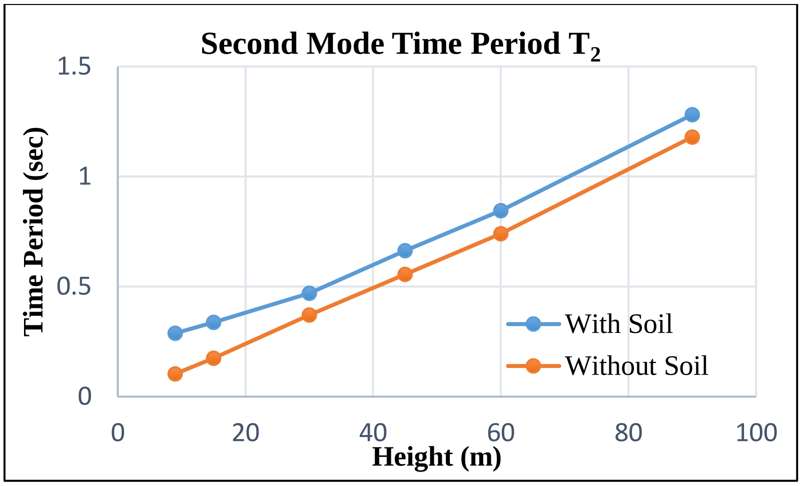


Figure 4-3: Second mode time-periods comparison of fixed and flexible base

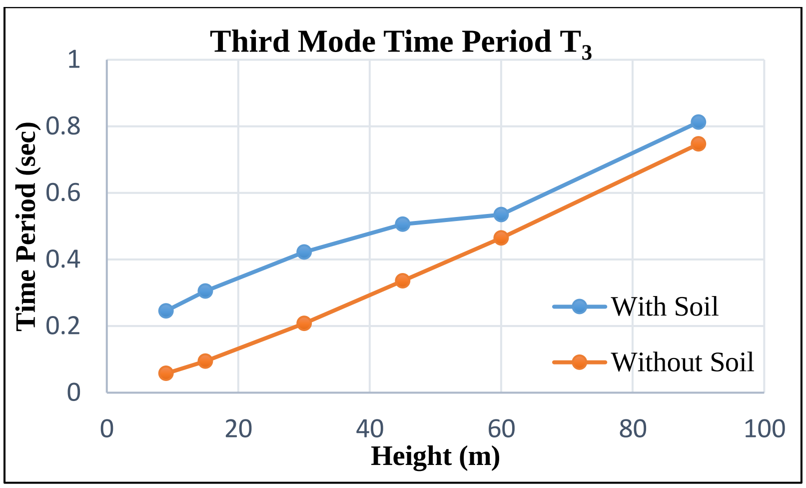


Figure 4-4: Third mode time-periods comparison of fixed and flexible base

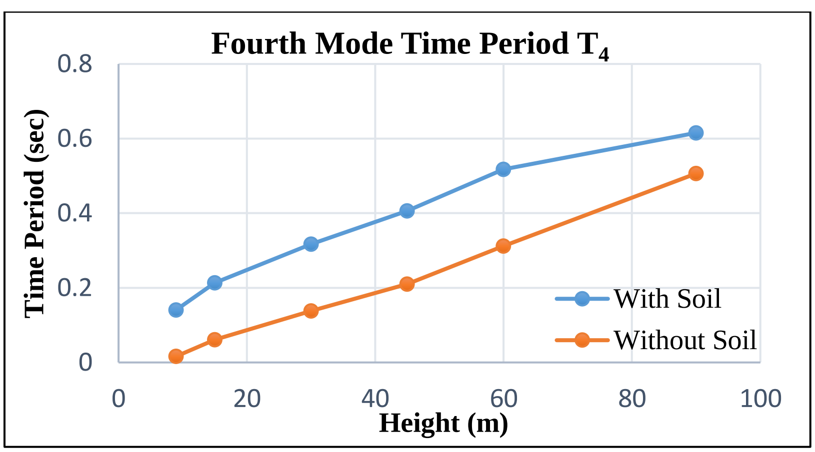


Figure 4-5: Fourth mode time-periods comparison of fixed and flexible base

4.2 Mode Shapes

Mode shapes of first four modes of 3,15 and 30 story-frame have been shown in the Fig 4-6, 4-7 and 4-8 respectively with fixed and flexible base. (Other frames mode shapes can be seen in Appendix C). It can be clearly seen that the mode shapes are different with fixed and flexible base. However, this difference is becoming smaller and smaller as the height of the frame is increased and it is noticeable that for 30 story frame the mode shapes are nearly same for fixed and flexible base. This is because that with increasing height of the frame the structural flexibility is dominating, and thus base flexibility is not affecting much the mode shapes. Accordingly, it can be inferred that short and stiff frames are more prone to be affected by the incorporation of soil effect and as the flexibility of the structure is increased, base flexibility effect tends to decrease.

