

**ADDIS ABABA UNIVERSITY**  
**ADDIS ABABA INSTITUTE OF TECHNOLOGY**  
**SCHOOL OF GRADUATE STUDIES**



**Effects of Soil-Structure Interaction on the Seismic  
Response of Building Considering Three  
Categories of Soil**

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**A Thesis Submitted to the School of Graduate studies in Partial Fulfillment of  
the Requirement for the Degree of Master of Science in civil engineering  
(structures)**

By

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Addis Ababa

A Thesis  
Submitted in Partial Fulfillment of the Requirements for the Degree of Master of Science

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## **UNDERTAKING**

I certify that research work titled “Effects of Soil-Structure Interaction on the Seismic Response of Buildings considering three categories of Soil” is my own work. The work has not been presented elsewhere for assessment. Where material has been used from other sources it has been properly acknowledged / referred.

Miskir Tadesse

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## **ABSTRACT**

The primary aim of this study was to investigate the influence of soil-structure interaction (SSI) on the seismic performance of a building under three different soil categories. To achieve this, the behavior of a structure placed on three categories of soil—hard, medium, and soft—was assessed. A comprehensive literature review was conducted to understand the fundamental concepts of SSI, the techniques used to analyze it, and its influence on the seismic demands of buildings. The adopted methodology began with site assessment and gathering the geometric data of the building. Subsequently, a standard 11 story hospital building was analyzed following European standards. The underlying soil was modeled using the direct approach to account for soil flexibility. The building was then subjected to the 1952 Hollywood Storage earthquake, utilizing Midas Gen and Midas GTS NX for structural and geotechnical analysis, respectively. The results were evaluated and compared based on vibration period, lateral deformation, inter-story drift, bending moment, and shear force, across different soil conditions. Findings indicated that as the stiffness of the subsoil decreases, the influence of SSI on the building's seismic response becomes more significant and adverse. Specifically, SSI led to an increase in vibration period, lateral displacement, and inter-story drift, especially on soft soil. These results emphasize the importance of considering SSI in the dynamic design of buildings on soft soil to ensure structural safety.

Key words: Seismic soil-structure interaction (SSI), soil flexibility, Hollywood storage earthquake

## **CHAPTER 1: INTRODUCTION**

Earthquakes are among the most devastating natural hazards, posing a significant threat to human lives and infrastructure. In recent years, Addis Ababa has experienced an increasing frequency of seismic activities, raising concerns over the safety of structures, particularly critical facilities such as hospitals. Given that hospitals must remain operational during and after an earthquake, their seismic resilience is of paramount importance.

The growing number of earthquakes in Addis Ababa highlights the urgent need for research that considers soil-structure interaction (SSI) effects in seismic design. Most commonly used structural analysis often assumes a fixed-base condition, neglecting the role of soil flexibility (SSI). However, ignoring SSI can lead to an underestimation of structural displacements, inter-story drift, and internal forces, potentially compromising the safety of buildings.

Researches on the influence of SSI on structural performance have been ongoing for over five decades, yet there remains considerable debate on its implications. Current building codes often provide limited guidelines for addressing SSI, and clear, standardized methods to account for its effects in structural design are still largely absent. The primary goal of this study is to assess and quantify the impact of SSI on the seismic response of a reinforced concrete building under varying soil conditions.

This study investigates the effects of SSI on 11-story reinforced concrete hospital building in Addis Ababa, considering different soil conditions and seismic risks.

To achieve this objective, the study is organized into five chapters. The first chapter outlines the purpose and scope of the research. The second chapter presents a literature review to establish a theoretical foundation. The third chapter details the methodology used to analyze the effects of SSI, the fourth chapter discusses the results obtained from comparing seismic demands across different soil types and finally in chapter five conclusion and recommendation are made.

## **1.1 Research objectives**

### **General objective:**

The main objective of this work is to study the influence of soil flexibility (variations in soil stiffness) on the seismic response of the building.

### **Specific objectives:**

The specific objectives include:

- To identify the key parameters which influence the seismic response of raft supported building structures when the effects of SSI are considered.
- To understand how each type of soil impacts the building's structural behavior during seismic events.

## **1.2 Scope of the study**

The outcome of the research is aimed to show the drawbacks of fixed base analysis for seismic design. In order to show the limitation of fixed base analysis three different categories of soil is taken and modeled in Midas gts nx geotechnical software for representing soil flexibility. Then the building with 11 stories is set on and connected with these three categories of soil (hard, medium and soft). Finally real ground motion is applied at the base and the analysis result is compared with fixed base model.

In this study, time history analysis approach is used for estimating the seismic response of reinforced concrete building models. The study focus specifically on Ethiopia, Addis Ababa by considering the local soil conditions, building practices, and seismicity of the region. The study methodology is focus on Numerical analysis

The following are the limitations of the research:

- Irregularity in the building is excluded
- The foundation considered is only raft foundation
- The variation of seismic response when the building height varies is also not considered

### **1.3 Research Methodology**

In order to achieve the objectives stated above, the following methodology is employed:

1. Idealized building models of (G+10) – framed structure, assumed to be located in Addis Ababa are employed. The models are symmetrical with respect to plan, and elevation.
2. Mat or raft foundations are assumed to support the building models with flexible base.
3. Ethiopian Building Code Standard ES EN1998:2015, Euro-code (EC) 8, 2004, and other relevant research journals are studied to identify the criteria for building to be regular, when to consider SSI in our structural analysis and how to breakdown SSI problems.
4. Four number of total building models are used, one for fixed base and three for flexible base
5. Three different soil categories (soft, medium and hard) are selected with appropriate strength parameter for modeling of the three flexible models using Mohr-Coulomb constitutive model.
6. The fixed-base building models are analyzed using Midas gen structural analysis software and their seismic responses obtained.
7. The flexible-base building models assumed to be supported on Raft foundations are analyzed using Midas gts nx geotechnical software and their seismic responses obtained.
8. Real ground motion of " 1952, Hollywood Storage, Peak acceleration, 0.042 g," is selected and applied on both fixed base and flexible base model.
9. Finally analyses results obtained from the fixed-base and flexible-base model buildings are systematically compared .Finally, conclusions are drawn and recommendations made.

### **1.4. Organization of the Thesis**

The thesis is organized in five chapters. Chapter 1 covers the background, objectives and scope of the study. A comprehensive literature review on Seismic Soil-Structure Interaction (SSI) analysis, its effects and methods of accounting for it are covered in Chapter 2. In Chapter 3, the building models under study are presented. Chapter 4 is devoted to sensitivity study, interpretation of results and comparisons of the analyses results. Finally chapter 5 provides conclusion and recommendations.

## CHAPTER 2: LITERATURE REVIEW

### 2.1 Soil-structure interaction

Wolf [1] demonstrated the key aspects of soil-structure interaction in Figure 2.1 by comparing the seismic responses of a structure placed on rock to one built on a rigid base, which includes the base-mat and side walls embedded in soil. The seismic waves, shown by solid arrows, propagate vertically through the rock toward the structure, with the magnitude of motion indicated by the length of these arrows. Point A, located at the free surface of the rock, is chosen as the reference point for comparing the motion at different locations. For the structure positioned on rock (which can be approximated as a fixed-base scenario), the motions at points A and B are nearly identical and correspond to the horizontal motion induced by the seismic waves.

Thus, the motion at the reference point A can be directly applied to the structure's base. If the lateral stiffness of the structure is significant, the input motion at the base would result in horizontal accelerations that remain constant throughout the structure's height. As a consequence, a transverse shear force and overturning moment will be generated at the base. Given the high stiffness of the rock, these forces would cause negligible additional deformation in the surrounding soil. Therefore, the rigid foundation remains securely attached to the rock, moving in agreement with the horizontal motion of the rock.

In contrast, the structure built into the soil exhibits a noticeably different dynamic behavior compared to the structure placed on rock. This variation is primarily due to the fact that the motion at the base center (point O) differs from that at the reference point A, as a result of three main factors.

First, the so-called free-field motion, which refers to the site motion in the absence of structures and excavations, is altered. As shown in Figure 2.1(c), the soft soil layer above the rock reduces the motion at point C (denoted as  $\ddot{u}_g$ ), which would match the motion at point A if there were no soil covering the rock (as shown in Figure 2.1(b)).

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The wave propagation through the soil layer causes vibrations in the soil particles, leading to either amplification or attenuation of the free-field motion. Typically, the motion is amplified depending on the frequency characteristics of the seismic excitation. Consequently, the motions at points D and E (represented as  $\ddot{u}_{g,b}$ ), which correspond to the soil-structure interface once the structure is in place, differ from the motion at point C in Figure 2.1(c).

Second, the construction of the rigid foundation within the soil alters the motion at its base, which may undergo an average swaying displacement in addition to an extra rocking motion (Figure 2.1(d)). The combined swaying and rocking responses cause lateral acceleration variations along the height of the rigid superstructure. This effect results mainly from the stiffness contrast between the foundation and the surrounding soil, and it occurs even when the foundation has no mass, a phenomenon known as Kinematic Interaction (KI).

Third, the transverse shear force and overturning moment induced by the inertial forces generate additional deformation in the soil, which further modifies the input motion at point O, the base center (Figure 2.1(e)). This interaction between the excited structure and the adjacent soil is referred to as Inertial Interaction (II), Lu [2].

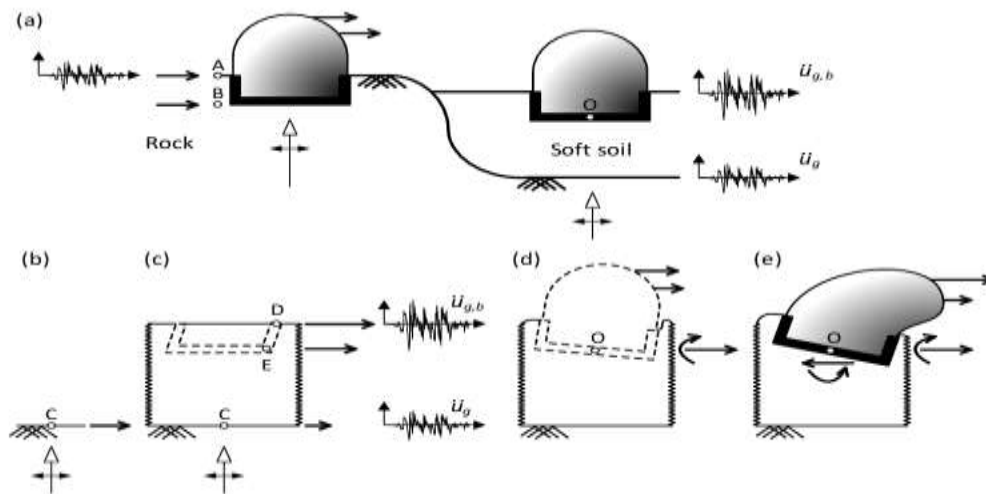


Figure 2.1: Seismic response of structures founded on rock and on soil. (a) Sites; (b) outcropping rock; (c) free-field; (d) kinematic interaction; (e) inertial interaction (after Wolf, 1985)

## **2.2 Kinematic and inertial interaction**

In line with the concepts discussed in the preceding section, two phenomena arise as a result of a dynamically excited structure situated on a soil site. These phenomena are widely known as kinematic and inertial interaction.

### **2.2.1 Kinematic interaction**

The kinematic interaction effect arises due to the stiffness difference between the soil and the foundation. In the free field, without a structure, the soil particles move according to the wave propagation patterns. However, when a foundation, either resting on or embedded in the soil, is sufficiently stiff to resist conforming to the free-field displacement, a deviation from the free-field motion occurs, even if the foundation has no mass. Figure 2.2 demonstrates scenarios where the kinematic interaction effect dominates.

In the entire figure, foundations are considered massless, with dotted curves indicating the free-field motions. In Figure 2.2(a), an embedded pile subjected to shear waves shows an increase in amplitude as the waves propagate upward through the soil. The pile's flexural stiffness restricts it from following the free-field motion, altering the soil displacements near the pile shaft. At the same time, the soil's motion around the pile induces bending moments, potentially threatening the pile's stability.

Figures 2.2(b) and (c) illustrate how the frequency components of motion impact the response of a mass-less embedded foundation. For high-frequency, horizontally varying motion (Figure 2.2(b)), the kinematic forces on the foundation cancel out, leaving the foundation unaffected by the wave motion. Conversely, when subjected to lower-frequency motion (Figure 2.2(c)), the foundation exhibits rocking and translation, resulting in a foundation input motion composed of swaying and rocking, despite the free-field motion being purely horizontal.

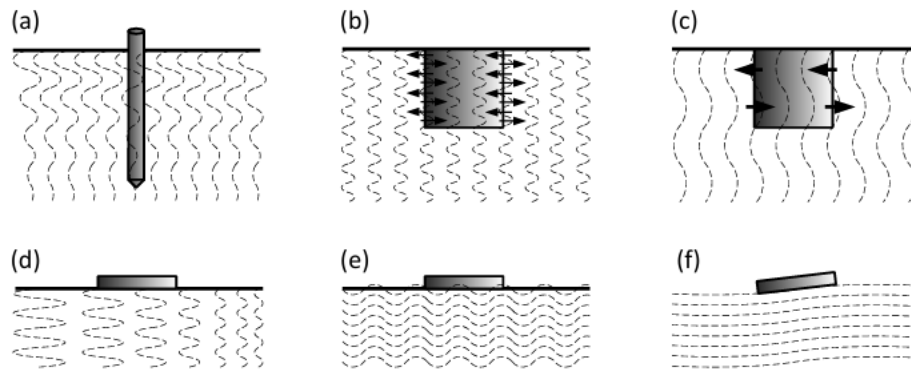


Figure 2.2: Kinematic interaction effect for shallow and deep foundations.