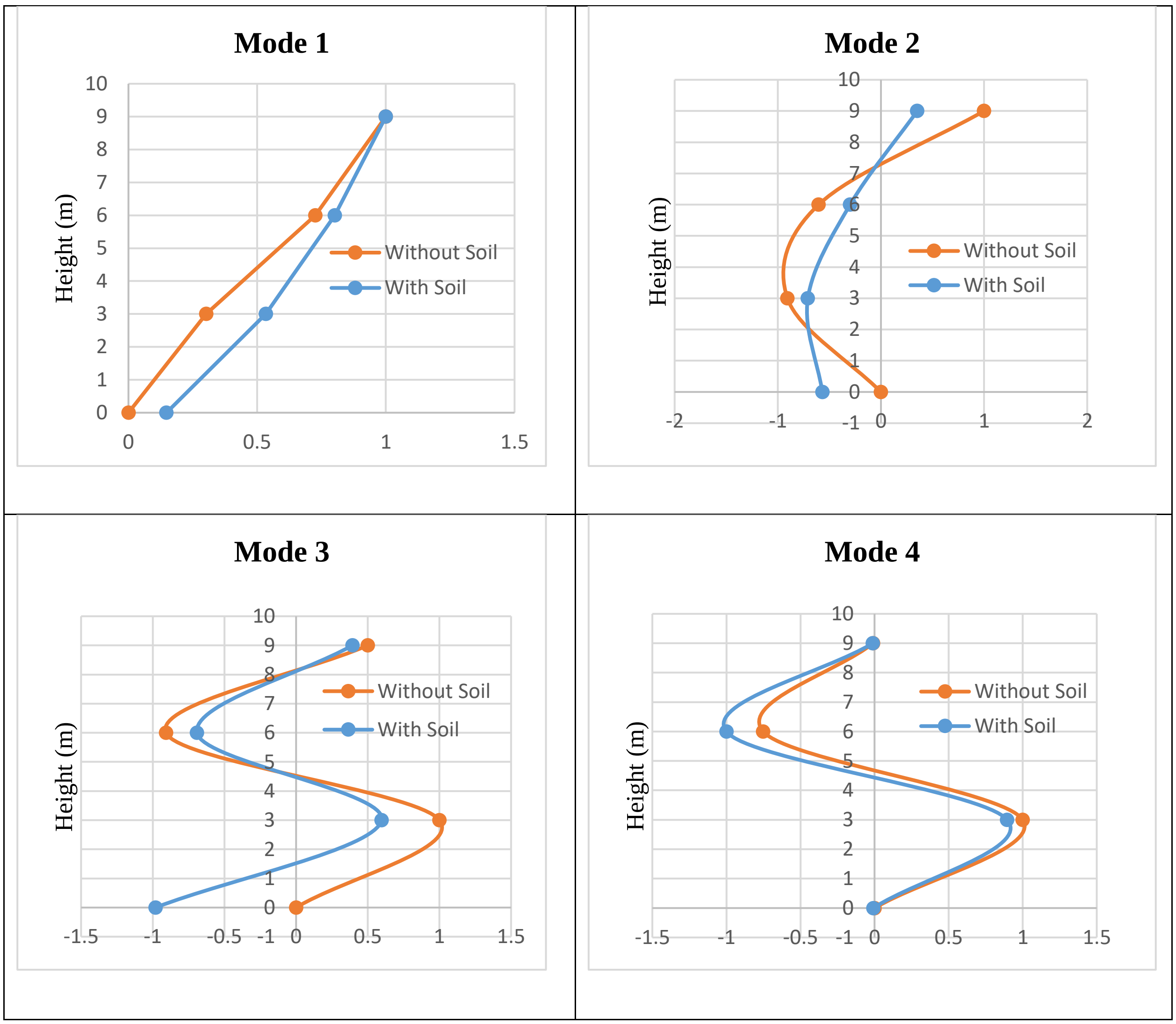


Figure 4-6: Modes shape comparison for 3 story-frame

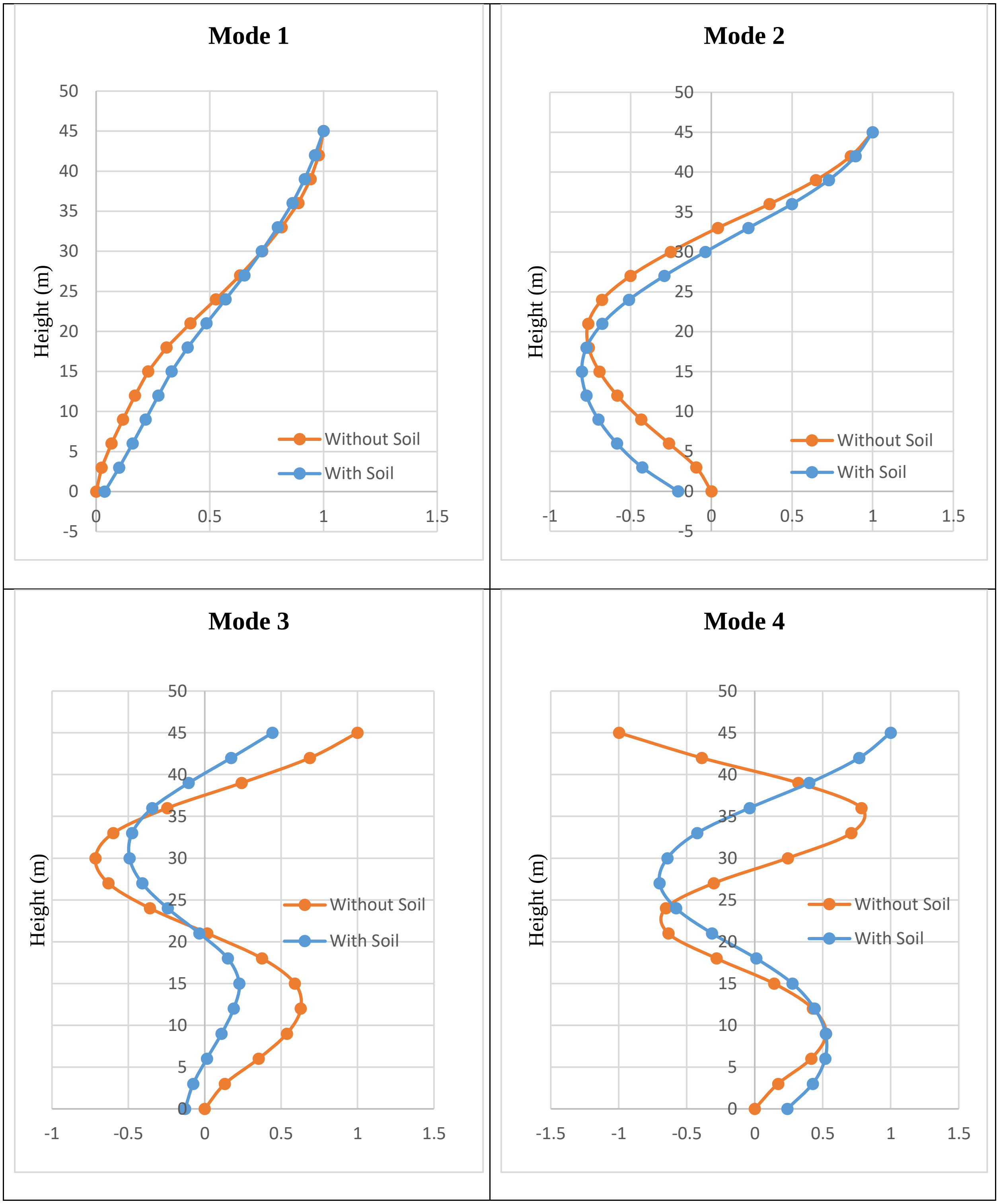


Figure 4-7: Modes shape comparison for 15 story-frame

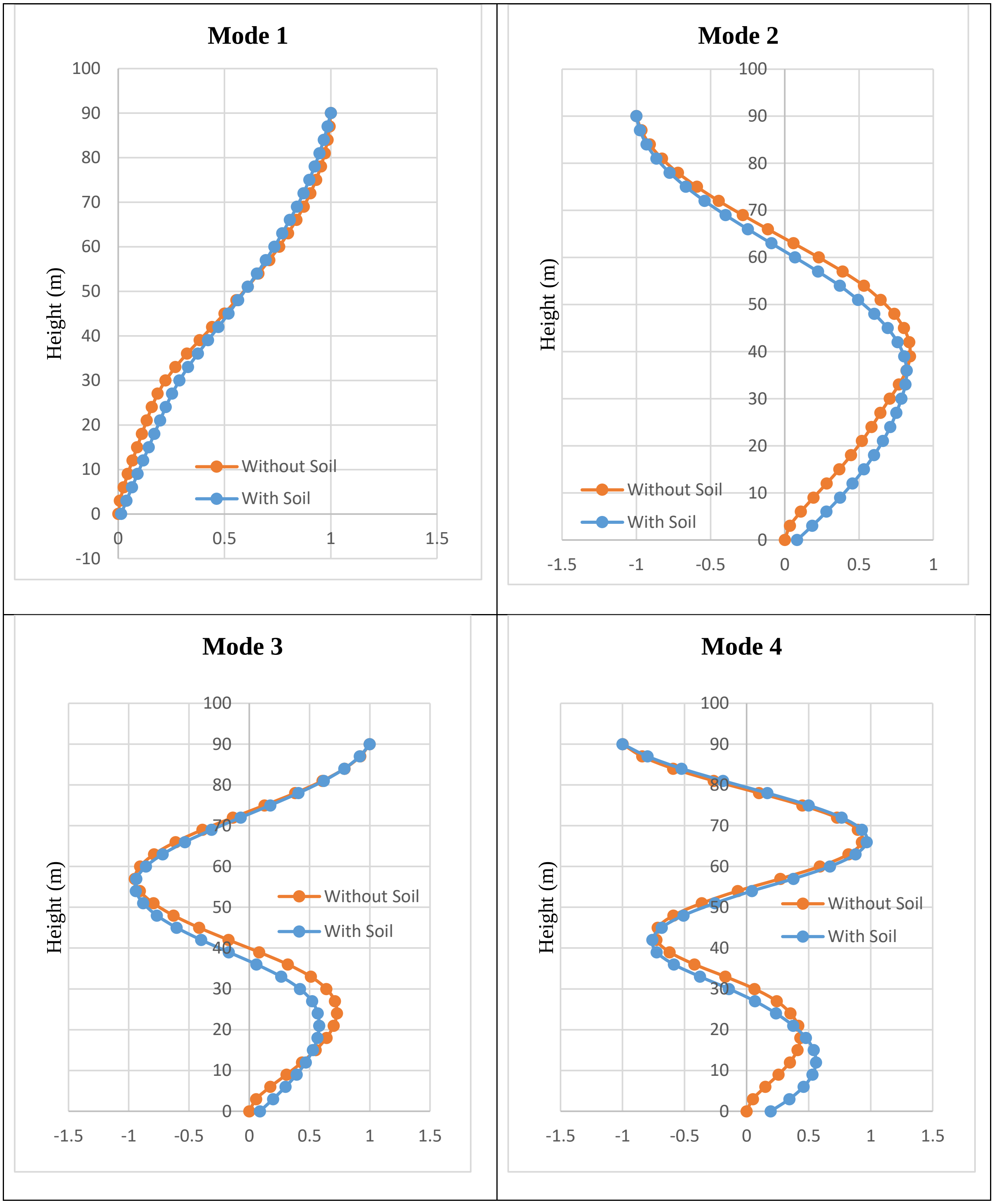


Figure 4-8: Modes shape comparison for 30 story-frame

4.3 Modal Time History Analysis Results

After performing modal time history analysis on six generic frames and for the seven ground motions with modeling the effect of soil and without modeling it, there are a lot of results i.e., combined responses and modes response and then responses for fixed base case and responses for flexible base case. These various results can be better understood with the help of flow chart as depicted in Fig 4-9.

The comparison can be made between fixed and flexible base cases for the combined response or for some mode response. For example, the comparison graph between combined roof displacement in the x-direction for the two cases of fixed and flexible case for 3 story-frame has been shown in Fig 4-10.

Also, the comparison can be drawn for some particular response between the combined response and the modes response for fixed or flexible case. For example, in Fig 4-11 story shear combined and modes responses have been compared for 10 story-frame with fixed base.

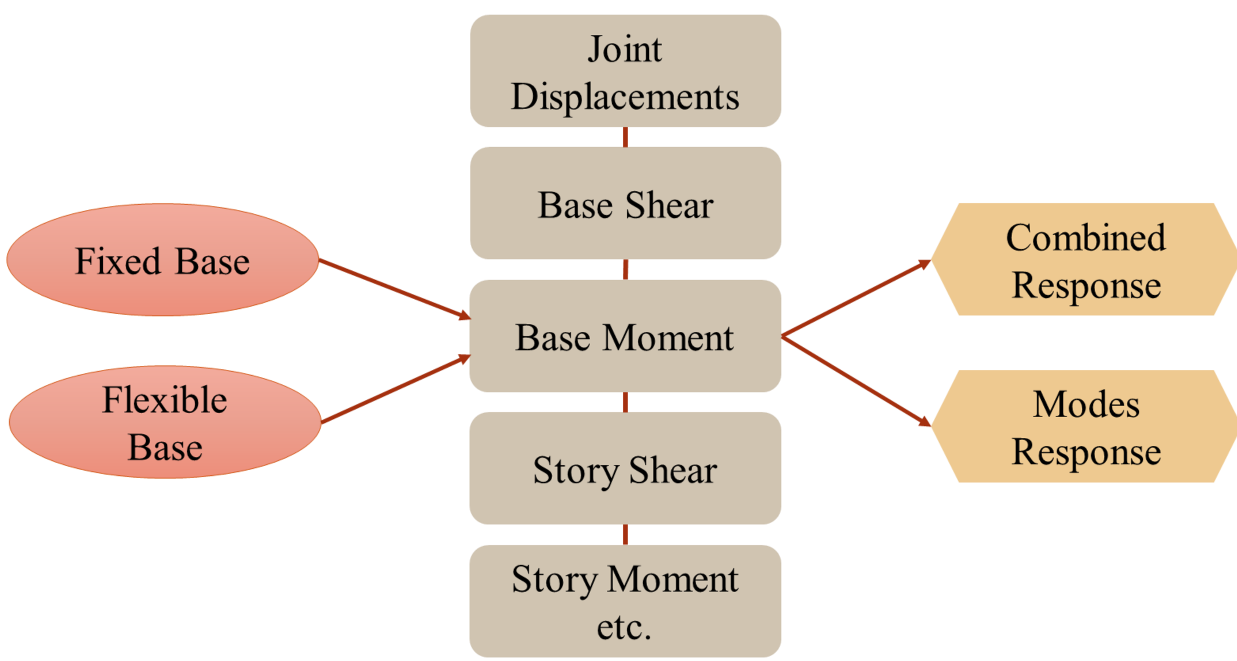


Figure 4-9: Flowchart of various results obtained from Modal Time History Analysis

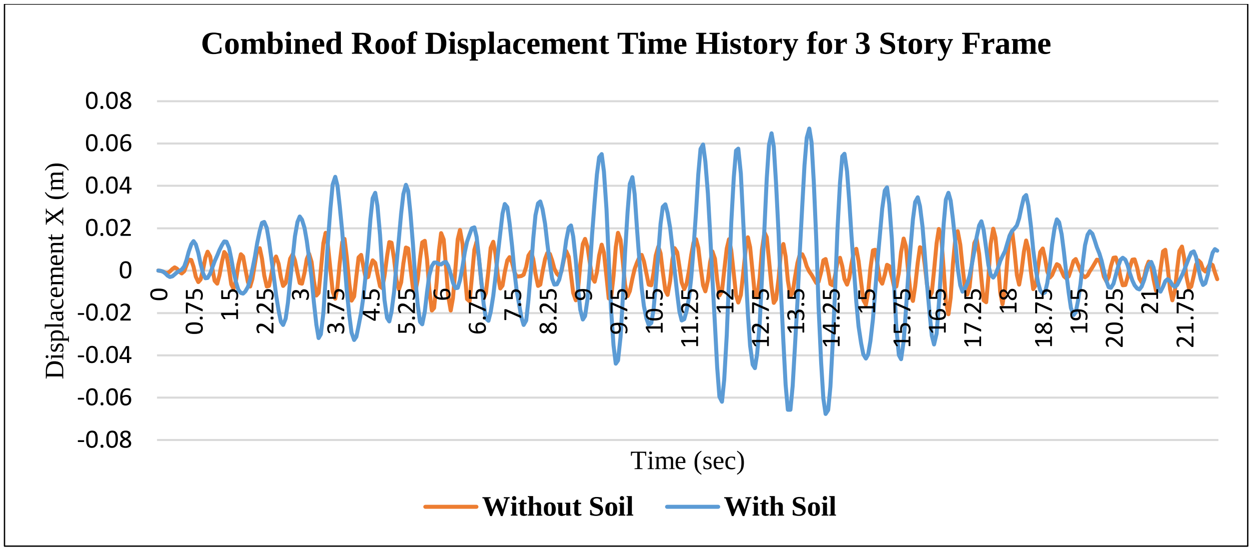


Figure 4-10: Combined roof displacement time history comparison for fixed and flexible base case for 3 story-frame

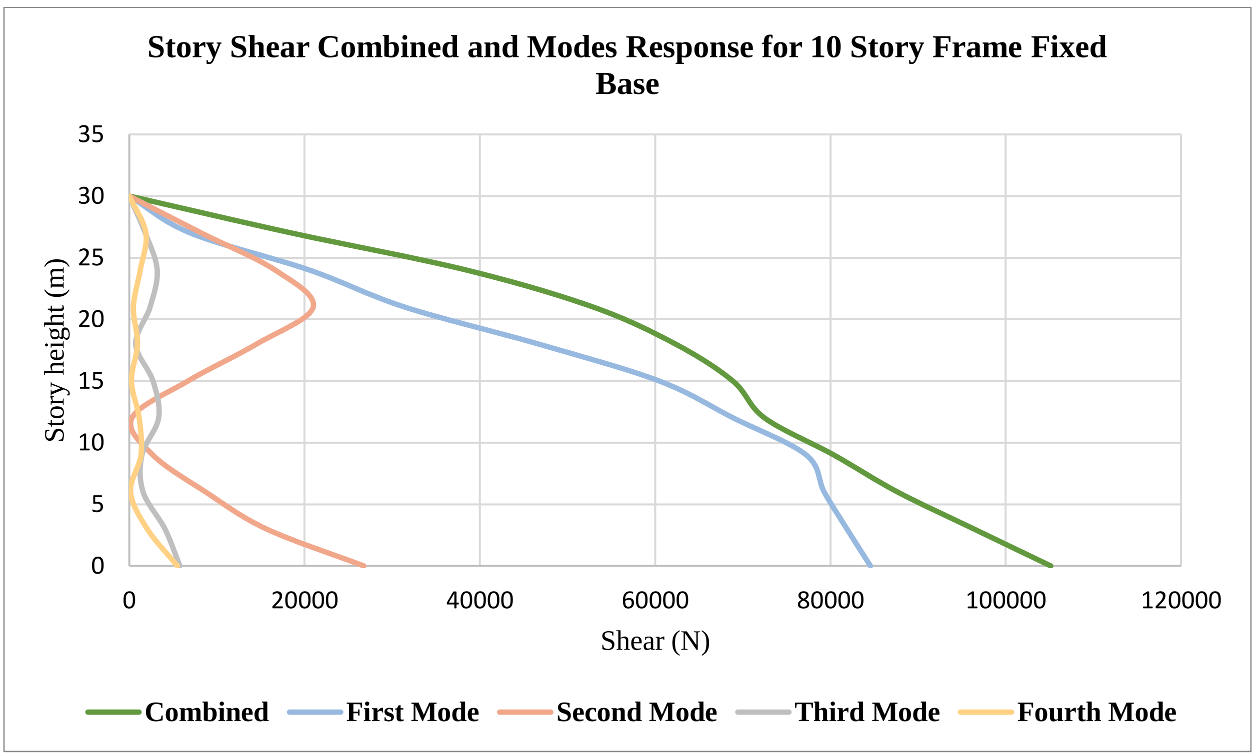


Figure 4-11: Story shear comparison between combined and modes response for 10 story fixed base frame

Now, in the next sections, higher modes effects have been quantified with no soil and with the modeling of soil for roof displacement, base moment, base shear, story shear and story moment.

4.3.1 Roof Displacement

Roof joint max displacement in horizontal x direction has been compared for the two cases of fixed and flexible base. For this purpose, for all six frames average values of maximum roof displacement in the x direction have been calculated for all the seven ground motions and can be seen below in the Table 4-1.

Table 4-1 Roof Joint Displacement Maximum Average Values

Frame Story	Fixed Base									Flexible Base								
	Comb. (m)	First Mode (m)	First Mode Contr. (%)	Second Mode (m)	Second Mode Contr. (%)	Third Mode (m)	Third Mode Cont. (%)	Fourth Mode (m)	Fourth Mode Contr.%)	Comb. (m)	First Mode (m)	First Mode Contr. (%)	Second Mode (m)	Second Mode Contr.%)	Third Mode (m)	Third Mode Contr. (%)	Fourth Mode (m)	Fourth Mode Contr. (%)
3	0.0199	0.0199	99.4464	2.8E-05	0.5872	6.6E-06	0.0455	2.1E-25	2.725E-21	0.0660	0.0659	99.8392	1.62E-20	8.727E-17	6.16E-05	0.1465	5.478E-21	3.903E-16
5	0.0533	0.0529	98.7032	0.0004	1.3055	5.8E-06	0.1893	5.3E-06	0.0201	0.0921	0.0913	99.1525	-1.84E-19	6.227E-16	0.0003	0.8037	8.398E-05	0.2169
10	0.1083	0.1038	95.5242	0.0045	4.5514	-0.0001	0.3944	0.0001	0.1958	0.1584	0.1548	97.1161	0.0036	2.9992	-3.7E-19	8.761E-15	9.568E-06	0.0132
15	0.1560	0.148	94.2754	0.008	5.6210	-0.0001	0.9771	0.0002	0.2094	0.2031	0.1959	96.4224	0.0068	3.3776	-1.8E-17	1.607E-14	6.962E-05	0.0984
20	0.1815	0.1666	90.4123	0.0137	11.0173	0.0005	2.2541	0.0009	0.5307	0.2301	0.2198	94.3111	0.0104	5.7609	1.79E-19	1.413E-15	-0.0004	0.5992
30	0.2675	0.2204	78.8493	0.0348	15.3548	0.0085	5.1729	0.0038	1.9008	0.4038	0.3649	86.1075	0.0365	12.7374	-0.0003	1.6228	9.605E-18	2.80E-14

From the table, we can see that the combined response got increased due to the incorporation of soil-structure interaction. However, if we compare the first mode contribution for fixed and flexible support, we can see that the percentage got increased with the introduction of flexible base. (Fig 4-12) Accordingly, it can be inferred that higher mode effects got decreased with taking the effect of soil and their rate of decreasing is also increasing with increasing height of the frame. According to Chopra, all modes other than first mode are higher modes. So, based on first mode contribution percentage in the combined response, the higher modes effect can be quantified. If the percentage of first mode contribution increases in the combined response, then the other modes contribution will be decreased and vice versa. In roof displacement, though the difference in first mode contribution percentage of fixed and flexible case for 3 story-frame is not much i.e., 99.44% and 99.83%. However, as we move from 3 story-frame to 30 story-frame it can be clearly seen that the difference is increasing, as in 30 story-frame it is 78.84% and 86.10%. Accordingly, the incorporation of soil has affected the higher modes effect in roof joint response, and it is becoming larger and larger as the height of frame is increasing. This trend can also be seen with the help of graph in Fig 4-12.

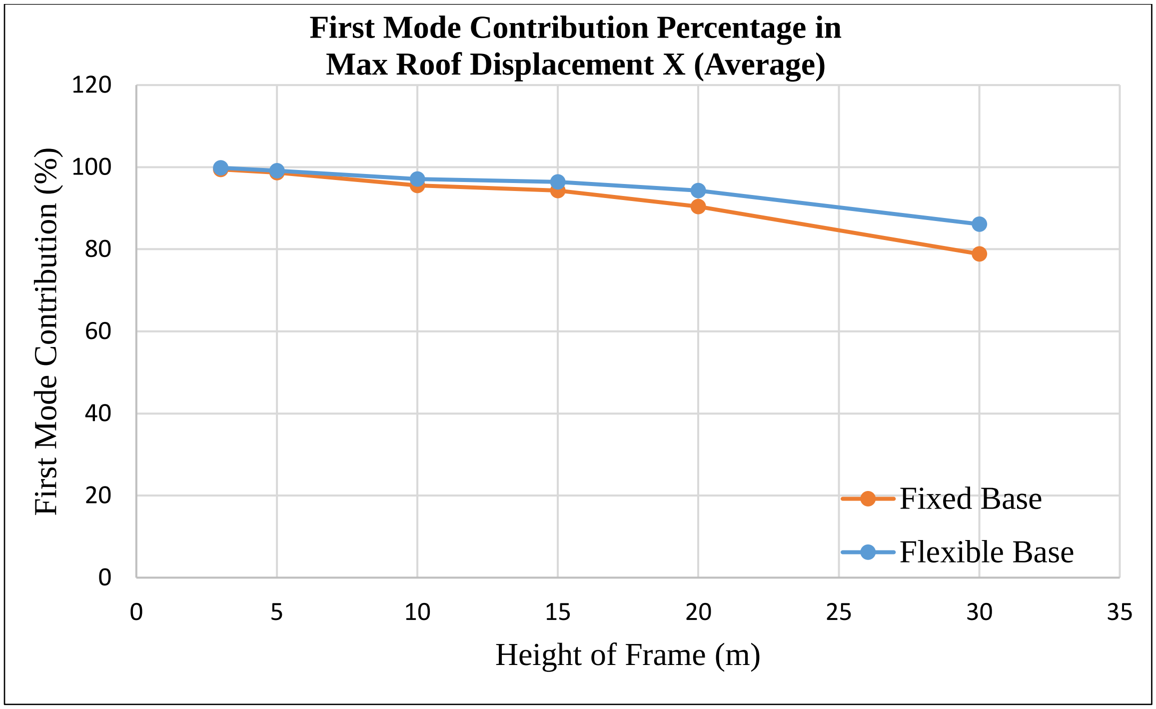


Figure 4-12: First mode contribution percentage comparison for max roof displacement for fixed and flexible base case

4.3.2 Base Moment

Base moment has the same trend as that of roof displacement. The percentage contribution of first mode in the combined average response of base moment for all the six frames has been shown in Table 4-2. It is evident that for flexible base case, the contribution from first mode is more as compared to the corresponding fixed base case. i.e., For 3 story-frame, it is 96.28% and 99.54% for fixed and flexible case, respectively and for 30 story-frame it is 21.94% and 50.05%.

Table 4-2 Base Moment Maximum Average Values

Frame Story	Fixed Base									Flexible Base								
	Comb. (N-m)	First Mode (N-m)	First Mode Contr. (%)	Second Mode (N-m)	Second Mode Contr. (%)	Third Mode (N-m)	Third Mode Contr. (%)	Fourth Mode (N-m)	Fourth Mode Contr. (%)	Comb. (N-m)	First Mode (N-m)	First Mode Contr. (%)	Second Mode (N-m)	Second Mode Contr. (%)	Third Mode (N-m)	Third Mode Contr. (%)	Fourth Mode (N-m)	Fourth Mode Contr. (%)
3	51745.06	50003.00	96.288	1325.9331	2.9954	416.12	0.7906	-1.49E-30	3.59E-33	2592.83	2597.07	99.549	-1.15E-23	1.565E-24	-6.124186	0.3652	2.670E-25	4.737E-25
5	183553.62	172862.79	93.254	9817.8362	6.4451	37.561	0.9051	424.896	0.4207	9427.46	9430.11	98.659	-6.2E-22	2.425E-23	-2.690129	1.1448	-5.8873	0.3119
10	299941.24	235043.27	79.971	42245.944	14.1753	9356.3	3.3085	4052.552	1.9554	31978.6	28529.1	88.982	3311.768	10.5902	8.946E-20	5.07E-22	18.6662	0.1096
15	630955.18	286507.40	45.401	295575.597	46.9381	23317.1	4.4920	9079.193	2.0443	124897.4	87209.2	70.060	37607.60	29.6492	5.486E-19	6.75E-22	-30.9363	0.0419
20	746561.50	274958.91	36.122	363023.623	49.4797	79684.3	11.5067	11829.47	1.9963	204688.2	122971.3	58.662	80322.74	40.1285	2.753E-20	7.55E-23	1144.0447	2.7474
30	1187155.0	260542.69	21.946	457238.779	38.5155	338063.	28.4767	63845.29	5.3780	501527.4	251047.7	50.056	202900.6	40.4565	26235.934	5.2312	1.031E-18	2.056E-22

Accordingly, it can be concluded that the incorporation of soil-structure interaction will help in decreasing the higher mode effects in base moment and the difference is becoming more and more with the increasing height of frame. This trend can also be seen in Fig 4-13, where it is noticeable that the difference is small for low heights frames, and it goes on increasing with the increasing height of the frame.