While kinematic interaction is more commonly observed with embedded foundations, particularly for deep foundations, there are instances where this effect is also significant for surface foundations. For example, Figure 2.2(d) demonstrates a case in which the surface foundation's in-plane stiffness restricts it from conforming to the deformation behavior of the underlying soil. Additionally, Figures 2.2(e) and (f) again show how excitation frequency influences the foundation's response, similar to the patterns observed in Figures 2.2(b) and (c).

Overall, the impact of kinematic interaction on foundation behavior depends on the relationship between the predominant wavelength and the foundation's dimensions. For high-frequency motions with wavelengths much smaller than the foundation's characteristic dimension, the contribution of these motions to the foundation's response is minimal (as shown in Figures 2.2(a), (b), and (e)). In such cases, the foundation can be considered as a high-period-pass (low-frequency-pass) filter for the high-frequency components of the free-field motion. This filtering effect is especially noticeable for short-period structures, which are mainly influenced by high-frequency motions, resulting in a substantial reduction in seismic demands on these structures.

When the foundation's dimensions are similar to the wavelength, the kinematic interaction will likely modify the foundation's vibrational modes (as seen in Figures 2.2(c) and (f)). However, if the foundation's characteristic dimension becomes significantly smaller than the wavelength, the kinematic interaction effect can typically be neglected, Lu [2].

### **2.2.2 Inertial interaction**

In an inertial interaction, the foundation input motion activates the superstructure, generating inertial forces within the structure. Two key features resulting from this interaction are discussed below.

Firstly, the transverse shear force and overturning moment, induced by the inertia at the foundation's base, cause additional deformation in the soil, on top of the deformation caused by the free-field motion. The extent of this deformation is influenced by the vibration amplitude and the soil's compliance. The presence of deformable soil beneath the foundation increases the flexibility of the entire Soil-Structure Interaction (SSI) system, making it more susceptible to longer-period components of the ground motion.

Secondly, the excited foundation acts as a finite vibration source that emits waves traveling through the soil toward infinity.

For an embedded shallow foundation, the swaying motion (denoted by displacement  $U_h$  in Figure 2.3(a)) generates P-waves (dilatational waves, denoted by P) and S-waves (shear waves, denoted by S) through compression/extension (at vertical interfaces) and friction (at the horizontal interface) between the foundation and the surrounding soil. In the rocking mode of vibration (with rotation angle denoted by  $\theta$  in Figure 2.3(b)), P-waves primarily arise from the compressive stresses transferred from the foundation base to the underlying soil.

Two main mechanisms of wave energy dissipation are involved in inertial interaction. The first type of energy dissipation occurs due to geometric attenuation as the wave propagates, with the wave front expanding from a point source. This process is known as radiation damping, as the foundation's vibration radiates waves into the surrounding soil. The second mechanism involves the nonlinear behavior of the soil, referred to as hysteretic damping. If the foundation is allowed to uplift, the interaction between the foundation and the soil, along with the vertical oscillatory motion, dissipates some of the kinetic energy imparted to the foundation.

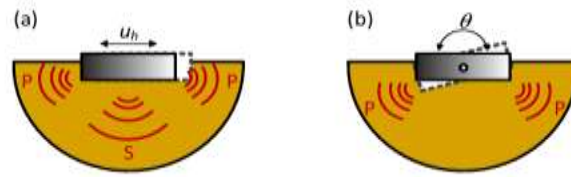


Figure 2.3: Foundation vibration of (a) swaying mode and (b) rocking mode in inertial interaction dissipates wave energy into the surrounding soil domain.

To summaries, two main features of inertial interaction are, respectively; lengthening of the vibration period and introducing radiation damping and soil hysteretic damping into the vibrating system, Lu [2].

### 2.3 Methods of analyses

The general methods used for soil-structure interaction analysis can be categorized into direct and or substructure approaches. The key characteristics of both methods are described in the following sections.

#### 2.3.1. Direct approach

In the direct approach, the soil volume and the structure are both part of the same model (Figure2.4) which is analyzed in a single step by using one of several numerical discretization techniques (e.g. Finite Element Method, Boundary Element Method, Finite Difference Method, etc.). For example in the Finite Element Method (FEM), the soil can be modeled with solid 3D elements and the structure as beam and/or frame elements. The dynamic equation of motion for the soil and structure system can be written as follows:

$$[M] \{\ddot{u}\} + [C] \{\dot{u}\} + [K] \{u\} = - [M] \ddot{u}_g$$

Where,  $[M]$ ,  $[C]$  and  $[K]$  are the mass, damping, and stiffness matrices of the structure, respectively.  $\{u\}$ ,  $\{\dot{u}\}$  and  $\{\ddot{u}\}$  are the relative nodal displacements, velocities and accelerations of the structure with respect to the underlying soil foundation, respectively. And  $\{\ddot{u}_g\}$  is ground acceleration,

The above equation can be integrated using standard explicit or implicit schemes (e.g., the Newmark or Wilson- $\theta$  methods) and is applicable to both linear and nonlinear analyses.

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The direct method, for instance, allows consideration of geometrical nonlinearities such as foundation uplift and gap formation at the soil-pile shaft interface under both static and dynamic loading. This technique inherently accounts for soil-structure interaction and its associated phenomena. However, the direct approach can be computationally expensive, depending on factors such as the model size, the constitutive laws used to describe the dynamic response of soils and structural elements, the boundary conditions at the soil-structure interfaces, and, in saturated soils, whether the analysis is conducted under drained or undrained conditions.

For the results to be meaningful, the direct method requires a detailed geotechnical characterization of the soil deposit. During numerical simulations, once the structure is subjected to seismic waves, it behaves as a vibrating system; emitting waves from the soil-foundation interface that eventually propagate downward into the unbounded medium. To accurately simulate energy radiation in a deformable and unbounded continuum, artificial absorbing boundaries are introduced at an appropriate distance from the structure. Several techniques can be employed to address this issue, including the use of infinite elements—a special formulation of finite elements with shape functions that describe a displacement field attenuating exponentially with distance. Alternative methods involve the introduction of non-reflecting boundaries (e.g., using local, consistent, or integral formulations) or absorbing layers such as Perfectly Matched Layers (PML). These absorbing boundaries act as transmitting surfaces, preventing reflections of outwardly propagating waves (Figure 2.4)

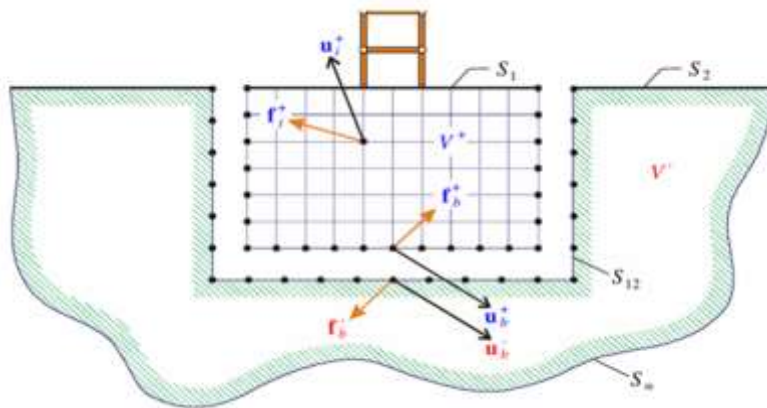


Figure 2.4: Boundary condition is imposed along artificial boundary borders of the model to simulate the radiation energy in a deformable, unbounded continuum

Although artificial boundaries are used to limit the model size, achieving accurate results still demands a large surrounding soil domain. As a result, the soil-structure system exhibits a very high number of degrees of freedom. Additionally, to accurately capture the dynamic behavior of both soil and structure at higher frequencies, a highly refined mesh is necessary. This requirement leads to significant computational demands, especially in the case of three-dimensional nonlinear simulations. Therefore, the direct method is seldom applied in routine engineering projects and is generally reserved for highly critical or specialized applications, Kotronis et al [3].

### **2.3.2 Substructure approach**

This approach divides the soil-structure system into two regions: the near-field, which captures the interaction between the structure and the adjacent soil, and the far-field, which represents the infinite extent of the soil for site response analysis. According to Kramer [4], this method relies on the principle of superposition and assumes linear behavior for both the soil and the structure.

Figure 2.5 outlines the three primary steps of the substructure method as follows:

- Step 1: Compute the Foundation Input Motion (FIM), representing the motion at the base slab assuming the structure and foundation are massless.
- Step 2: Evaluate the impedance functions, which characterize the stiffness and damping properties of the soil-foundation system.
- Step 3: Perform a dynamic analysis of the structure, modeled on a flexible base using the impedance functions and subjected to the FIM as the input ground motion.

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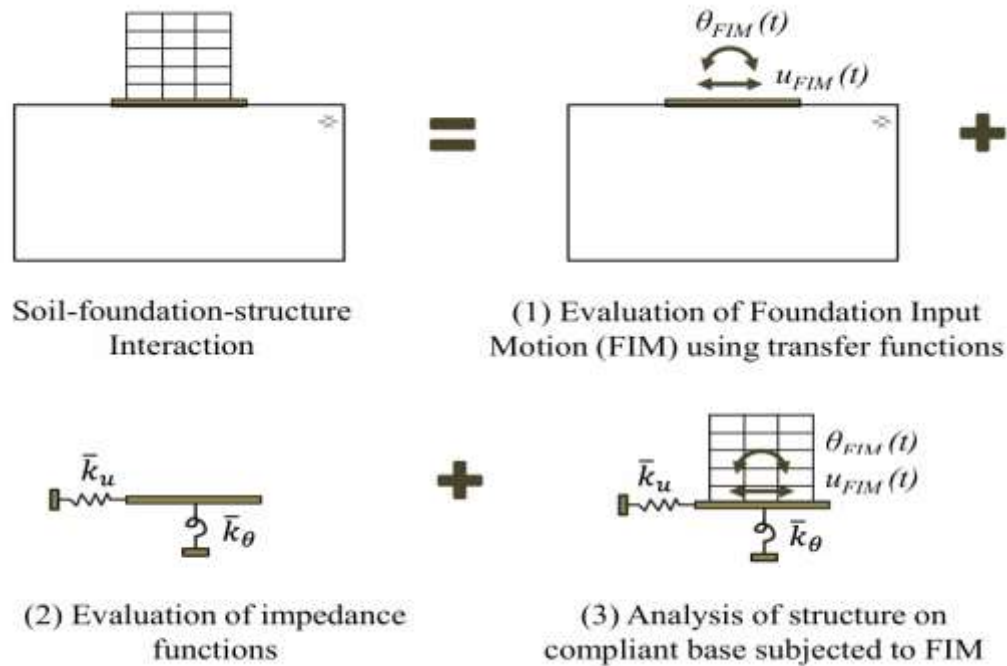


Figure 2.5 Components of Soil-structure Interaction problem used in Substructure approach

Several numerical investigations (e.g., Kutanis and Elmas) [5] have utilized the substructure method to evaluate the seismic behavior of structural systems, incorporating soil-structure interaction effects. Gutierrez and Chopra [6] highlighted that a major benefit of this approach lies in its versatility. Since each phase of the method is conducted independently, analysts can allocate computational resources more efficiently, focusing on the most critical aspects of the problem. Wolf (1998) pointed out that because the substructure method relies on the principle of superposition—valid only under linear conditions—some degree of soil nonlinearity can be incorporated using iterative wave propagation procedures. This makes the method suitable for systems exhibiting moderate nonlinearity. However, accurately capturing the full nonlinear behavior of the soil in dynamic analysis remains a challenge with this technique. Kutanis and Elmas [5] emphasized that further research is needed to evaluate the accuracy of this method and improve the numerical modeling techniques, especially in relation to the various parameters that affect soil-structure interaction, as also discussed by Hokmabadi [7].

## 2.4 Commonly used models for seismic SSI problems

Modeling of superstructures and foundations is generally found to be more straightforward and less complicated in comparison to the soil medium underneath, Bhattacharya and Dutta [8]. The most challenging part of dealing with soil-structure interaction problems is modeling the soil domain. This section attempts to present a comprehensive review of the most common available modeling techniques and computational methods of the soil domain. The advantages and disadvantages of each model will also be discussed. An overview of the commonly used modeling techniques and approaches to evaluate the SSI problem is illustrated.