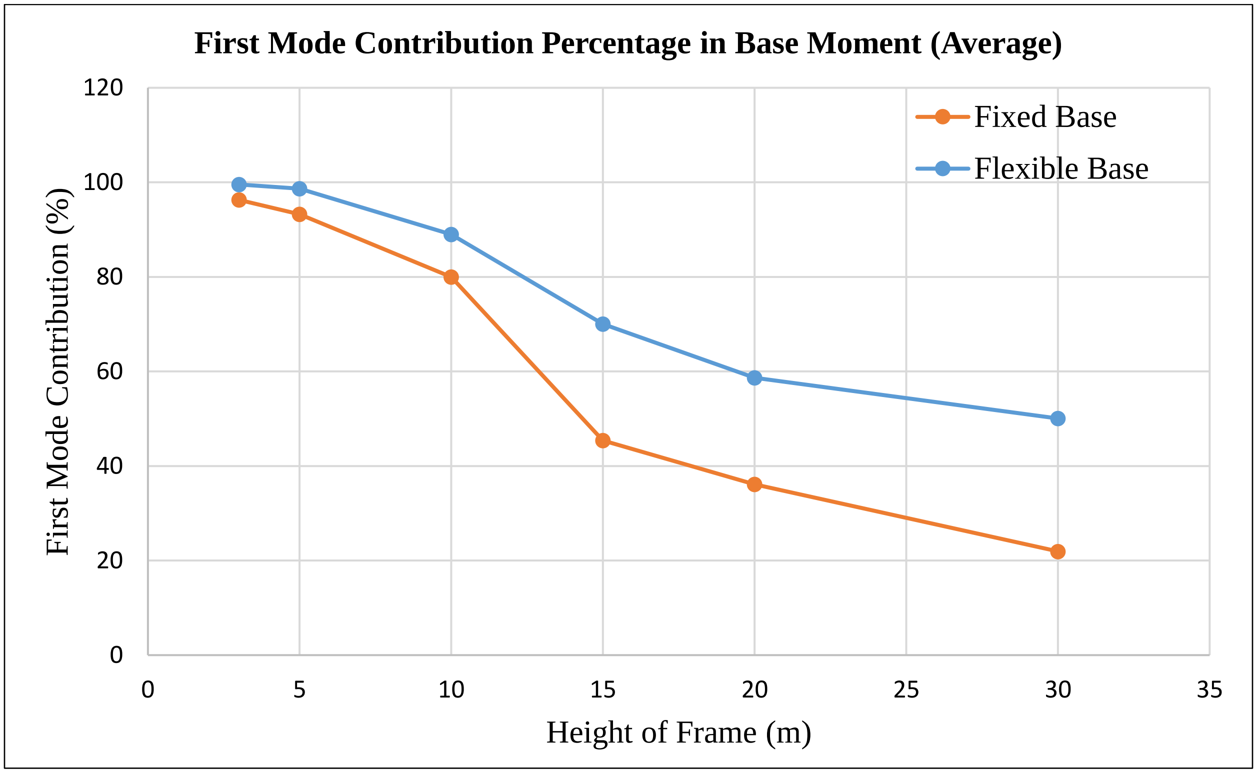


Figure 4-13: First mode contribution percentage comparison for base moment for fixed and flexible base case

4.3.3 Base Shear

For base shear, interesting results can be seen in the Table 4-3. It is noticeable that up to 20 story, higher modes effect got increased and their rate is also increasing but after that their effect starts decreasing as compared to the higher modes effect in fixed base apart from the 3 story-frame where the contribution percentages are nearly equal for first mode contribution. So, it can be concluded that for short story frame (small fundamental time periods) higher mode effects will increase in the base shear and they keep on increasing but after reaching the certain limit they start decreasing. Accordingly, for large period frames (30 story) with flexible base, higher mode effects will be less in the base shear if we compare it from the corresponding frame with fixed base.

Table 4-3 Base Shear Maximum Average Values

Frame Story	Fixed Base									Flexible Base								
	Comb. (N)	First Mode (N)	First Mode Contr. (%)	Second Mode (N)	Second Mode Contr. (%)	Third Mode (N)	Third Mode Contr. (%)	Fourth Mode (N)	Fourth Mode Contr. (%)	Comb. (N)	First Mode (N)	First Mode Contr. (%)	Second Mode (N)	Second Mode Contr. (%)	Third Mode (N)	Third Mode Contr. (%)	Fourth Mode (N)	Fourth Mode Contr. (%)
3	25162.9	24312.89	96.565	719.207	3.1494	130.805	0.5209	-1.052E-30	5.72E-33	21159.55	20990.34	98.0325	-4.1571E-23	1.52E-24	142.5991	2.0536	3.4E-24	5.66E-26
5	62412.6	58507.18	92.706	2617.80	5.0097	571.753	1.3821	306.3978	0.5951	42375.27	38414.71	89.4800	5.4585E-21	2.57E-23	3708.4788	9.8539	62.3407	0.6442
10	102152	64739.31	64.378	19389.8	18.323	5827.05	5.7541	2845.1635	4.0210	89686.75	46445.91	51.0667	40713.2761	45.7587	1.55E-18	1.76E-21	137.166	0.2235
15	249821.7	130584.3	53.358	93657.9	36.555	9208.02	6.6050	2817.2457	2.2499	188887.4	64920.59	30.4920	114485.4002	64.7764	-2.184E-18	1.86E-21	660.863	0.5500
20	308582.3	100929.9	29.698	158720.6	52.540	26006.3	9.3272	11141.5804	4.968	275078.7	83159.2	25.8825	171704.6993	66.6310	1.045E-19	5.43E-23	9308.25	4.6846
30	360848.2	59469.9	19.985	143610.1	43.621	103694.6	29.3667	19527.6053	6.4358	346504.4	132206.9	36.1890	116930.9312	34.1210	46440.8250	13.7494	6.09E-18	1.79E-21

The same trend has also been shown with the help of graph in Fig 4-14.

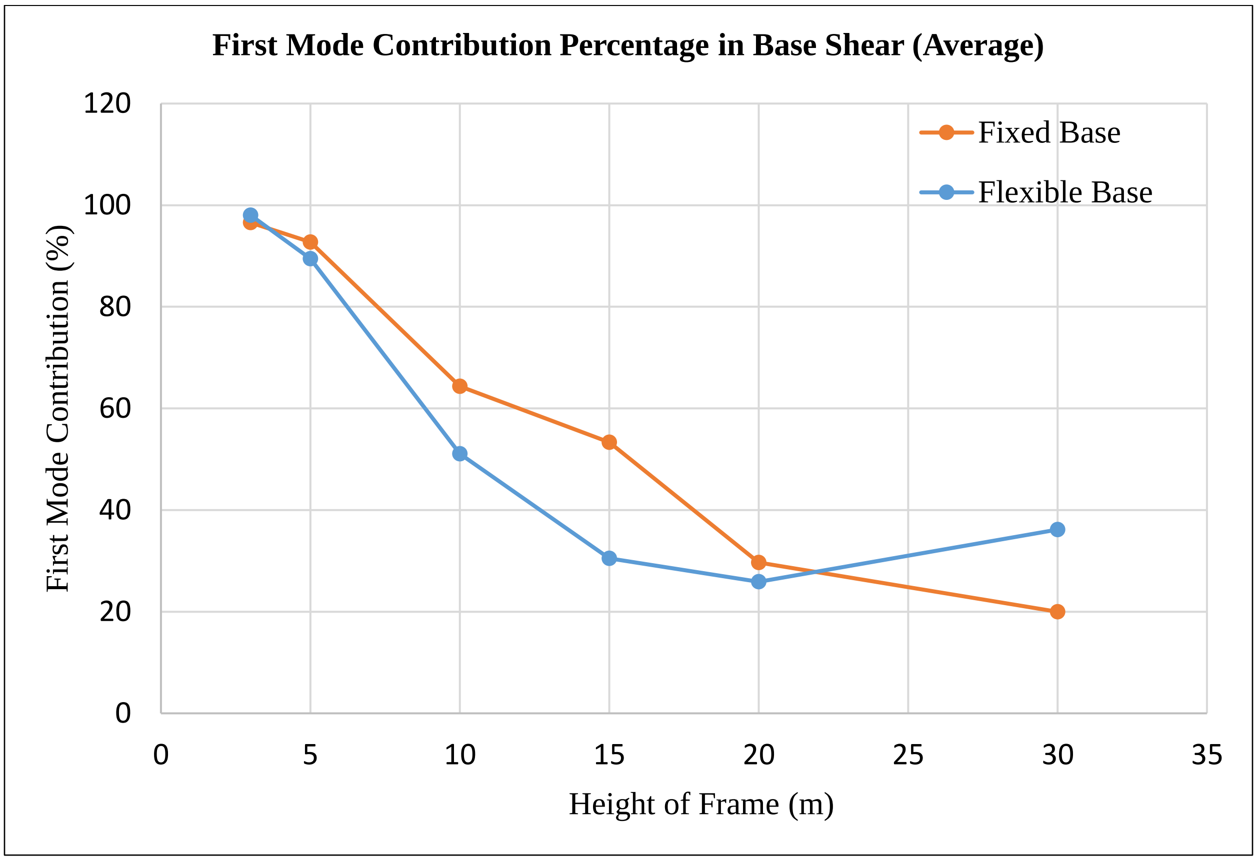


Figure 4-14: First mode contribution percentage comparison for base shear for fixed and flexible base case

4.3.4 Story Shear

Incorporation of soil also affected the higher modes effect in story shear. First mode contribution percentage (average) for fixed and flexible base case for all the six frames can be seen in Fig 4-15. The trend of increasing or decreasing percentage contribution for the first mode in the combined response is not same in all the frames. However, for most of the frames, it is evident that the percentage contribution has got increased in the mid to bottom stories except from the 5 story-frame. Moreover, for 30 story-frame it is noticeable that the contribution percentage from the first mode has got increased for all the stories in flexible base case as compared to that from the fixed base.

4.3.5 Story Moment

Higher mode effects also got changed with the introduction of flexible base in story moments response. Fig 4-16 shows the average first mode contribution percentage in the combined response of story moment for all the seven ground motions. The trend is somehow similar to that of story shear trend. For 3 to 30 story-frame, percentage contribution from first mode has been increased in the mid to bottom stories except for 5 and 10 story-frame. In 30 story-frame first mode contribution percentage has been increased for almost all the stories. At the end, it can be concluded that the introduction of base flexibility significantly affected the higher modes effect in story moment too.

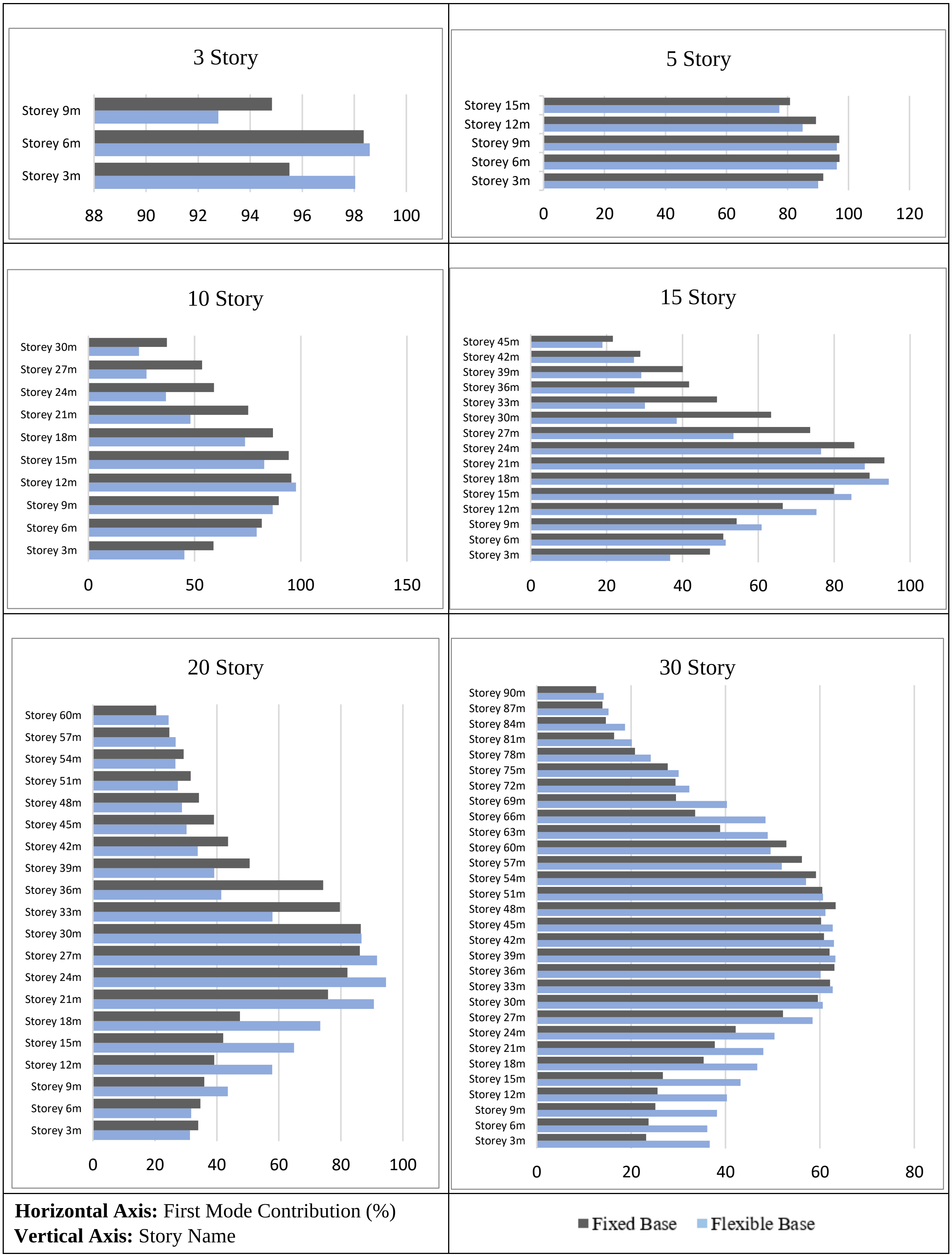


Figure 4-15: Story shear first mode contribution percentage for fixed and flexible base case

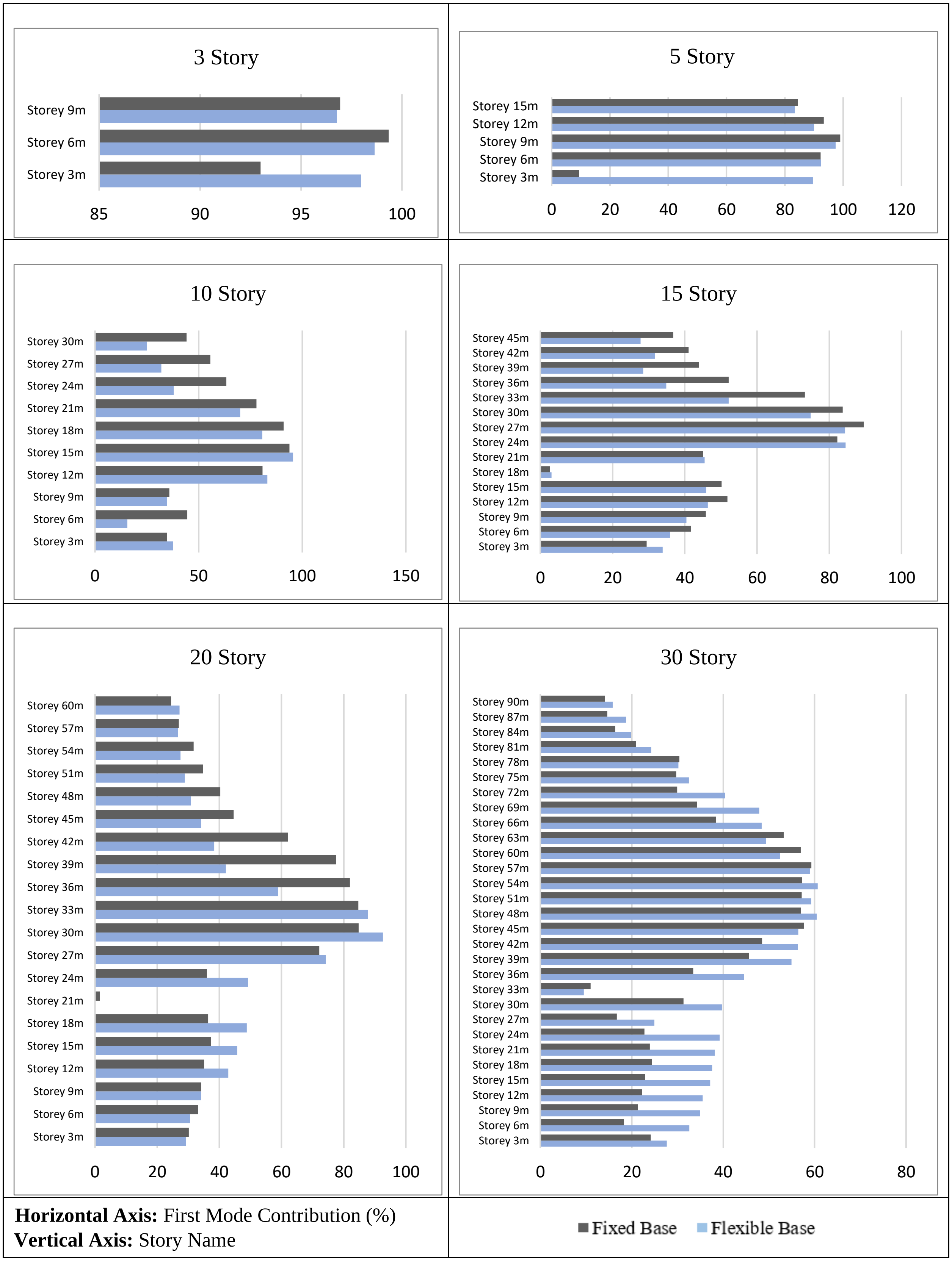


Figure 4-16: Story moment first mode contribution percentage for fixed and flexible base case

Chapter 5

Conclusions and Recommendations

5.1 Conclusions

Effect of soil-structure interaction on higher mode effects have been studied for six generic two-dimensional frames subjected to seven ground motions. For this purpose, structural properties like time-periods, mode shapes and dynamic responses like roof displacement, base shear, base moment, story shear and story moment have been compared thoroughly for the cases of fixed and flexible base to quantify the higher mode effects in these responses. Following conclusions have been made in accordance with this study.

First mode time-period got increased for all the six frames with the incorporation of soil. However, the difference between fixed and flexible base T_1 is becoming small and small as the height of the frame is increased which is also logical in the sense that as the height of frame is increased, global stiffness also increases and the reduction in global stiffness with the introduction of base flexibility is not much and hence structural flexibility dominates the base flexibility and there is not much difference in the time-periods of fixed and flexible base case. The same trend has been observed for the higher modes time-periods too.

Mode shapes also got changed with the introduction of flexible base. Just like time-periods, difference in mode shapes tends to decrease with the increasing height of the frame and the justification is also same that for short story frame, base flexibility is dominating and with increasing height structural flexibility starts dominating the base flexibility (soil stiffness) and the difference is becoming smaller and smaller between the mode shapes of fixed and flexible base case.

Maximum roof displacement and the higher mode effects in this response have been thoroughly analyzed for the two cases of fixed and flexible base. It has been observed that with taking the effect of soil, higher modes contribution got decreased as compared to that in fixed base case in the maximum roof displacement response. Moreover, the rate of increasing of first mode contribution in the combined response is also increasing going from three to thirty story frame. Hence, it can be concluded that base flexibility has affected the higher mode effects on the roof displacement since higher modes contribution percentage got changed with taking the effect of soil.

For base moment, the results are same as that in the case of roof displacement. However, the difference between flexible and fixed base first mode contribution percentage is much large as compared to that in roof joint displacement. For 3 story-frame, the difference between the fixed and flexible first mode contribution percentage is very less, and it goes on increasing with the increasing height of frame, since for 30 story-frame, the percentage contributions are 21.94% and 50.05% from the first mode for fixed and flexible base case respectively. Thus, it is predictable that with increasing height the higher mode effects will be affected much in the base moment as the incorporation of soil results in the changed higher modes contribution to the combined response.

For base shear, interesting results have been seen. Up to twenty story-frame, the higher modes effect has been increased as compared to the effects in frames with fixed base and then they tend to decrease. And for thirty-story frame the higher modes effect got decreased as compared to that of thirty-story frame with fixed base. And with this trend, it can be predicted that if the story number is increased more, the higher modes effect decreasing percentage between fixed and flexible base case will become more and hence it can be concluded that tall frames or frames with large fundamental period having flexible base will have less contribution from the higher modes if compared with their corresponding frame with fixed base. Accordingly, it is inferred that with the introduction of soil, higher mode effects have been changed in base shear.

For story shear and story moment, the incorporation of soil effects with the introduction of base flexibility has also resulted in the changed higher modes contribution to the combined response as compared to the higher modes contribution of the frames with fixed base. However, the contribution percentage trend is not same for all the frames.

5.2 Recommendations

- Different soil types should be taken for quantifying the higher mode effects with the incorporation of soil-structure interaction.
- The parametric studies should be extended to three-dimensional frames.
- Other soil modeling technique (i.e., direct analysis approach) can also be used and then the results can be compared.
- Column to beam stiffness ratio should also be taken into consideration while selecting the generic frames.
- Response 2D program should be extended to 3D structural analysis software.

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Appendices

Appendix A: An Example for the Footing Design

The following example elaborates the footing design procedure for 20 story-frame.

Axial Load (P) = 522.122kN (From Response 2D)

Length of Column (L_{col}) = 0.381m

Width of Column (B_{col}) = 0.381m

Soil Bearing Capacity (SBC) = 95.76kN/m^2

Concrete Compressive Strength (fck) = 20000kN/m^2

Area of Footing

Factored Axial Load (P_{fac}) $= 1.1 \times \text{P} = 574.3342\text{kN}$

Area of Footing $= \frac{P_{fac}}{SBC} = 5.9976\text{m}^2$

Length or Width of Footing $= \sqrt{\text{Area of footing}} = 2.449\text{m}$

Soil Reaction

Ultimate Load (P_u) $= 1.5 \times P_{fac} = 861.5013\text{kN}$

Ultimate Soil Reaction (q_u) $= \frac{P_u}{\text{Area of footing}} = 143.64\text{kN/m}^2$

Factored Moment

Ultimate Moment (M_u) $= \frac{q_u \times B \times (B - B_{col})^2}{8} = 188.053\text{kNm}$

Required Depth of Footing

Depth (D) $= \sqrt{\frac{M_u}{0.138 \times \text{fck} \times B}} = 0.1668\text{m}$

Hence the width and length of footing are 2.449m and its depth is 0.1668m.