### 2.4.1 Winkler model

In this model, the soil medium is idealized as a system of linearly elastic springs, which are independently and discretely distributed at close intervals along the foundation area, as shown in Figure 2.6

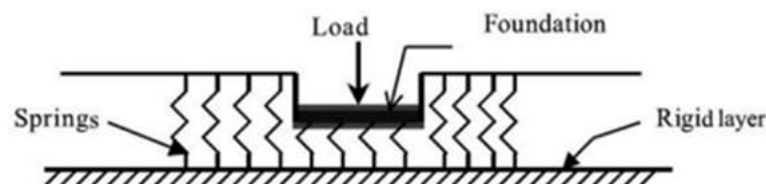


Figure 2.6: Winkler foundation model (Dutta and Roy, 2002)

Since the springs are independent, the deformation of the foundation due to the applied load is localized in the loaded region as Bowles [9]. The relationship between load and deflection at any point in the soil is given by the following expression:

$$P = kw$$

Where  $p$  denotes the applied pressure,  $k$  is the coefficient of subgrade reaction and  $w$  is the deflection. Due to its simplicity, several studies in the area of soil-structure interaction have been conducted based on the Winkler hypothesis, Daloglu and Vallabhan. [10]

In their comprehensive review, Dutta and Roy [11] noted that although this method offers simplicity and reduced computational demand; its primary drawback lies in the assumption of linear stress-strain behavior for the soil. Additionally, accurately defining the elastic springs that represent the supporting soil is a significant challenge, as the subgrade modulus depends not only on the inherent properties of the soil but also on the size and shape of the loaded area. Given that Winkler's model uses the subgrade reaction as its sole parameter, precise estimation of this value is essential. To address these limitations, several enhanced formulations of the Winkler model have been proposed in the literature, including those by Hetényi [12], Pasternak [13], and more recently by Bapir et al[14].

#### **2.4.2 Coupling and the pseudo-coupled approach**

In the static analogue of the Winkler spring model (Fig. 2.6), the springs act independently, meaning that each one responds without affecting the others and the compression of one spring does not impact the behavior of adjacent springs. However, in reality, soil compression at a specific location beneath the mat foundation generates vertical shear stresses and strains that propagate laterally, influencing the deformation of neighboring springs.

This coupling effect is particularly pronounced near the edges of the mat. Neglecting these coupling effects introduces errors that significantly underestimate differential settlements and peak bending moments in the mat. The limitation of the original (uncoupled) spring model is evident in a simple case of a perfectly flexible rectangular foundation (with no shear or bending stiffness) resting on an elastic half-space and subjected to a uniform vertical distributed load. Under these conditions, the uncoupled spring method would predict uniform settlement across the mat, whereas the exact solution from elasticity theory indicates that settlement at the center of the loaded area should be twice that at the corners.

Several more precise soil-mat foundation interaction models have been developed to account for coupling effects and accurately predict the deformation shape of the loaded soil surface. Recently, soil non-linearity has been incorporated into coupled soil springs for piled rafts. However, despite their increased accuracy, these refinements also lead to greater complexity, making the implementation of coupled foundation-soil interaction models more challenging in conventional structural analysis software.

A simpler, more approximate alternative to coupled models is the "pseudo-coupled" approach, which retains the static analogue of the classical Winkler spring method but assigns varying values of  $k_s$  across the mat, with higher values near the foundation edges compared to the center. To date, the suitable spatial distribution of  $k_s$  has not been firmly established and the existing variations of the pseudo-coupled method differ significantly between each other. The most basic form of pseudo-coupling is to simply use for the springs connected to the edge of the foundation twice as much  $k_s$  as for the springs at rest of the foundation. A more elaborate version dictates that  $k_s$  should decrease progressively from the edge to the center of the mat, Loukidisa and Tamiolakis [15].

Loukidis and Tamiolakis [15] proposed a simpler version of the pseudo-coupled method, which does not require a trial analysis or iterations. In this approach, the mat foundation is partitioned into  $N$  (two or more) concentric zones, where the central zone has half the width and half the length of the mat. Each zone  $i$  is assigned a different value of  $k_{s,i}$  in such a way that  $k_s$  increases from the inner to the outer zones and the outmost zone has twice as much  $k_s$  as the central zone (i.e.  $k_{sN} = 2k_{s1}$ ),

Applying this concept to a mat divided in three zones as shown in Fig. 2.7 with areas  $A$  (central),  $B$  (intermediate),  $C$  (outer), and assuming that for the intermediate zone  $k_{s2} = 1.5k_{s1}$ , we obtain

$$K_{s1} = K_{s,ave} \times \frac{A + B + C}{A + 1.5B + 2C}$$

$$K_{s,ave} = \frac{P}{W}$$

**A, B and C** are Central, Intermediate and Outer zone areas respectively

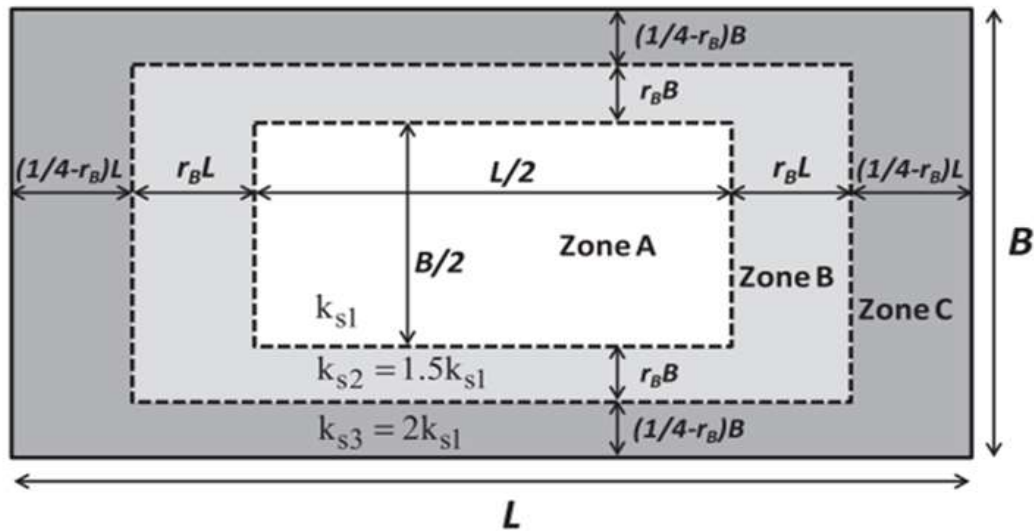


Figure 2.7 Division of mat onto zones with different modulus of subgrade reaction values for pseudo-coupled mat analysis

### 2.4.3 Micro-element model (Continuum)

Micro-element models provide a direct method for addressing SSI problems in the time domain by simultaneously modeling the soil and superstructure. The soil is represented as a continuum with an infinite number of degrees of freedom. To solve the problem, the entire domain is divided into smaller sub-domains, each with a finite number of degrees of freedom per element. This discretization can be achieved using various computational methods, including (FEM, FDM, and BEM). The micro-element model can handle any complex SSI problem with a high level of accuracy. However, the modeling process and computational demands are significant, particularly for complex geometries and non-linear problems. Despite this, advances in computational software have led to the broader adoption of FEM numerical methods over analytical approaches. Several researchers have utilized this technique to investigate non-linear dynamic SSI problems in various foundation and structural systems, Yeganeh and Fatahi [16].

Furthermore, Bowles [9]; Dutta and Roy [11] strongly advocate for the use of micro-element modeling when analyzing critical structures or when soil non-linearity effects must be considered. Besides computational techniques, three additional factors must be carefully addressed to achieve more accurate and realistic modeling, Bapir et al[14].

#### **2.4.3.1 Dynamic behavior of soil**

This relates to the mechanical behavior of the soil material during earthquakes which is described by the stress-strain relationship of the soil under cyclic loading. These relations are a set of mathematical expressions that models the soil behavior known as the constitutive model. Several constitutive models have been developed in the past such as Mohr-Coulomb, Drucker-Prager, Duncan-Chang, hyper elastic, and cam clay model as Wani and Showkat [17]. Nevertheless, owing to its simplicity and practicality in geotechnical engineering applications, the Mohr-Coulomb model has been extensively utilized by numerous researchers in soil-structure interaction studies. This model follows an elastic-perfectly plastic approach and characterizes failure criteria using fundamental geotechnical parameters, including friction angle, shear strength, and dilation angle of the soil.

Alternatively, soil nonlinearity can be characterized using a pair of dimensionless curves, known as the modulus reduction curve and the damping ratio curve as given by Seed and Idriss [18]. These curves illustrate the relationship between shear modulus and damping in response to the cyclic shear strain level in the soil. Due to the simplicity in practical application, these dimensionless curves are widely used by researchers in SSI simulations. Zhang and Far [19], Bapir et al [14].

#### **2.4.3.2 Interface element**

The interface between the foundation and soil must be modeled properly to simulate frictional contact. Generally, the interface is modeled to understand the effects of slipping, bonding, and rebonding between the foundation and the soil. Among the limited interface elements developed over the past 40 years, zero-thickness and thin-layer elements are the most widely recognized models among researchers in soil-structure interaction studies. This is because of its compatibility with the available constitutive models (Day and Potts) [21]. A comprehensive review of these two types of interface elements has been conducted by Dhadse et al [21] where the computational difficulties of the zero-thickness interface element and its solution are discussed in detail.

Moreover, various versions of the thin-layer interface element have been introduced, highlighting their applicability in soil-structure interaction problems. Nevertheless, the simplified approach presented by Rayhani and El Nagggar [22] to define the contact between rigid foundations and soft clays has been widely used by several researchers in the SSI investigations, where the performance of this approach has been experimentally verified by Tabatabaiefar et al [23].

#### **2.4.3.3 Boundary condition**

In direct approach modeling, a significant portion of the soil must be included in the SSI model to prevent wave reflections from affecting the structure's response. However, increasing the soil domain leads to a greater number of finite elements, which in turn raises the computational cost. As an alternative, a smaller soil domain can be considered, provided that suitable artificial boundaries are applied to prevent wave reflections.

Generally, boundary conditions fall into three primary categories: Elementary, Consistent (global), and Viscous (local) boundary conditions, Kant and Samanta [24]. Elementary boundary conditions are applicable only for static analysis. Consistent boundary conditions, known for their non-reflecting nature, can be employed in dynamic analysis with high accuracy. However, their implementation in finite element programs is complex and computationally demanding compared to local boundary conditions, Kant and Samanta [24]. The third category, known as local (or viscous) boundary conditions, was initially introduced by Lysmer and Kuhlemeyer [25] and is commonly referred to as the LK boundary condition. This approach involves attaching dashpots in both normal and shear directions to absorb seismic waves. Many studies have explored different boundary conditions for dynamic SSI problems. Among them, Jingbo and Yandong [26] demonstrated that the viscous boundary condition is the most effective choice for time-domain analysis due to its optimal balance between efficiency and accuracy.

## **2.5 SSI in Seismic Design guidelines**

The concept of soil-structure interaction (SSI) was first formally addressed in the U.S. through the ATC-3 report [27], which provided preliminary guidelines for seismic design and later laid the foundation for the National Earthquake Hazard Reduction Program (NEHRP) provisions. Within ATC-3, simplified SSI procedures recommended reducing the design base shear relative to that of a fixed-base structure. This potential advantage of SSI led to its treatment as an optional design consideration rather than a mandatory requirement in building codes. One reason for the continued exclusion of SSI from standard code provisions is the persistent uncertainty and debate surrounding its effects on structural performance, Lu [2].

### **2.5.1 Eurocode 8 and ES EN 1998-1:2015**

Eurocode 8 [28] and (ES EN 1998-1:2015) does not suggest specific design procedures for how to account soil- structure interaction, they provides a qualitative explanation of SSI effects. This code highlights and identifies the following cases where SSI effects should be accounted for in structural design:

- Structures where second-order (P- $\Delta$ ) effects are significant;
- Structures with heavy or deep foundations, including offshore caissons, bridge piers, and silos;
- Slender and tall structures, such as chimneys and towers;
- Structures resting on highly compressible soils, where the average shear wave velocity is below 100 m/s.

### **2.5.2 U.S. Standards and Design Guidelines**

SSI provisions currently exist in several important U.S. seismic design standards. Seismic Provisions for New Buildings and other Structures which integrate SSI into the equivalent lateral force.

The procedure is provided in Chapter 19 of ASCE [29], Minimum design loads for building, presents simplified SSI procedures that have been implemented in existing force based and displacement-based design for building structures.

### 2.5.2.1 Equivalent Lateral Force Procedure

ASCE (2010) [29] neglects the effect of kinematic interaction and deals with the period lengthening and modeled damping resulting from inertial interaction.

#### 1. Base Shear

To account the effects of soil–structure interaction, the reduction of base shear ( $\Delta V$ ) is calculated by

$$\Delta V = \left[ C_s - C_{s,ssi} \left( \frac{\xi_s}{\xi_{ssi}} \right)^{0.4} \right] \bar{W} \leq 0.3 C_s \bar{W}$$

Where  $C_s$  is a seismic response coefficient (i.e., the design pseudo-acceleration  $S_a$  normalized by the acceleration of gravity  $g$ ) of a system having a period of vibration  $T$  and a damping ratio  $\xi$ . The subscript  $ssi$  denotes an SSI system. For fixed-base systems (denoted by using subscript  $s$ ), the damping ratio, defined as a fraction of critical damping, is usually taken as 5%.  $\bar{W}$  is the effective seismic weight of the superstructure, usually taken as 70% of the total seismic weight. For structures supported on mat foundations that are resting on the ground surface or shallowly embedded in the soil where the effect of contact between the side walls and soil is deemed insignificant, the effective period of the flexible-base structure  $T_{ssi}$  can be evaluated from an adapted form of Veletsos and Meek [30].