Appendix B: An Example for the Calculation of Soil Spring Stiffness

The following example elaborates the calculation of soil spring stiffnesses for 5 story-frame.

The dimensions of square footing calculated from the design procedure are as follows:

Length of footing = 2L = 0.47205m

Width of footing = 2B = 0.47205m

Depth of footing = D = 0.0515m

For Soil Type D, we have:

Shear modulus = G = 367000N/m^2

Poisson's ratio = ν = 0.4

Now, from Eq 2.10, 2.11 and 2.12 we have:

$$K_{x,sur} = \frac{GB}{2-\nu}\left[6.8\left(\frac{L}{B}\right)^{0.65} + 2.4\right]$$

$$K_{z,sur} = \frac{GB}{1-\nu}\left[3.1\left(\frac{L}{B}\right)^{0.75} + 1.6\right]$$

$$K_{yy,sur} = \frac{GB^3}{1-\nu}\left[3.73\left(\frac{L}{B}\right)^{2.4} + 0.27\right]$$

So, we have:

$K_{x,sur} = 996143.5125 N/m$; $K_{z,sur} = 1357065.075 N/m$; $K_{yy,sur} = 257358.6745 Nm/rad$

Now from Eq 2.13, 2.14 and 2.15, we have embedment correction factors as follows:

$$\eta_x = \left[1.0 + \left(0.33 + \frac{1.34}{1 + L/B}\right)\left(\frac{D}{B}\right)^{0.8}\right]$$

$$\eta_z = \left[1.0 + \left(0.25 + \frac{0.25}{L/B}\right)\left(\frac{D}{B}\right)^{0.8}\right]$$

$$\eta_{yy} = \left[1.0 + \frac{D}{B} + \left(\frac{1.6}{0.35 + (L/B)^4}\right)\left(\frac{D}{B}\right)^{2}\right]$$

Therefore, the calculated embedment correction factor values are:

$$\eta_x = 1.1699;\ \eta_z = 1.0849;\ \eta_{yy} = 1.1232$$

The final stiffness values calculated by the multiplication of stiffness values and the embedment correction factors are as follows:

$K_x = K_{x,sur} \times \eta_x = 996143.5125 \times 1.1699 = 1165412.46 N/m$

$K_z = K_{z,sur} \times \eta_z = 1357065.075 \times 1.0849 = 1472364.213 N/m$

$K_{yy} = K_{yy,sur} \times \eta_{yy} = 257358.6745 \times 1.1232 = 289066.624 Nm/rad$

Appendix C: Modes Shape Comparison for Fixed and Flexible Base Case

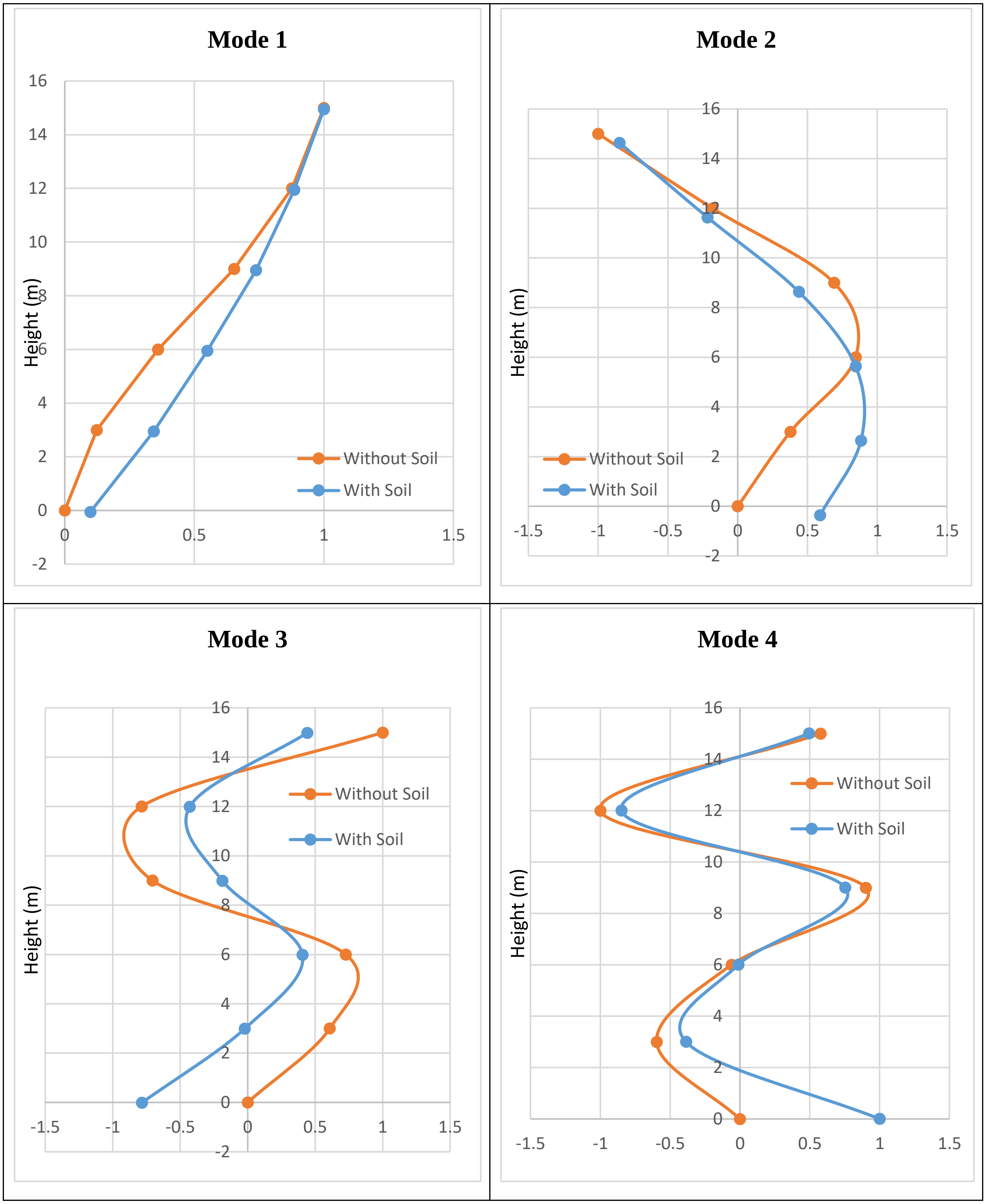

Figure C-1: Modes shapes comparison for 5 story-frame for fixed and flexible base case

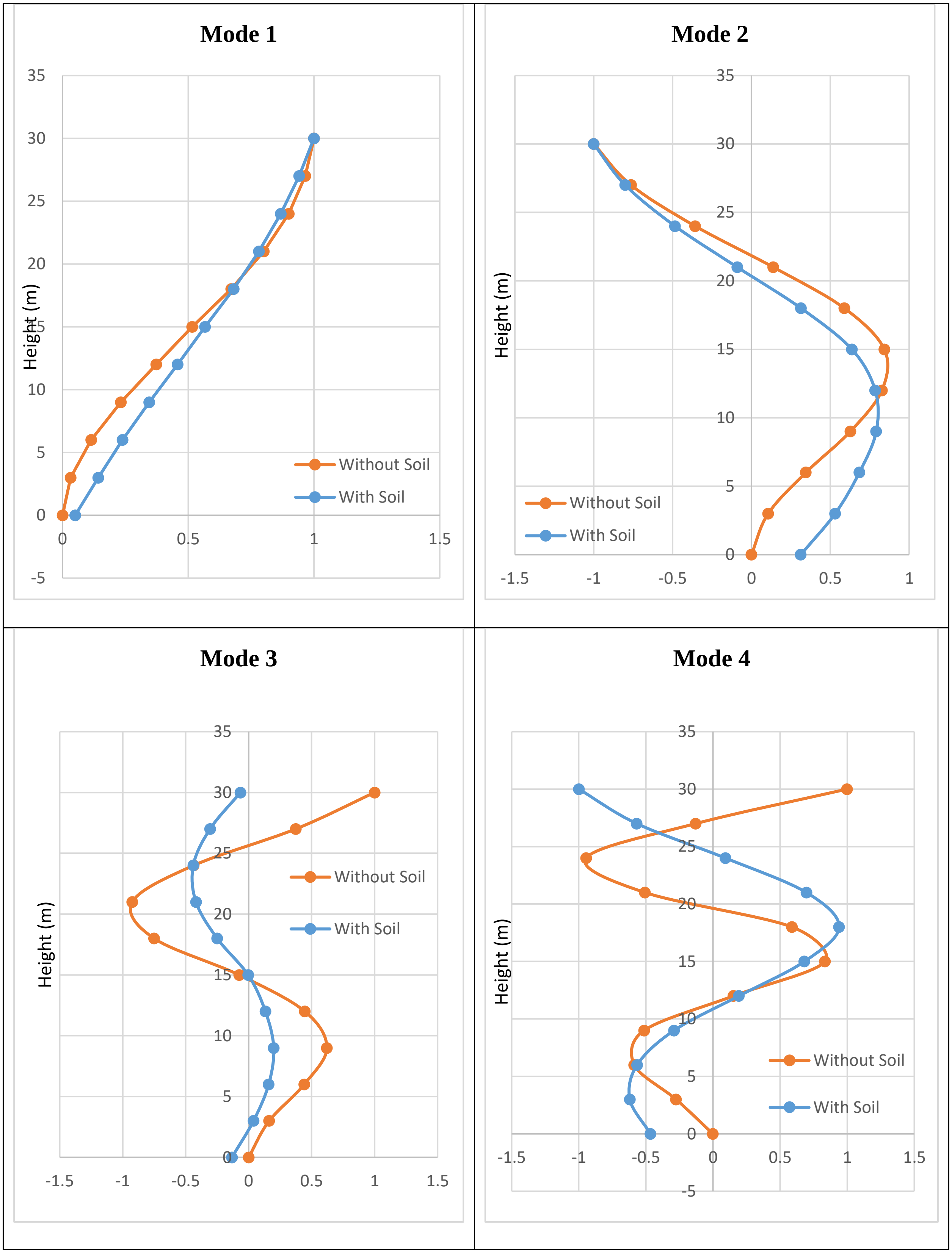

Figure C-2: Modes shapes comparison for 10 story-frame for fixed and flexible base case

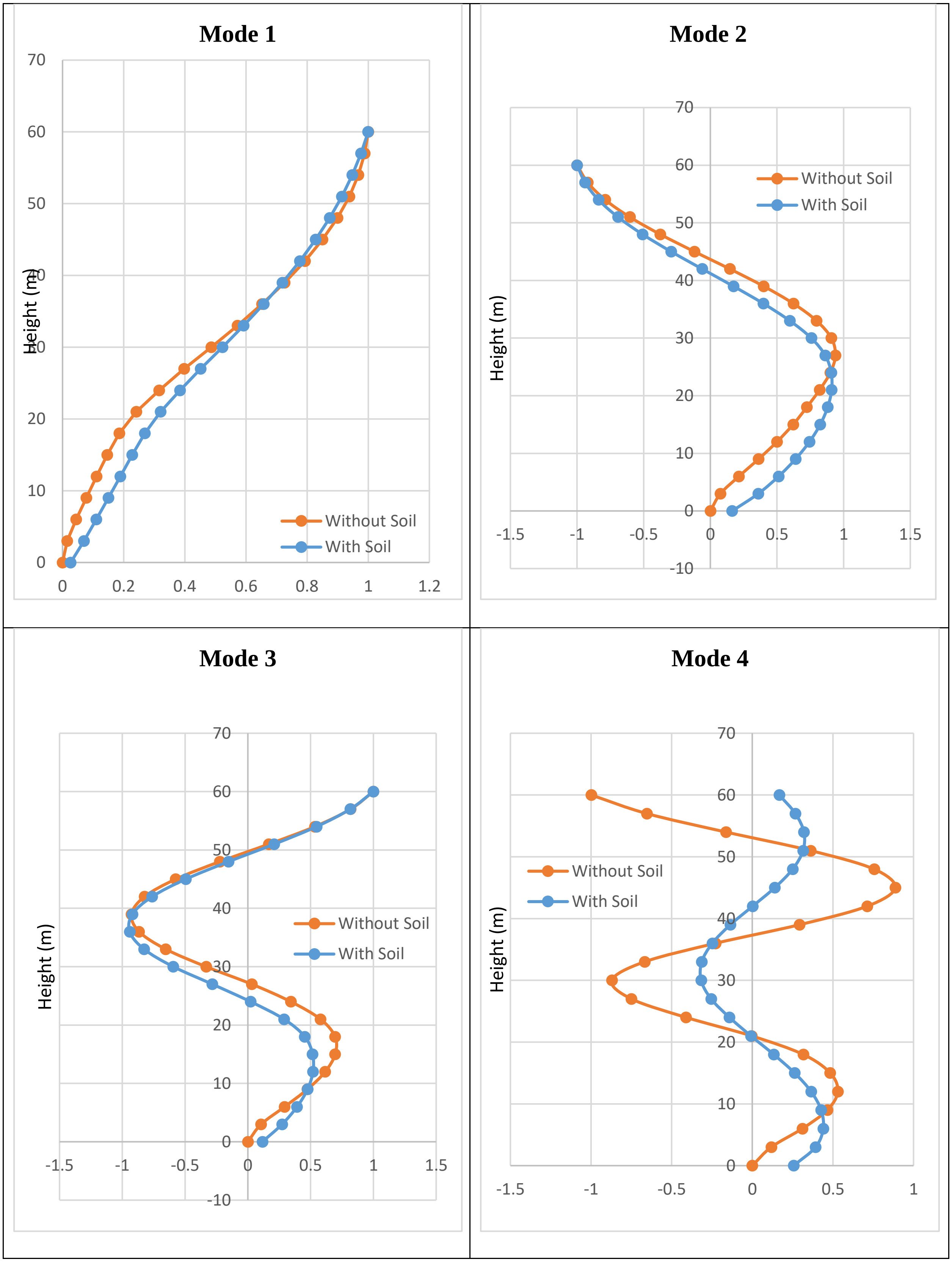

Figure C-3: Modes shapes comparison for 20 story-frame for fixed and flexible base case

Appendix D: Flowcharts of Analysis Procedures Algorithms

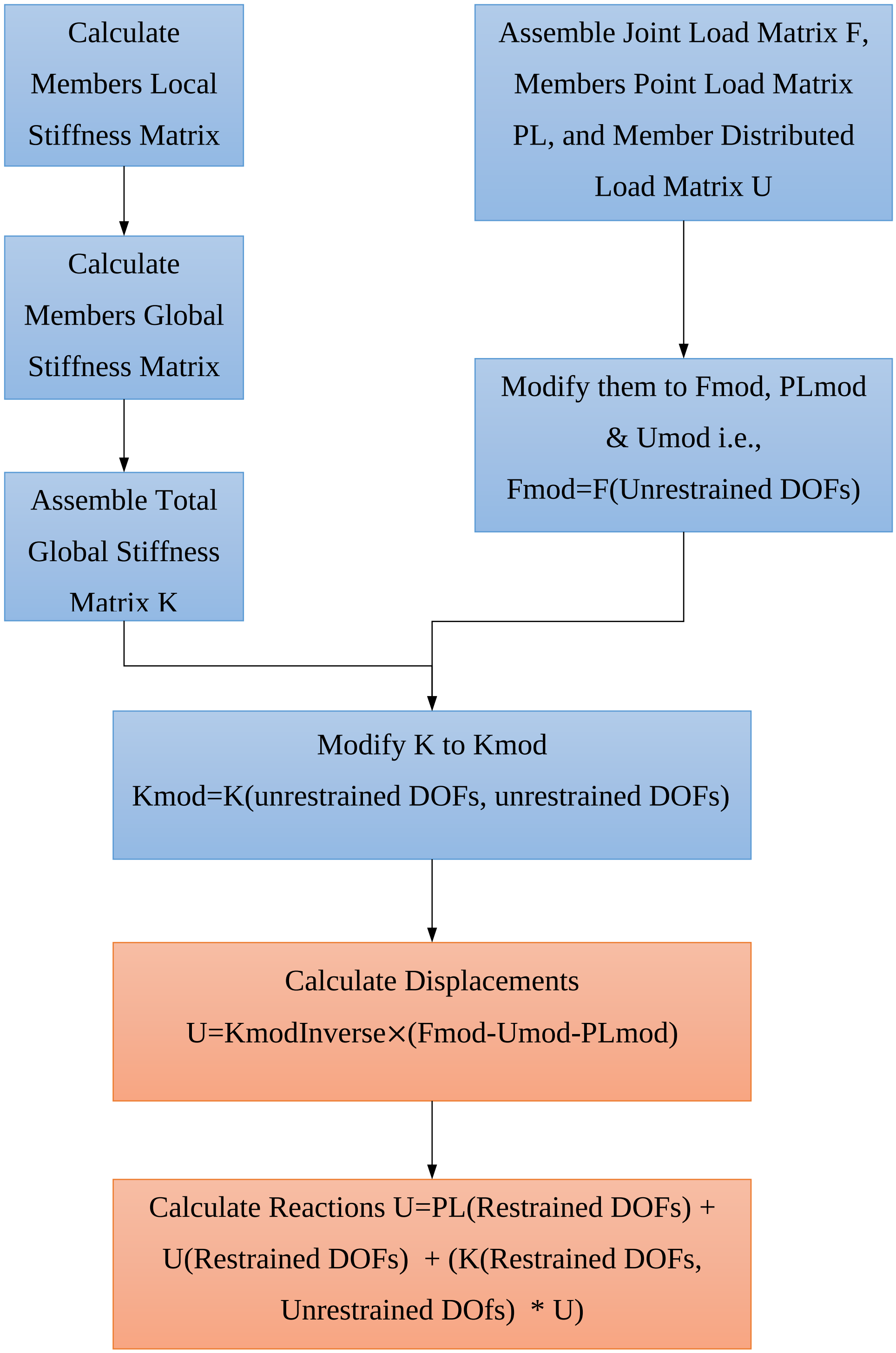

Figure D-1: Flow chart algorithms of liner static analysis (Stiffness Method)