#### 2. Effective Building Period

The effective period ( $T_{ssi}$ ) shall be determined as follows:

$$T_{ssi} = T \sqrt{1 + \frac{25\alpha R_h \bar{h}}{V_s^2 T_s^2} \left( 1 + \frac{1.12 R_h \bar{h}^2}{\alpha_\theta R_\theta^3} \right)}$$

Where  $\alpha$  = the relative weight density of the structure and the soil defined by:

$$\alpha = \frac{\bar{W}}{A \bar{h}}$$

$R_h$  and  $R_\theta$  = characteristic foundation lengths defined by:

$$R_h = \sqrt{\frac{A}{\pi}} \quad R_\theta = \sqrt[4]{\frac{4I}{\pi}}$$

Where

A=the area of the load-carrying foundation.

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$\bar{h}$  = the effective height of the structure, taken as 0.7 times the total height (for multi-story structures) except for structures where the gravity load is effectively concentrated at a single level (e.g., for one-story structures) in which case  $\bar{h}$  is taken as the height to that level.

I = the in-plane static moment of inertia of the load-carrying foundation about a centroidal axis normal to the direction in which the structure is analyzed.

Ts = the fundamental period of fixed base structure

Vs = shear wave velocity within influence depth

$\alpha_\theta$  = dynamic foundation stiffness modifier for the rocking motion as determined from Table 2.1

Table 2.1: Values of  $\alpha_\theta$

R $\theta$ /VsTs	$\alpha_\theta$
<0.05	1
0.15	0.85
0.35	0.7
0.5	0.6

## 2.6. Some key parameters of ES EN1998:2015

### 2.6.1. Ground types

Section 3.1.2 of ES EN1998:2015, classifies ground types into seven, A, B, C, D, E, S1 and S2. The classification is done based on the average shear wave velocity, Vs30, Standard Penetration test blow-count, N-value, and Undrained Shear Strength of soil (Cu), where Vs30 = average value of propagation velocity of S waves in the upper 30 m of the soil profile at shear strain of 10<sup>-5</sup> or less.

Table 2.2: Ground types based on ES EN 1998-1:2015

Ground Type	Description of stratigraphic profile	Parameters		
		$V_{s,30}$ (m/s)	$N_{SPT}$ (blows/30cm)	$C_u$ (kPa)
A	Rock or other rock- like geological formation, including at most 5m of weaker material at the surface.	>800	–	–
B	Deposits of very dense sand, gravel of very stiff clay, at least several tens of meter in thickness.	360-800	>50	>250
C	Deep deposits of dense or medium dense sand, gravel or stiff clay with thickness from several tens to many hundreds of meters	180-360	15-50	70-250
D	Deposit of loose to medium cohesionless soil or soft-to -firm cohesive soil.	<180	<15	<70
E	A soil profile consisting of surface alluvium layer with $V_s$ values of type C or D and thickness varying between about 5m and 20m.			
$S_1$	Deposits consisting, or containing a layer at least 10 m thick, of soft clays/silts with a high plasticity index ( $PI > 40$ ) and high water content	<100 (indicative)	–	10-20
$S_2$	Deposits of liquefiable soils, of sensitive clays, or any other soil profile not in types A-E or $S_1$			

### 2.6.2. Design ground acceleration, $g_a$

The seismic hazard map of Ethiopia, as given in ES EN 1998:2015, is developed based on a 475 years return period (with an exceedance rate of 10% within 50 years). The Seismic hazard map is divided into 5 zones, where the ratio of the design bedrock acceleration to the acceleration of gravity  $g = \alpha_0$  for the respective zones is indicated in Table 2.3

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Table 2.3: Bedrock Acceleration Ratio,  $\alpha_0$  (ES EN 1998:2015)

zone	5	4	3	2	1
$\alpha_0 = \mathbf{a}g/g$	0.2	0.15	0.1	0.07	0.04

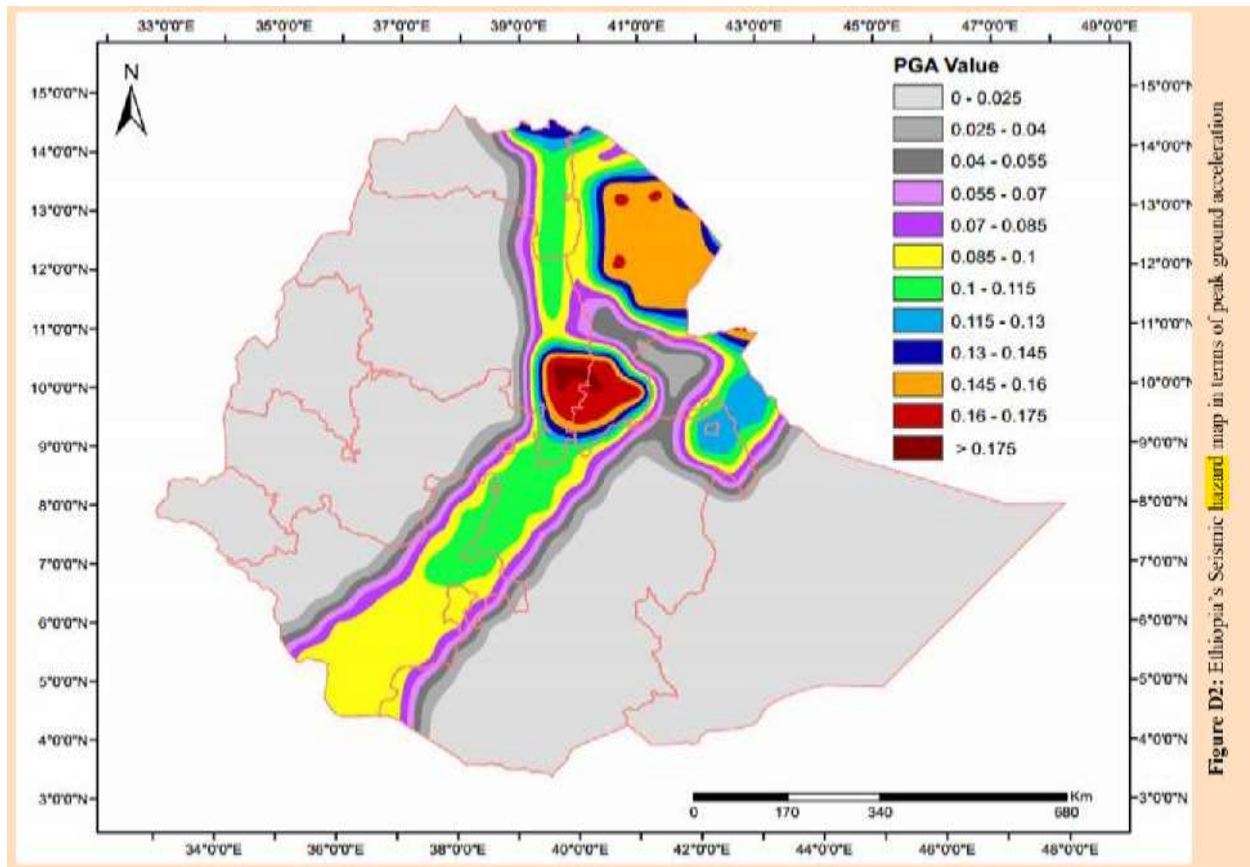


Figure 2.8: Ethiopian's seismic hazard map in terms of peak ground acceleration.

### 2.6.3 Damping correction factor, $\eta$

The damping correction factor  $\eta$  is used to account for the effect of SSI in modifying the basic response spectra provided for fixed-base conditions.

The spectral shape may be adjusted for other damping values with the correction factor  $\eta$  given by ES EN 1998-1: 2015 as:

$$\eta = \sqrt{10 / (5 + \xi)} \geq 0.55$$

Where:  $\xi$  is the viscous damping ratio of the structure, expressed as a percentage.

Studies reveal that using higher values of foundation damping (i.e.  $> 5\%$ ) results in reduction of base shear by up to the upper limit allowed 30%, Worku [31]. For reinforced concrete structures, a damping ratio of 5% is used, so that the corresponding damping correction factor  $\eta$  is 1.

#### **2.6.4 Response Spectra Types**

In ES EN 1998-1:2015, the impact of ground-motion intensity is addressed by defining two types of Response Spectra (RS). Type-1 Response Spectra applies to regions where earthquakes are expected to have a magnitude  $M_s$  greater than 5.5, covering areas of moderate to high seismicity. Conversely, Type-2 Response Spectra is designated for regions with lower seismic activity ( $M_s < 5.5$ ). The site amplification factors associated with Type-2 Response Spectra are higher compared to those of Type-1 Response Spectra.

#### **2.6.5 Behavior Factor, $q$**

To avoid direct inelastic structural analysis in design, the structure's ability to absorb seismic energy, primarily through ductile behavior and other dissipation mechanisms is considered by modifying the response spectrum. This modification results in a "design spectrum," which is a reduced form of the elastic response spectrum, achieved using the behavior factor,  $q$ . The behavior factor,  $q$ , represents the estimated ratio of the seismic forces a structure would withstand under fully elastic response (assuming 5% viscous damping) to the forces used in design, ensuring an adequate structural performance. The values of  $q$ , which also reflect the effect of damping variations, are specified in ES EN 1998-1: 2015 for different materials and structural configurations based on their ductility class.

### **2.7 Seismic analysis method**

In the early stages of design, equivalent static seismic forces are applied to evaluate the internal force demands of structural members using linear elastic analysis. These forces are subsequently used to determine the required strength of the members.

The static seismic forces are obtained by dividing the elastic design acceleration spectrum by a structural strength reduction factor, specifically referred to as the structural behavior factor,  $q$  (EC8) [28]. Typically, the elastic design spectrum, which is associated with a 10% Probability of exceedance (POE) over a 50-year period, is defined in a smooth manner to provide a rational representation of the seismic demand on the structure at the designated site. The chosen strength reduction factor serves as an estimate of the expected inelastic response or damage level that the structure may undergo during seismic excitation, Raheem et al [32].

### 2.7.1 Lateral force method of analysis

This analytical approach is suitable for buildings where the impact of higher vibration modes beyond the fundamental mode in each principal direction is insignificant. As stated in EC8 sub-clause 4.3.3.2.1 [28], the seismic base shear force,  $F_b$ , for each considered horizontal direction, is determined using the following equation:

$$F_b = S_d(T_1) \cdot m \cdot \lambda$$

Where

$S_d(T_1)$  is the ordinate of the design spectrum at period  $T_1$

$T_1$  is the fundamental period of vibration of the building for lateral motion in the direction considered

$m$  is the total mass of the building, above the foundation or above the top of rigid basement

$T_c$  is the upper limit of the period of the constant spectral acceleration branch

$\lambda$  is the correction factor, the value of which is equal to:  $\lambda = 0,85$  if  $T_1 < 2 T_c$  and the building has more than two story, or  $\lambda = 1,0$  otherwise.

*Note: The coefficient  $\lambda$  accounts for the reduction in effective modal mass of the fundamental mode in buildings with at least three stories and translational degrees of freedom in both horizontal directions. On average, this reduction amounts to approximately 15% of the total structural mass.*

The Distribution of the horizontal seismic forces can be determined by

$$F_i = F_b \times \frac{S_i \cdot m_i}{\sum S_j \cdot m_j}$$

Where

$F_i$  is the horizontal force acting on story  $i$ ;

$F_b$  is the seismic base shear force

$s_i$  ,  $s_j$  are the displacements of the masses  $m_i$  ,  $m_j$  in the fundamental mode shape;

$m_i$  ,  $m_j$  are the story masses

When the fundamental mode shape is approximated by horizontal displacements increasing linearly along the height, the horizontal forces  $F_i$  should be taken as being given by:

$$F_i = F_b \times \frac{z_i \cdot m_i}{\sum z_j \cdot m_j}$$

Where  $z_i$  ,  $z_j$  are the heights of the masses  $m_i$  ,  $m_j$  above the level of application of the seismic action (foundation or top of a rigid basement).

### **2.7.2 Modal response spectrum (RS) method**

The modal response spectrum analysis is applicable to all building types, whereas the lateral force method is subject to multiple limitations due to concerns that it may yield un-conservative results under certain conditions. Despite this drawback, the lateral force method remains widely utilized because of its simplicity in application as given by Crowley & Pinho [33].

Response spectrum analysis considers a sufficient number of vibration modes to ensure that at least 90% of the building's total mass is captured in both principal horizontal directions. Most structural design codes define the target elastic acceleration spectrum based on seismic intensity, expressed through the expected design peak ground acceleration (PGA), and the influence of ground conditions, which are described by the response spectral periods  $T_b$  and  $T_c$ .

These spectral periods vary depending on the soil classification, ranging from hard rock (A-type) to soft soil (E-type).

Several building codes acknowledge that the period of vibration estimated using the empirical height-based formula is more reliable, as it is derived from measured building responses to earthquake motions (Goel and Chopra [34]; Ghosh and Fanella) [35]. However, in cases where higher vibration modes significantly affect the response—such as in tall or irregular buildings—the modal response spectrum method provides a more accurate representation of lateral force distribution, Raheem et al [32].