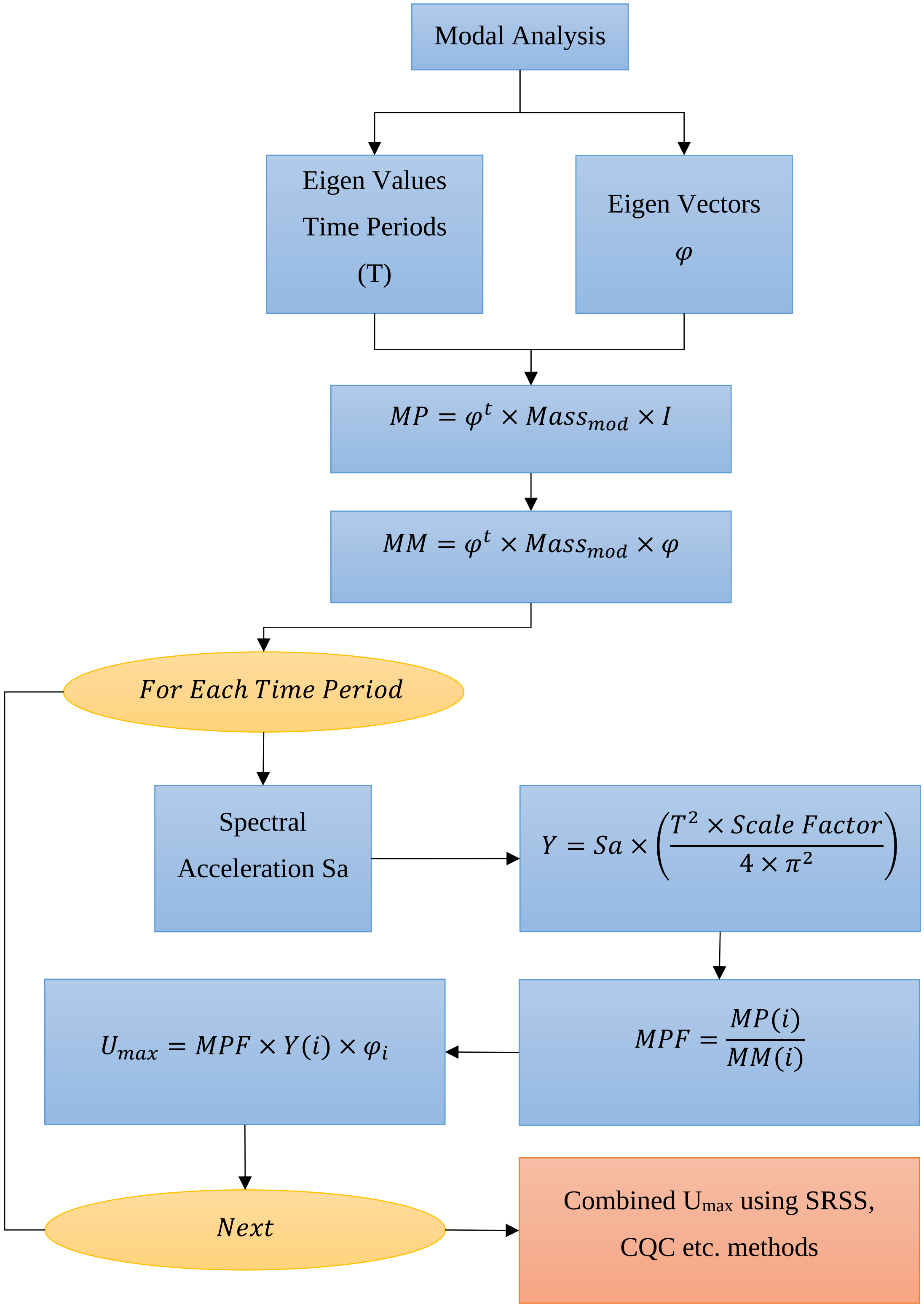

Figure D-2: Flow chart algorithms of response spectrum analysis

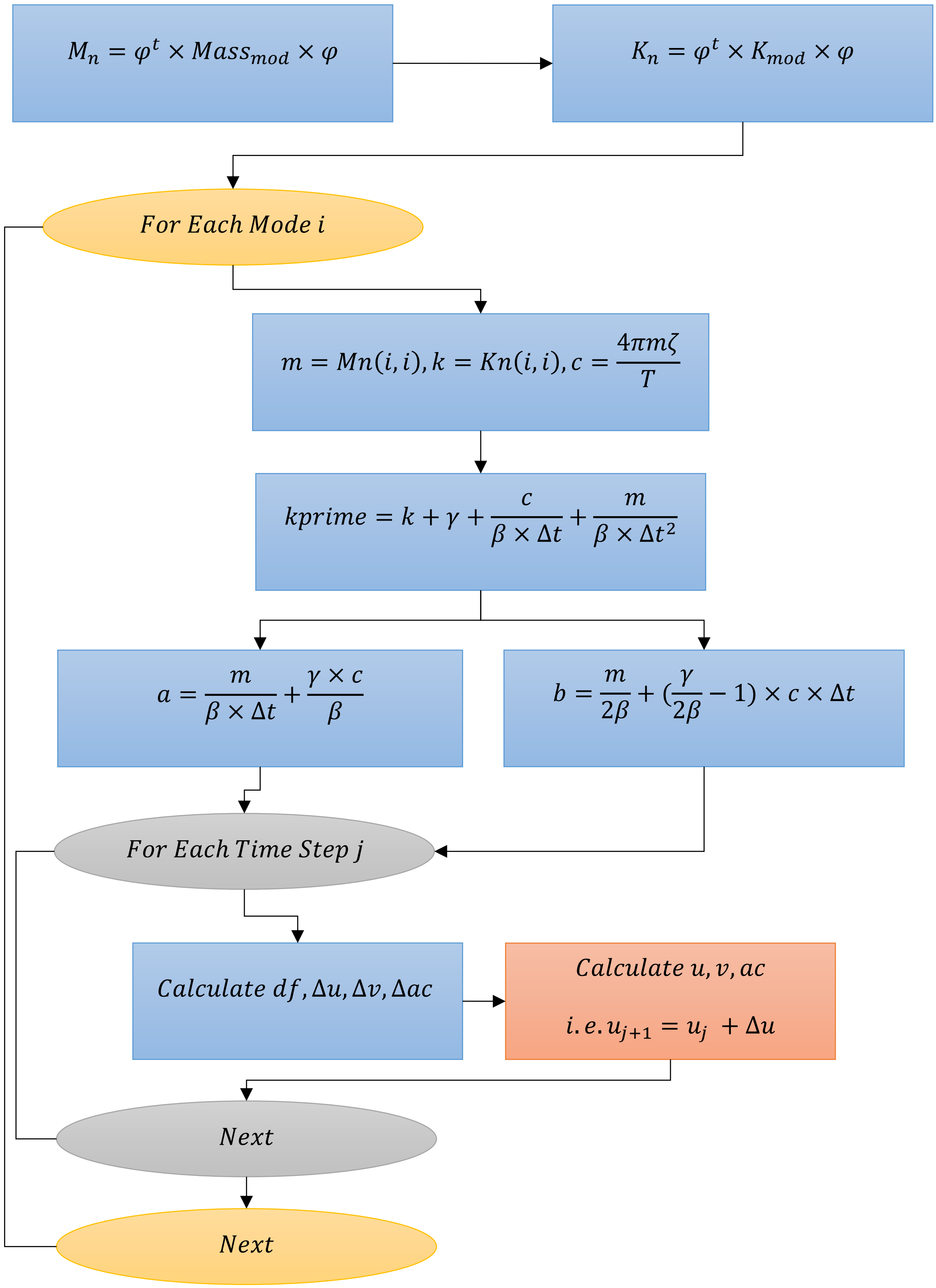

Figure D-3: Flow chart algorithms of modal time history analysis

Appendix E: Selected Ground Motions Details

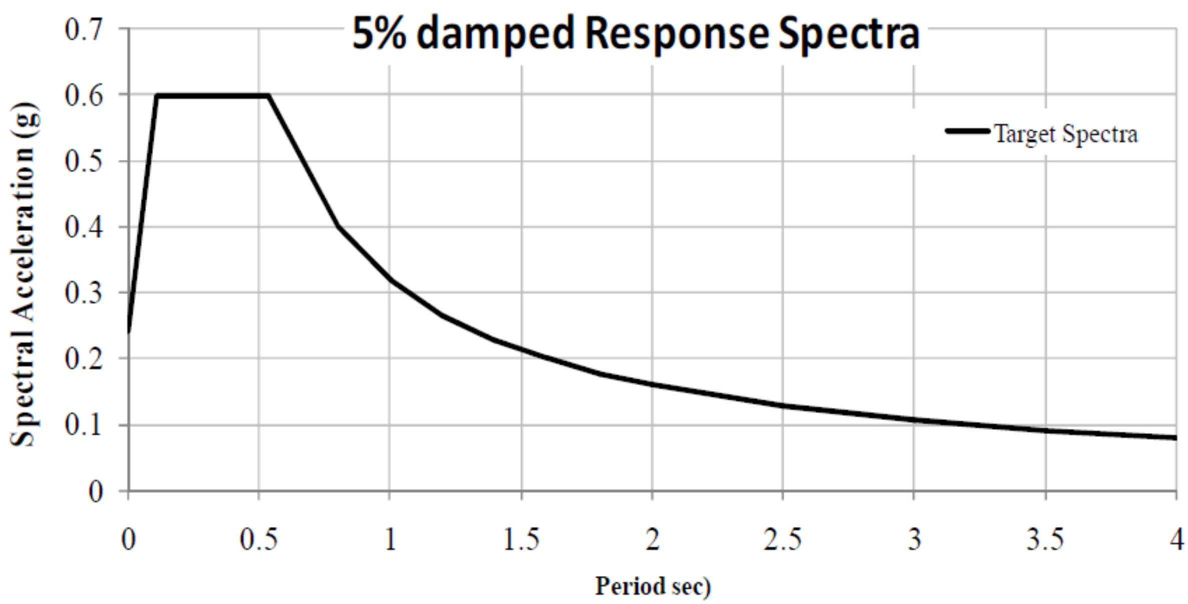

Figure E-1: Target response spectrum to which ground motions spectra is matched

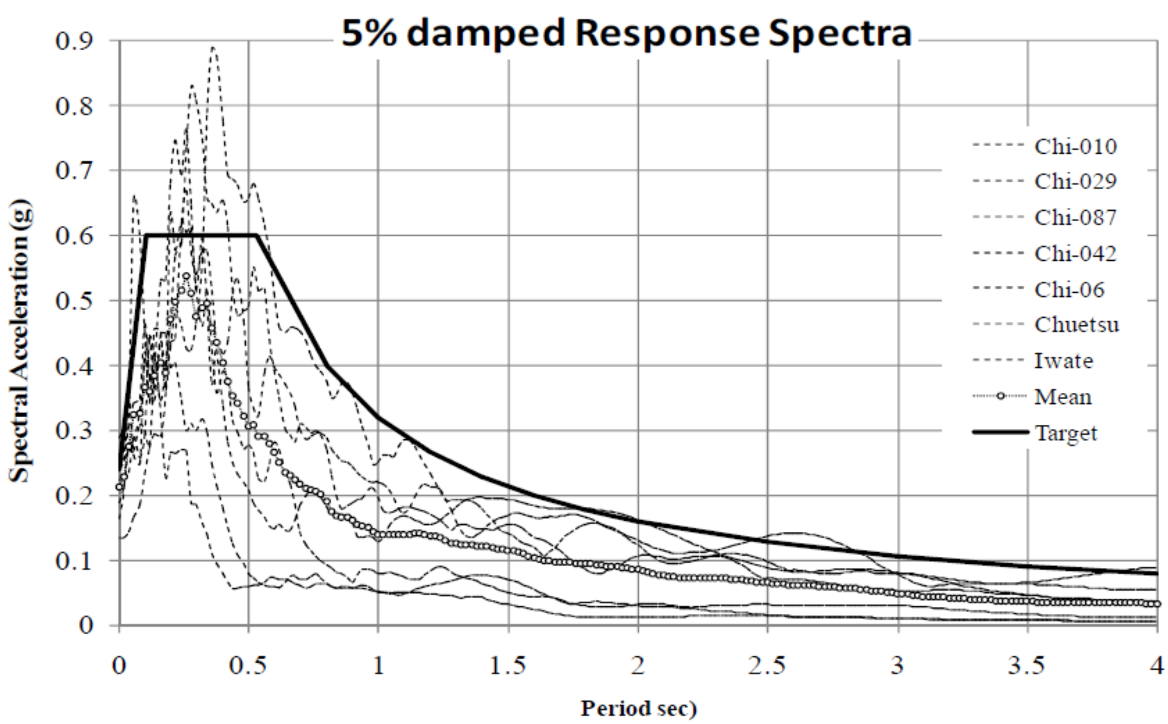

Figure E-2: The unmatched acceleration response spectra of ground motions

Table E-1: Ground motion parameters of time histories used in this study

Parameter	Direction	GM-1	GM-2	GM-3	GM-4	GM-5	GM-6	GM-7
PGA (g)	H1 H2	0.174 0.224	0.289 0.237	0.135 0.127	0.252 0.212	0.242 0.138	0.189 0.119	0.165 0.202
Max Velocity (cm/s)	H1 H2	24.21 18.52	35.25 39.71	10.23 14.11	37.03 36.17	22.09 12.94	13.46 11.71	12.19 12.5
Max Displacement (cm)	H1 H2	9.86 9.19	11.11 25.67	7.67 8.05	38.60 21.93	8.49 6.95	6.46 7.36	6.83 7.05
Arias Intensity (m/s)	H1 H2	0.664 0.715	0.796 0.87	0.576 0.425	1.139 0.924	0.26 0.11	0.31 0.20	0.429 0.402
Specific Energy Density (cm2/s)	H1 H2	958.6 798.4	2134 5915	424.8 452.3	7255 5397	878.7 396.4	383.3 278.1	526.1 851.8
Cumulative absolute velocity (cm/s)	H1 H2	1038 1005	1067 1082	969.9 857.7	1215 1152	450.3 354.8	516.7 447.3	602.2 631.4
Housner Intensity (cm)	H1 H2	71.52 75.35	116.7 125.3	43.17 49.65	102.5 100.7	91.81 45.19	53.02 44.23	42.16 40.62
Sustained max acceleration (g)	H1 H2	0.146 0.182	0.17 0.198	0.13 0.108	0.174 0.169	0.125 0.06	0.154 0.092	0.147 0.138
Effective Design acceleration (g)	H1 H2	0.174 0.223	0.28 0.235	0.135 0.126	0.251 0.211	0.241 0.132	0.181 0.120	0.166 0.200
A-95 parameter	H1 H2	0.172 0.221	0.286 0.234	0.132 0.128	0.249 0.209	0.24 0.137	0.186 0.116	0.162 0.198
Predominant period (s)	H1 H2	0.44 0.28	0.46 0.62	0.26 0.2	0.32 0.26	0.32 0.18	0.42 0.22	0.16 0.16
Mean period (sec)	H1 H2	0.6 0.56	0.88 0.999	0.46 0.5	0.63 0.67	1.02 1.02	0.61 0.55	0.42 0.55
Damage index	H1 H2	0.827 0.98	0.628 0.592	0.82 0.645	1.326 1.029	0.242 0.107	0.399 0.312	0.893 0.845
No of effective cycles	H1 H2	4.85 3.53	1.48 1.40	6.38 5.14	6.33 5.64	0.91 1.84	2.30 5.21	4.66 4.15
IP index	H1 H2	44.88 56.99	31.05 27.94	97.75 62.90	33.62 32.68	21.88 29.96	39.78 39.89	50.77 51.53
Average Spectral acceleration (g)	H1 H2	0.175 0.19	0.278 0.286	0.103 0.121	0.247 0.239	0.211 0.11	0.129 0.112	0.108 0.105
Significant Duration (s)	H1 H2	29.8 26.3	32.27 24.12	31.92 31.15	19.8 21.35	13.65 21.73	18.19 21.13	12.6 20.28