### **2.7.3 Nonlinear time history (TH) method**

The nonlinear time-history analysis is the most advanced technique for evaluating a structure's seismic response. In this method, an earthquake acceleration time-history is applied at the base of the structure, and the structural response is computed at every second throughout the earthquake duration. Unlike the response spectrum approach, this method explicitly accounts for the time-dependent nature of seismic loading, meaning that the stresses and deformations at any given moment serve as initial conditions for the subsequent time step. Moreover, it allows for the inclusion of nonlinear behavior commonly observed during seismic events, which cannot be easily incorporated in response spectrum analysis. Since this method does not assume a predefined mode combination approach, the results are more accurate and less conservative.

Additionally, it ensures 100% mass participation, whereas response spectrum analysis typically captures only 90–95% of the total mass, which is necessary for accurately estimating seismic forces. All forms of nonlinearity can be represented in this analysis, making it particularly useful in seismic retrofitting scenarios that rely on energy dissipation through plastic hinge formation or member yielding. However, this method is computationally demanding, time-intensive, and produces large volumes of data. Another challenge is the uncertainty in selecting input ground motions. To address this, three to five different acceleration time-histories are often used, increasing the cost and effort required.

The equation of motion for a system subjected to earthquake forces is expressed as Algreane et al [36].

$$[M] \ddot{u} + [C] \dot{u} + [K] u = F(t)$$

In which, M, C and K are the mass; damping and stiffness matrices, respectively. F (t) is the seismic excitation and  $\ddot{u}$ ,  $\dot{u}$  and u are accelerations, velocities and displacements time-dependent vectors, respectively.

The outcomes of time-history analysis are highly dependent on the characteristics of the selected ground motion records and their associated elastic response spectra, Kunnath and Kalkan [37]. A constant damping ratio of 0.05 was assumed for reinforced concrete structures. The direct integration technique was employed for solving the nonlinear time-history response. Nonlinear analysis can also serve as a validation tool for design cases that may not conform to prescriptive building codes. Furthermore, story drifts and floor accelerations play a critical role in assessing damage to nonstructural components and the overall seismic performance of buildings, Raheem et al [32].

## CHAPTER 3: METHODOLOGY

### 3.1 General

To achieve a fully coupled analysis of the entire soil-structure system, this study develops a three-dimensional numerical model that treats both the soil and structure with the same level of precision. Using the direct analysis method, the numerical model performs fully nonlinear time-history dynamic simulations to capture the realistic seismic response of the system. Since nonlinear dynamic responses of soil-structure systems cannot be accurately determined using frequency domain analysis because it assumes linearity -a time domain approach is essential. In this study, the three-dimensional explicit finite element software MIDAS GTS NX has been utilized. This program models different soil and material behaviors through elements that can be modified to match the model geometry, with each element responding according to a designated constitutive model based on the applied forces or boundary conditions.

Two primary analytical techniques exist for modeling soil behavior in dynamic analysis of soil-structure systems under seismic loads: the equivalent-linear and fully nonlinear methods. The equivalent-linear approach, first introduced by Seed and Idriss (1968) [38], does not directly incorporate nonlinear effects during the solution process. Instead, it applies constant linear properties to each element throughout the shaking duration, which are estimated based on the mean level of dynamic motion. Consequently, this method only approximates nonlinearity effects, leading to soil elements being overly soft and over damped during weak shaking and too stiff and under damped during strong shaking.

On the other hand, the fully nonlinear approach directly follows the nonlinear stress-strain behavior of soil elements, automatically adjusting damping and apparent modulus based on strain levels. Byrne et al and Beaty [39] reviewed these methodologies and concluded that the fully nonlinear method is the most suitable approach for dynamic soil-structure interaction analysis. Furthermore, studies by Lu et al [40] emphasized the growing reliance on computational modeling for assessing nonlinear seismic ground response using nonlinear dynamic analysis, Hokmabadi [41].

Based on these findings, this study adopts the fully nonlinear method for modeling soil behavior in the dynamic analysis of soil-structure systems.

### **3.2 Geographic location**

Addis Ababa, situated at approximately N9.02 and E38.45, lies near the western escarpment of the Rift Valley. As the economic, political, and social center of the country, any major disaster in the city could lead to severe consequences. Although it is not considered the most seismically active region in the country, the potential damage from even moderate earthquakes could be significant. A minor earthquake near a densely populated area can have considerable impacts, whereas a much stronger earthquake in a remote location might go largely unnoticed except by seismologists and monitoring stations. Throughout its history, Addis Ababa has experienced several earthquakes.; The 1906 earthquake with a magnitude of 6.8, approximately 100 km south of the city; the 1961 Karakore earthquake, which had a magnitude of 6.6 and occurred about 200 km away; and the July 1997 earthquake of magnitude 4.0, located 22 km southwest of Addis Ababa. Additionally, smaller earthquakes occurring in 1977, 1984, and 1985 were reported to be felt in upper floors of high-rise buildings, Haile, n.d [42].

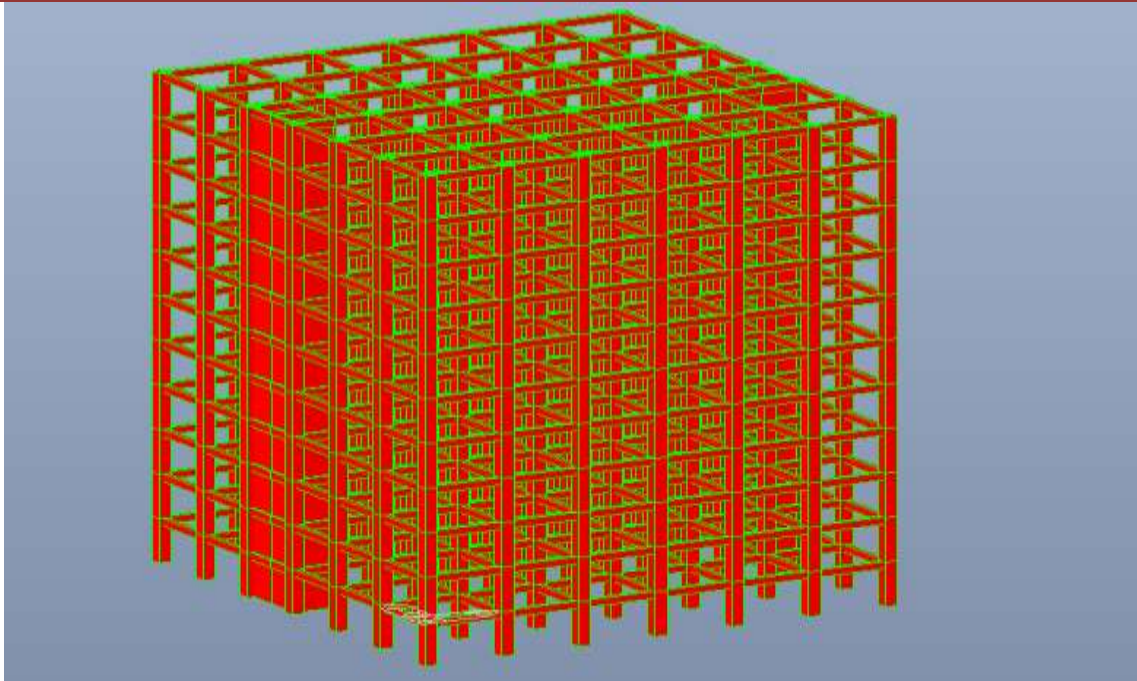
Recently more than 200 Small to moderate earthquake are occurred since late September 2024 in Awash area. Among them, the January 4, 2025 earthquake with 5.8 magnitudes is the maximum one.

### **3.3 Building Description**

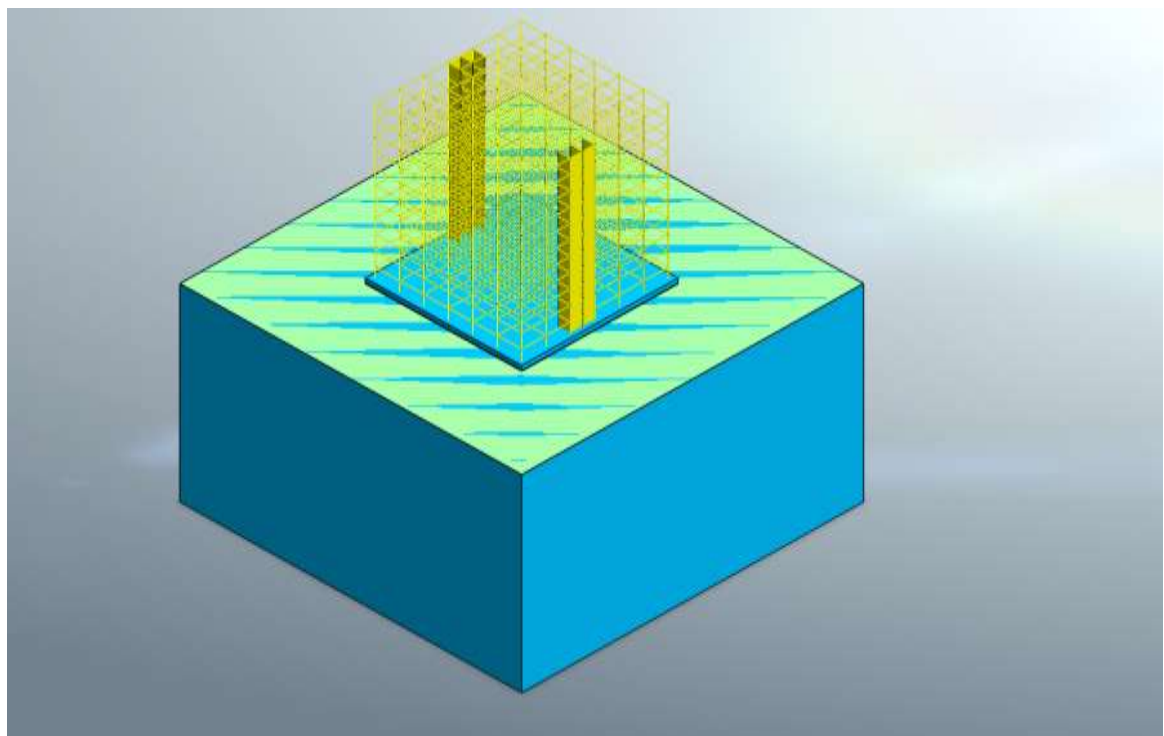
In this study, three 11 story buildings with different soil categories (soft, medium and hard) were analyzed in Midas gts nx geotechnical software for flexible base (SSI) case and another one building with the same story, configuration and layout were analyzed for fixed base(NSSI) case in Midas gen structural software. The lateral resisting systems used to resist the earthquake forces was moment resisting frame (SMF). The building's floor system consists of 30cmx50cm reinforced concrete beam sections, and 15cm lightweight concrete thickness. Fig 3.1A and 3.1B show isometric views of the fixed base and flexible base case model respectively.

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Three Categories of soil

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A)



B)

Figure 3.1 A) fixed base model in Midas gen, B) flexible base model in Midas gts nx,

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Figure 3.2 shows the typical floor system of the buildings described above. This building measures 33 m tall from ground surface to the roof. The area of the ground floors and all other floor is 900 m<sup>2</sup>.

All building model have 30mx30m in dimension and regular both in plan and elevation. The height of each floor is 3.m. All building models are assumed to be located at Addis Ababa and intended to function for hospital.

Table 3.1: Structural details and loading conditions of building model

Type of Structural System	Moment resisting framed structure
Importance class	Three ( i.e. importance factor of 1.4)
Beam size	300mmx500mm
Column size	700mmx700mm
Slab thickness	150mm
Shear wall thickness	20mm
partition wall load	2.5KN/m <sup>2</sup> is applied @ slab
Static gravity load <ul style="list-style-type: none"><li>➤ Live load</li><li>➤ Floor finishing load</li></ul>	<ul style="list-style-type: none"><li>➤ 5 KN/m<sup>2</sup></li><li>➤ 0.5 KN/m<sup>2</sup> @all floor slab</li></ul>

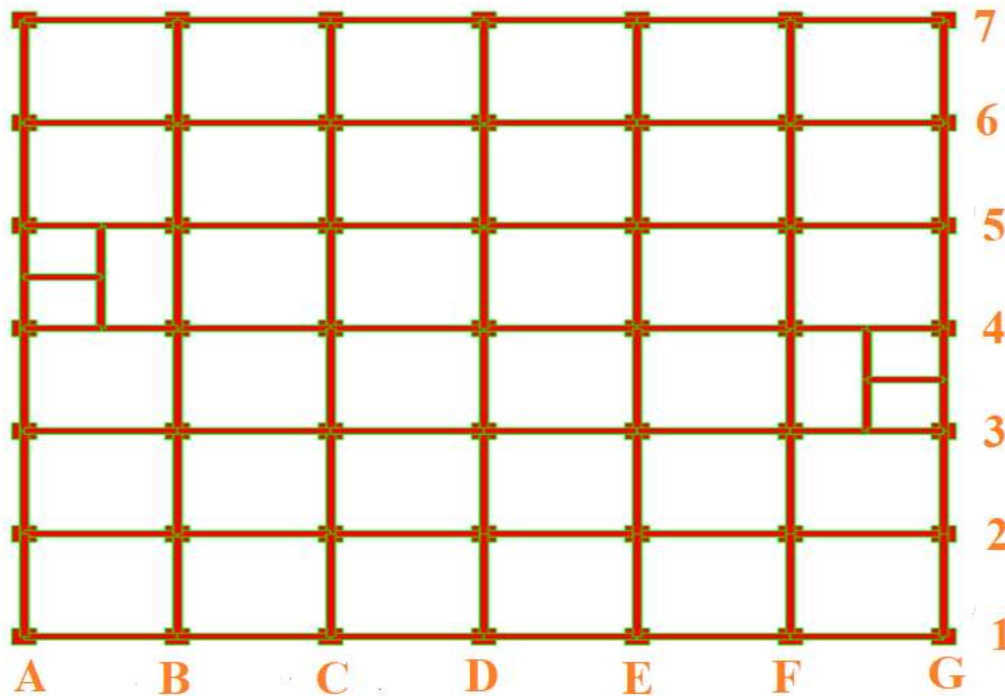


Figure 3.2: Typical floor plan for the 11-story building (similar for both fixed and flexible base)

### 3.4 Building Modeling

Soil-structure interaction under a seismic excitation for 11 story reinforced concrete building of moment resisting frame is undertaken and modeled for analysis in Midas Gen.

The analysis of the building is carried out by considering soil-structure interaction (SSI) on one hand and by neglecting SSI on the other hand. In the latter case, i.e., when the effect Soil-structure interaction is not considered, it implies that the support of a building is modeled as fixed base. In the former case, the building modeled in Midas gen is then exported to Midas gts nx geotechnical software in order to account soil flexibility. After exporting the building model from Midas gen, the building and the soil is analyzed in single step with equal rigor.

Four models are developed; the first model with a fixed base and the three other models integrate soil-structure interaction through direct method of soil modeling using Midas gtx nx for three different types of soil. The beam and column is modeled as 1D element (Generalized Beam element) and the soil is modeled as 3D (solid element).

### **3.5. Selected method of analysis**

There are two methods for the analysis of SSI (Section 2.3). These are: the substructure method, and the direct method. The Substructure method is also known as the multi-step approach. In this approach, the soil and the structure are considered separately, then solved independently and the final seismic response of the structure is calculated by combining these effects using superposition principles. Since the substructure technique is based on superposition principles, linear assumption of the soil and the structural behavior is necessary for the substructure approach. The direct approach is considered the most rigorous approach to solving SSI problems, especially in the case of complicated structural geometry and non-linear soil modeling. This approach is a very powerful method to solve a wide range of SSI problems including the linear and non-linear soil model for various foundation types. Accordingly the direct method/approach is selected for this study.

### **3.6 Selected Software for the study**

The selected software for this study is Midas Gen and Midas GTS NX. They are chosen for their comprehensive structural and geotechnical capabilities. Midas Gen is a strong structural analysis and design software used for various civil engineering and architectural applications. It can handle a wide range of structural analysis, including static, dynamic and nonlinear analysis. Midas GTS NX is comprehensive geotechnical analysis software, designed for wide range of geotechnical applications. It offers advanced 3D modeling capabilities for complex structures, including tunnels, excavations and deep foundations. GTS NX is specifically designed for analyzing soil-structure interaction.

Why Midas Gen and Midas GTS NX selected together? Due to

1. Data Transfer: they are designed to faultlessly import and export data between each other
2. Integrated Approach: they can be used in conjunction to provide a comprehensive analysis for both structural and geotechnical aspects of project
3. Enhanced Accuracy: by analyzing structural and geotechnical aspects, users can gain more complete understanding of the project and insure greater accuracy.

### 3.7 Foundations provided for the building models

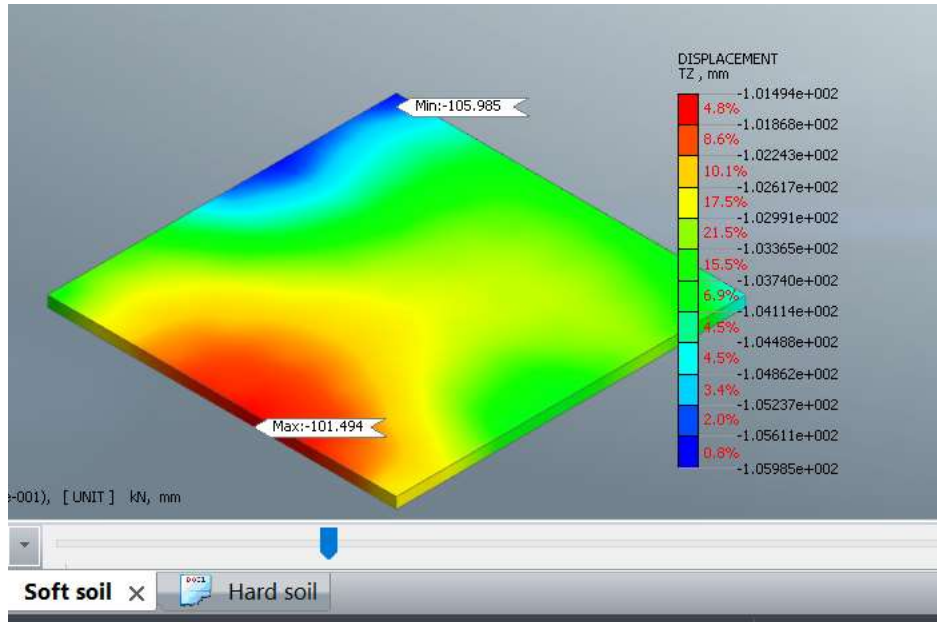
For this particular study, the foundation selected is a mat foundation with dimensions of 32m x 32m and a thickness of 1m. The mat foundation constructed using reinforced concrete, serves as a large rigid base that spreads the building loads evenly over a wider area, ensuring stability and minimizing differential settlement. To ensuring stability and minimizing settlement, foundations must meet two essential design criteria:

**I. Bearing capacity:** Foundations must be designed to avoid failure of the underlying soil. If the ground is unable to support the applied loads, it could result in the foundation collapse and risking the stability of the entire building.

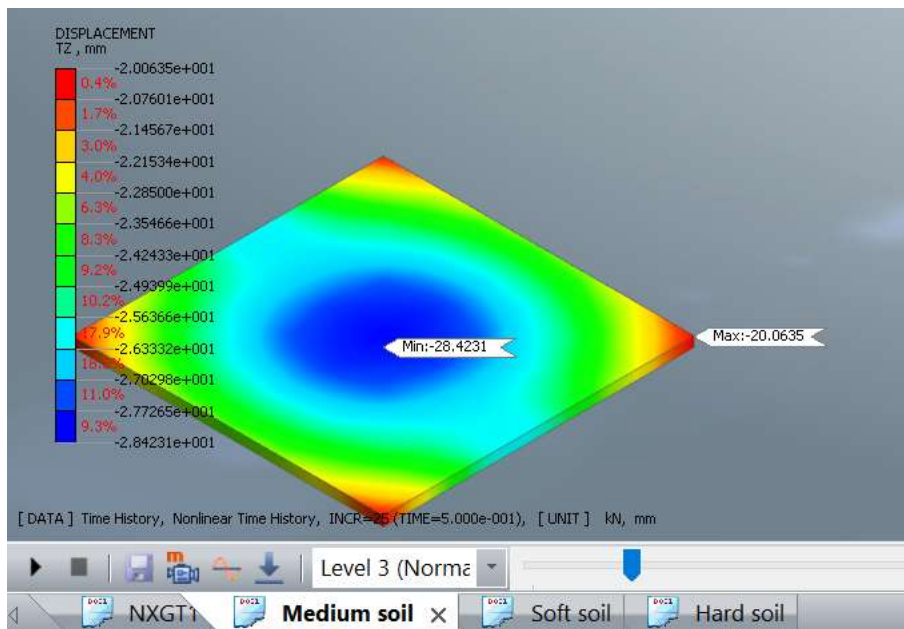
**II. Settlement:** Movement of the foundation due to the structure's load should remain within permissible levels. Significant settlement may lead to cracking, and possible structural issues.

According to EN 1997-1: 2004 [43] the total foundation settlement up to 50mm is acceptable. Additionally based on IS 1904,[44] for rafts founded on sands & hard clay the maximum allowable settlement is limited to 75mm whereas for soft clays it is allowed up to 100mm. Figure 3.3 shows the foundation settlement values obtained from GTS NX for the three different types of soil in this study. From the figure 3.3 it is clear that the settlement of the raft foundation for the two types of base conditions (Hard soil and Medium soil) are in agreement with allowable settlement. But for soft soil base condition the settlement of the raft the raft is above the limit. This implies the influence of soil flexibility (SSI) can significantly affect the response of foundation settlement.

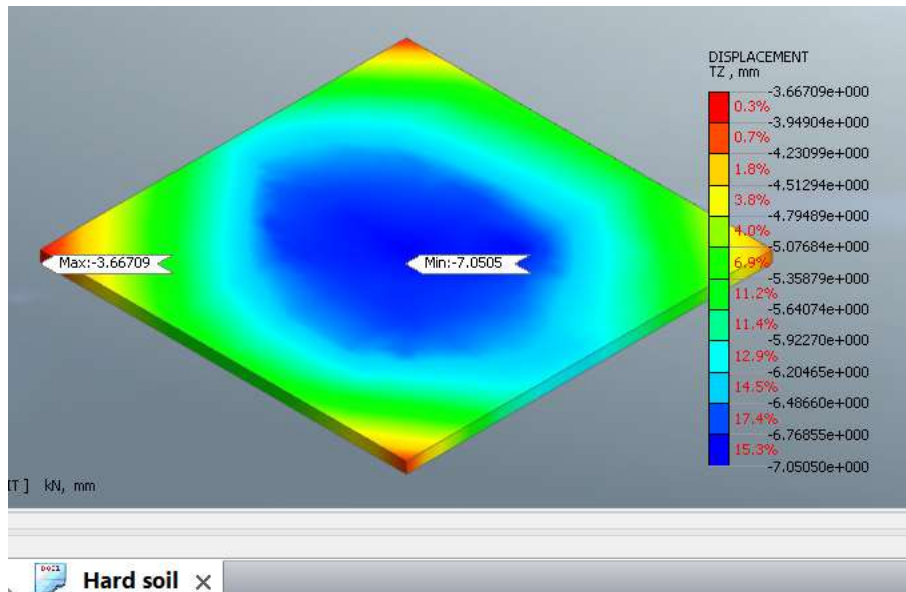
# Effects of soil-structure interaction on the seismic response of the building considering Three Categories of soil



A)



B)



C)

Figure 3.3: The foundation settlement values for A) Soft soil B) Medium soil C) Hard soil

### 3.8. The three categories of Soil

In this study the soils that are described as soft, medium and had are classified and defined based on IS 1983[45]. This is due to IS 1893 (Part 1) clearly classify and define the three types of soil using standard penetration value (N value).

IS: 1893-2002 - Part 1 has acknowledged the importance of local site effects and has defined three soil profile types, which are hard soils (Type I), medium soils (Type II), and soft soils (Type III).

#### A) Type I: Hard Soil

- 1) Well graded gravel or well graded sand both with less than 5% passing 75 mm sieve (Fines)
- 2) Well graded Gravel- Sand mixtures with or without fines
- 3) Poorly graded Sand or clayey sand, all having N above 30;
- 4) Stiff to hard clays having N above 16, where N is the Standard Penetration Test value

#### B) Type II : Medium Soil

- 1) Poorly graded sands or poorly graded sands with gravel with little or no fines having N

between 10 and 30;

2) stiff to medium stiff fine-grained soils, like Silts of Low compressibility (ML) or Clays of Low compressibility (CL) having  $N$  between 10 and 16.

### C) Type III - Soft Soils

All soft soils with  $N < 10$ . The various possible soils are

- 1) Silts of Intermediate compressibility (MI);
- 2) Silts of High compressibility (MH);
- 3) Clays of Intermediate compressibility (CI);
- 4) Clays of High compressibility (CH);
- 5) Silts and Clays of Intermediate to High compressibility (MI-MH or CI-CH);
- 6) Silt with Clay of Intermediate compressibility (MI-CI);
- 7) Silt with Clay of High compressibility (MH-CH)

### 3.9. Ground types of Idealized soil

Referring to ES EN 1998:2015,  $N_{SPT}$  (Standard penetration test blow-count) is selected to classify the idealized soil type in to their corresponding ground types based on their seismic response characteristics. In EN 1998-1, ground types are categorized into 7 soil types ranging from rock to soft soil formations

Table 3.2: Ground types of the idealized soil types according to ES EN 1998:2015

Soil Types	$V_s$	$N_{SPT}$	Ground type( ES EN1998:2015)
Hard Soil	>800	>50	A
Medium Soil	180-360	15-50	C
Soft Soil	< 180	<15	E

### **3.10 Modeling the Idealized soil domain**

In the numerical modeling process, soil medium is represented by elements, and each element behaves according to a prescribed linear or nonlinear stress/strain law in response to the applied forces or boundary restrains. Accordingly, a proper constitutive model representing the geotechnical behavior of soil elements should be implemented in Midas gts nx in order to conduct a rigorous SSI analysis.

Mohr-Coulomb model has been adopted in this study to simulate the nonlinear soil behavior and possible shear failure in the soil elements during the excitation. The adopted Mohr-Coulomb model is a nonlinear elastic-perfectly plastic model that has been employed by many researchers e.g. Conniff and Kioussis [46]; Rayhani and EL Naggar [47] to simulate soil behavior under seismic loads in soil structure interaction systems.

In direct approach modeling, a significant portion of the soil must be included in the SSI model to prevent wave reflections from affecting the structure's response. However, increasing the soil domain leads to a greater number of finite elements, which in turn raises the computational cost. As an alternative, a smaller soil domain can be considered, provided that suitable artificial boundaries are applied to prevent wave reflections ,Kant and Samanta [25]

Rayhani and El Naggar[47],recommended 30 meters as the maximum bedrock depth in the numerical analysis as the most amplification occurs within the first 30 meters of the soil profile. Thus, in this study, the bedrock depth of 40 meters is adopted.

Accordingly the size of soil area considered in this study is 70mx70m (2.4 times the width of the structure) and the soil depth taken in to account is 40 m (which is greater than the maximum bed rock depth recommended by Rayhani and El Naggar [47]

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The following soil parameters are required to define the soil elements in Midas gts nx, where Mohr-Coulomb model is implemented and the typical values of this parameter are given in table 3.2 based on different literature [52] [53][54][55][56]

- $\Phi$  : Friction angle (deg)
- C : Cohesion (Pa) :
- Mass density (kg/m<sup>3</sup>)
- E : Elastic modulus (Pa)
- Unit weight (KN/m<sup>3</sup>)

Table 3.3: Values of soil strength parameter used for this study

Types of soil parameters	Soft soil	Medium soil	Hard/stiff soil
Elastic Modulus (MPa)	10	35	100
Unit weight (KN/m <sup>3</sup> )	14	15	19
Poisson's ratio	0.3	0.35	0.35
Cohesion (KN/m <sup>2</sup> )	5	38	100
Friction angle (degree)	15	33	43

### 3.10.1 Fixed Boundary condition

Simulation of SSI problem revolves around media which are more appropriate to be modeled as unbounded half-space media. However, numerical methods, relying on the discretization of a finite region of space, require appropriate conditions to be enforced at the artificial numerical boundaries. The boundary conditions in the numerical models are prescribed at the boundaries of the numerical grids. Each of the two main steps of SSI numerical analysis, fixed base analysis and flexible base analysis, requires particular types of boundaries.

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During the fixed base analysis step, where the model should be analyzed under gravity loads and dynamic load in order to achieve the static equilibrium, preliminary boundary conditions should be adopted at the base of building. In the preliminary boundary conditions as shown in Figure 3.4, base boundary is fixed in all direction.

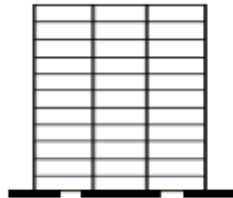


Figure 3.4 fixed boundary conditions for fixed base analysis

### 3.10.2 Free Field Boundaries

In order to accurately simulate SSI problem, the side boundary conditions of the numerical model must account for the free-field ground motion which would exist in the absence of the structure and foundation. As shown in Figure 3.5, the lateral boundaries or free field boundaries are applied at the sides of the model,

Prior to applying free field boundaries to the model, the model should be in static equilibrium and the dynamic boundary conditions at the base of the model should be specified.

After invoking the free field boundary, the static equilibrium and base conditions are automatically transferred to the free field region for the dynamic analysis. As plane waves travel upward, they remain undistorted at the boundary since the free-field grid provides conditions that are equivalent to those in an infinite medium.

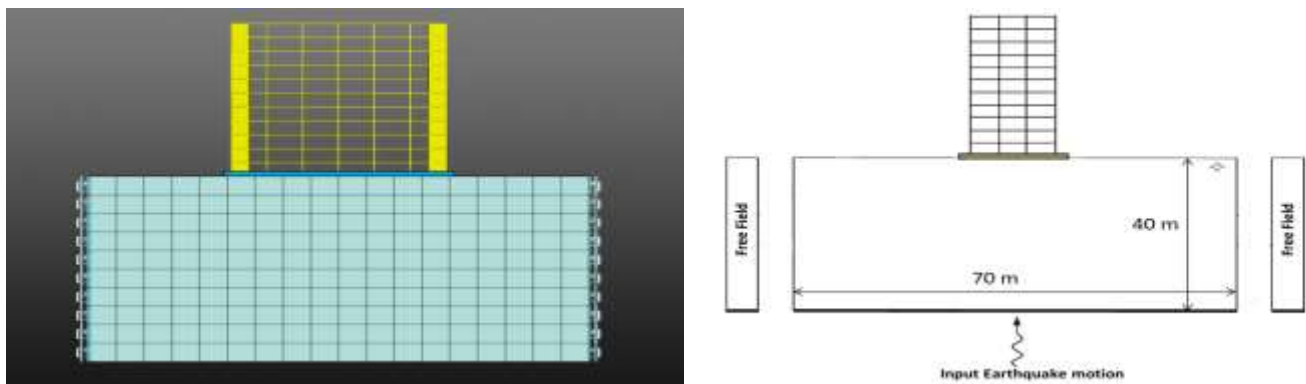


Figure 3.5: Free field boundary conditions for dynamic analysis of SSI used this study

### **3.11 Governing Equation of Motion for Soil-Structure Systems**

The governing equations of motion for the structure incorporating soil-structure interaction and the method of solving these equations are relatively complex. Therefore, direct numerical method, the method in which the entire soil-structure system is numerically modeled in a single step, is employed in this study. The dynamic equation of motion for the soil and structure system can be written as follows:

$$[M] \{\ddot{u}\} + [C]\{\dot{u}\} + [K] \{u\} = -[M] \{\ddot{u}_g\}$$

Where, [M], [C] and [K] are the mass, damping, and stiffness matrices of the structure, respectively. {u}, { $\dot{u}$ }, and { $\ddot{u}$ } are the relative nodal displacements, velocities and accelerations of the structure with respect to the underlying soil foundation, respectively. { $\ddot{u}_g$ } is ground acceleration.

### **3.12 Actions and combination of actions**

The norms that are used for the analysis are the Eurocode 0 (basis of structural design), Eurocode 1 (actions on structure), Eurocode 2 (design of concrete structures), and Eurocode 8 (design for earthquake resistance). These European standards define the actions and the combination of actions for the design.

#### **3.12.1 Actions**

Various types of actions can be applied on a structure. This analysis is focused on a building structure and considers different type of actions, including permanent actions, imposed actions and seismic action.

##### **3.12.1.1 Permanent actions**

Permanent actions include the self-weight of both structural and non-structural components. The self-weight of structural members is calculated by multiplying the unit weight of concrete by the cross-sectional area of each element. For non-structural elements, the self-weight is determined based on the guidelines provided in Eurocode 1.

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The only permanent structural load considered in the building is the self-weight of the slab. This is determined by multiplying the unit weight of concrete by the slab thickness. Given that the slab thickness is 15 cm, the resulting load (G1K) is 3.75 KN/m<sup>2</sup> across all floors. As for the permanent non-structural loads, their values remain consistent for the first ten stories and are reduced at the roof level, as detailed in Tables 3.4 and 3.5.

Table 3.4: permanent non-structural loads for story 1 up to 10

Nature	Designation	Value	Units
G2k	Screed	1.5	KN/m <sup>2</sup>
G2k	Sealing	0.5	KN/m <sup>2</sup>
G2k	Tiles	0.5	KN/m <sup>2</sup>
G2k	Partition wall	2.5	KN/m <sup>2</sup>
Total		5.0	KN/m <sup>2</sup>

Table 3.5: Permanent non-structural loads for the roof floor.

Nature	Designation	Value	Units
G2K	Waterproof	0.12	KN/m <sup>2</sup>
G2K	Concrete in the form of slope	2.50	KN/m <sup>2</sup>
Total		2.62	KN/m <sup>2</sup>

### 3.12.1.2. Imposed actions

Imposed actions are those arising from occupancy. It includes the normal use by people, the furniture and moveable objects and others. According to the Eurocode 1, different use categories of areas exist. Therefore, those ones are presented in the table A1 of the annex A. Based on these different categories the different values of loads recommended by this norm are presented in the table A2 of the Annex A

According to EC 2 the imposed loads for hospital (use category A)  $q_k$  is equal to 4.0 KN/m<sup>2</sup>.

The last floor is not accessible except for maintenance (use category H) so the imposed load can be taken as 0.4 KN/m<sup>2</sup>.

### **3.13: Input seismic excitation**

Precise evaluation of soil-structure interaction (SSI) in dynamic analysis requires carefully selected ground motion records. Given the scarcity of such data in Ethiopia, this research utilized the Pacific Earthquake Engineering Research Center (PEER) database (<https://ngawest2.berkeley.edu/>) as a trusted source. Maintained by PEER, this data base offers an extensive set of refined ground motion recordings from shallow crustal earthquakes globally.

The database provides important metadata, including source-to-site distances, site condition details, and earthquake source characteristics. A key feature is the inclusion of unscaled, original ground motion recordings, capturing the raw nature of seismic activity.

Furthermore, the NGA-East and NGA-West 2 databases supply a broad range of ground motion records, specifically suited for Central/Eastern North America and tectonically active zones, respectively, Rediet Lemma [48].

#### **3.13.1 Tectonic Regime**

The first stage in selecting input motions from the PEER database is identifying recordings from areas that share a similar tectonic setting with the target region. The NGA-West 2 database within PEER contains a wide range of ground motion records resulting from shallow crustal earthquakes in tectonically active zones worldwide. These records are especially relevant to NGA-West 2. According to Kebede and van Eck [49] seismic events in the Middle Ethiopian Rift (MER) predominantly originate in the shallow crust. Therefore, for seismic analysis in Addis Ababa, NGA-West 2 is the most suitable database. An earthquake ground motion (1952, Hollywood Storage Peak ground acceleration 0.042 g) is intended to be used and is applied at the base of the building after scaling/matching with target elastic response spectrum.

Table 3.6: Ground motion selected from PEER ground motion database for this study

Record number	sequence	Earthquake Name	Station Name	Year	Magnitude	Main shock duration
12		Kern country	Hollywood storage	1952	7.36	7.5 seconds

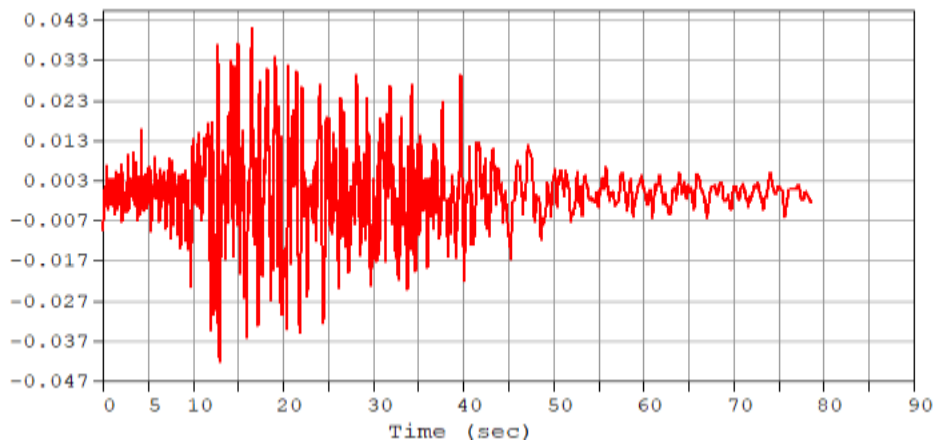


Figure 3.6: Horizontal component of the 1952, Hollywood storage earthquake

### 3.13.2 Defining Target Spectrum

Once the suitable database is chosen, the next step involves setting filtering criteria to obtain the desired target spectrum. This spectrum reflects the expected ground motion behavior at the reference location, usually described through pseudo-acceleration spectra. The initial phase in generating the target spectrum is to evaluate the seismic risk of the site. According to ES EN 1998-1:2015, a design PGA with a 10% chance of being exceeded in 50 years corresponds to a 475-year return period. For this study, a PGA value of 0.1g from Table D1 in ANNEX D is used.

According to ES EN 1998-1:2015, two forms of design response spectra are provided: Type 1 and Type 2. Type 2 spectra are intended for regions where seismic events generate surface wave magnitudes less than 5.5. In contrast, Type 1 spectra are designated for areas experiencing surface wave magnitudes greater than 5.5.

The study by Kebede and van Eck [49] shows that in the main Ethiopian rift, surface wave magnitudes range between 5.5 and 6.5. As these values surpass the 5.5 limit, the Type 1 spectrum is deemed appropriate for use in this study.

Accordingly, as described in section 3.9 ground Type A, ground Type C and ground Type E will be adopted to develop the target spectrum for the three different types of soil (i.e. Hard, Medium and soft soil respectively) with importance class III.

### **3.13.3 Scaling and Spectrally Matching of Ground Motion**

For conducting a nonlinear dynamic analysis, several methods of scaling/matching the input ground motions are available to modify accelerograms to become representative of the seismic demand. For instance: PGA scaling,  $S_a(T_1)$  scaling, ASCE scaling, SIa scaling and Time domain spectral matching are some of the methods used for scaling/matching input ground motion. Time domain spectral matching method is chosen for this study.

#### **3.13.1.1 Time domain spectral matching**

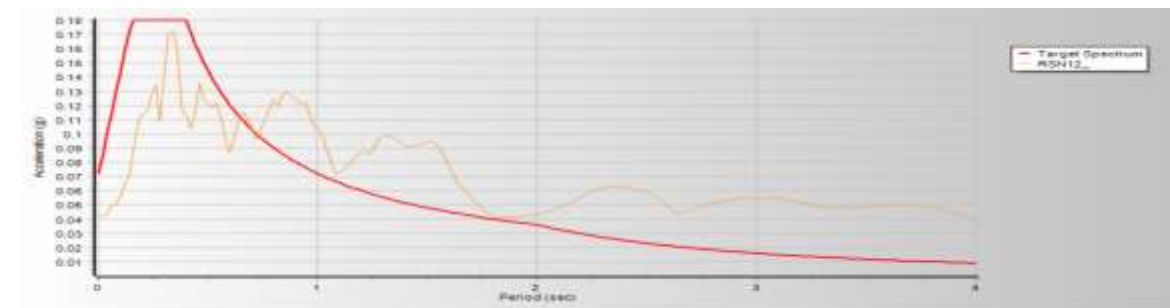
The spectral matching of ground motion records to a specified target spectrum was performed using SeismoMatch (Seismosoft, 2009). This software applies a wavelet-based algorithm to modify earthquake accelerograms so that their response spectra align closely with a desired target spectrum over a defined period range. The basic characteristic of the original record with respect to the amplitude and frequency content of the record over the time history duration is preserved and a developed design time histories have a spectra similar to the a design spectra (NEHRP 2011)[51]. The Kern County (LA - Hollywood Storage FF, 1952) earthquake used for this study is scaled and matched using time domain spectral matching for all (four) base condition as shown in figure 3.7. To match the selected input ground motion effectively, it is necessary to align it with the design response spectrum featuring 5% damping in the case of fixed base model and 5% to 20% damping in the case of flexible base.

The damping values in table 3.7 is calculated based Rayleigh damping (i.e. linear combination of the mass and stiffness matrices).

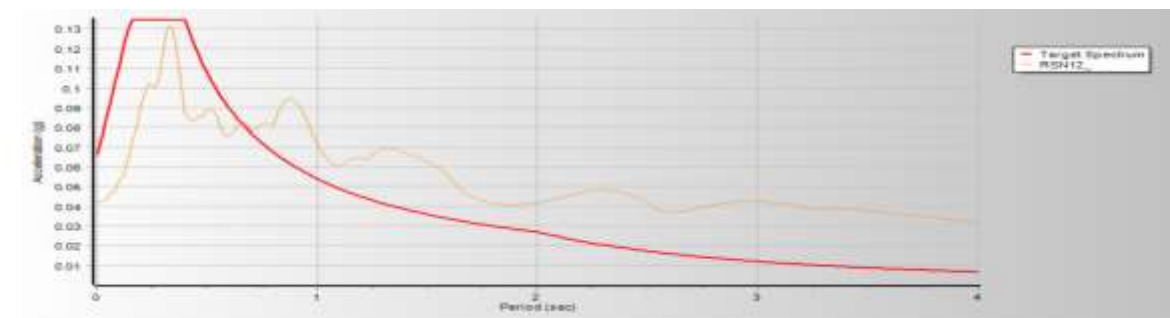
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Table 3.7: Different seismic parameters considered for matching the ground motion with target spectrum for all base condition.

Building supporting ground	Type of Spectrum	Ground Type	Importance Class	Damping (%)	PGA
Fixed base case	Type 1	Type A	Class III	5	0.1
Flexible base Hard soil	Type 1	Type A	Class III	5	0.1
Flexible base Medium soil	Type 1	Type C	Class III	5	0.1
Flexible base Soft soil	Type 1	Type E	Class III	5	0.1

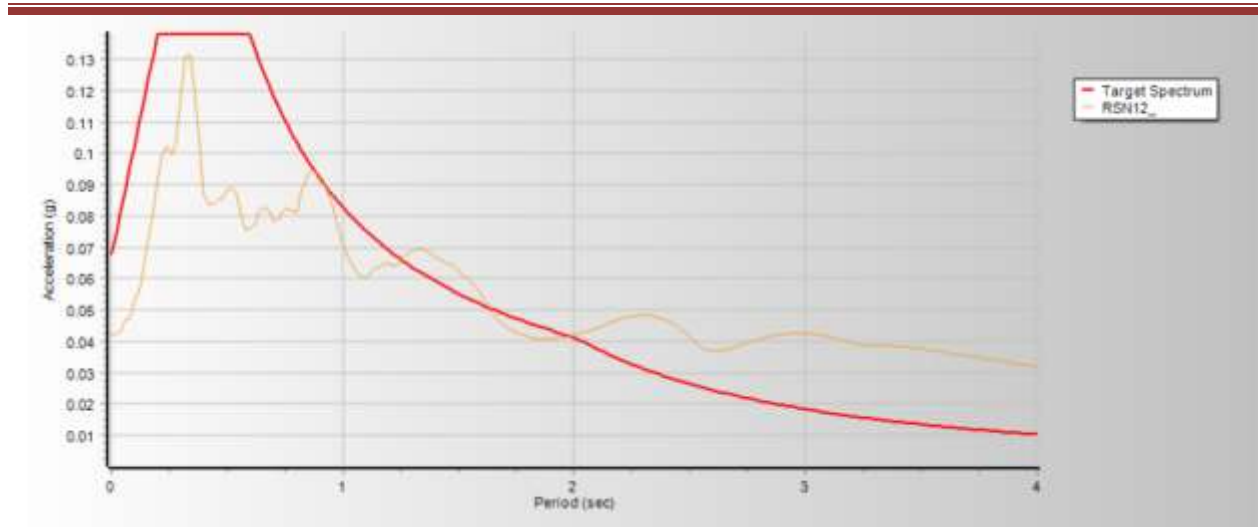


A)

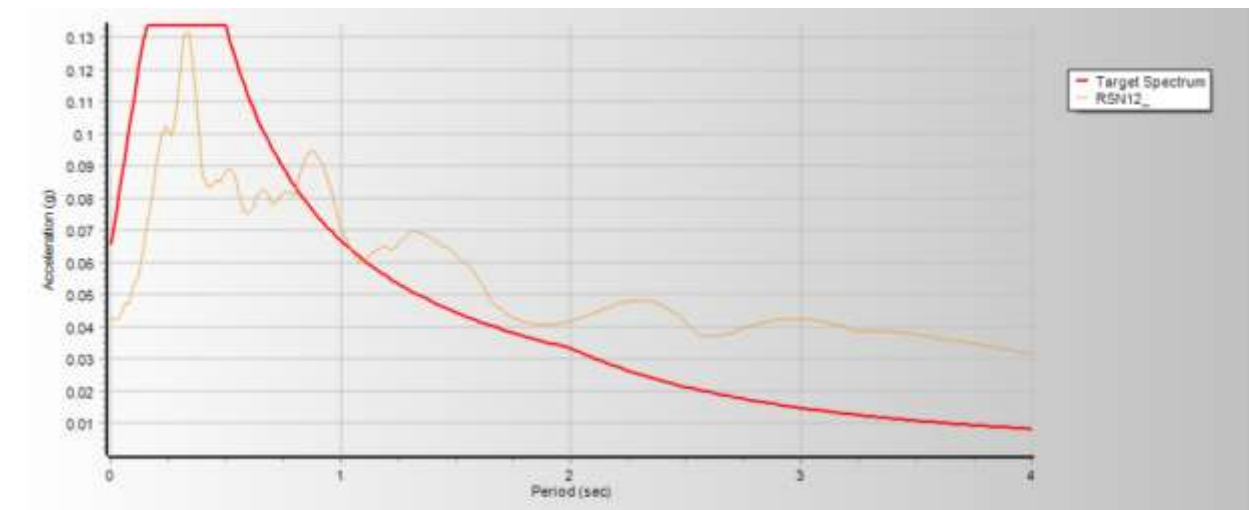


B)

Effects of soil-structure interaction on the seismic response of the building considering  
Three Categories of soil



C)



D)

Figure 3.7: Scaling/matching the input ground motions to the target EC 8 elastic spectrum for  
A) Fixed base B) Hard soil C) Medium soil D) Soft soil cases.

## CHAPTER 4: RESULTS AND DISCUSSION

### 4.1: General

Generally, some of the major factors which influence the seismic response of structures considering SSI are the magnitude of seismic excitation, the type of supporting soil, the type, shape and embedment depth of the foundation, and the type of the structural system used. In this study the following parameters are systematically varied to identify the major factors which influence the seismic responses of the structures. The results obtained for each case are then separately investigated and compared.

The following response parameters are used for comparison of the analyses results.

- Fundamental natural periods of vibrations,
- Top Floor displacement
- Inter story drift
- Bending moment variations along lengths of beams,
- Shear force variations along lengths of beams,

### 4.2: Natural time periods

This part consists to study the natural period of vibration at different soil conditions. The vibration period is an important parameter of a structure to estimate its seismic demand.

In this study the natural periods of vibrations of the models are obtained from Midas gen and Midas gts nx structural and geotechnical analyses program. The results obtained for the first three modes of vibrations for both the fixed-base and flexible-base models are presented in Table 4.1. The natural periods of vibrations for flexible-base models are found to be higher than their fixed-base counterpart models; and the period elongation becomes increase from hard soil type to soft soil type (i.e. period of vibration for hard soil < medium soil < soft soil)

Table 4.1: Natural period of vibration for both fixed and flexible base condition

Mode	Fixed base model	Flexible base soft soil	Flexible base Medium soil	Flexible base Hard soil
1	0.98	3.61	2.24	1.49
2	0.81	3.26	2.23	1.48
3	0.69	3.24	2.05	1.36

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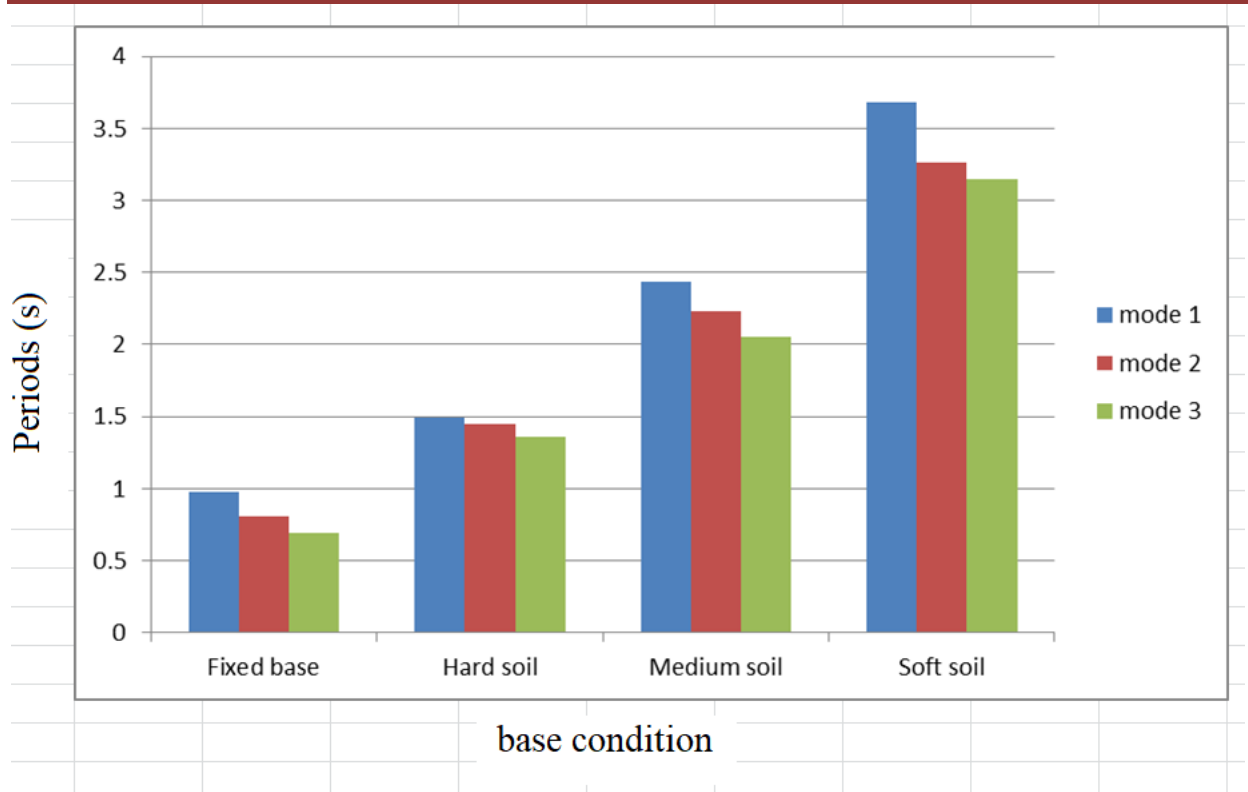


Figure 4.1: Natural time period at different base condition

### 4.3: Lateral Deformation

Story displacement is very essential parameter for nearby building collision effect in seismic event for making enough separation between nearby element. In order to draw a general conclusion to be used by practicing engineers, the average values of the 3D numerical predictions of deformations for each case were determined and compared in Figure 4.2 and 4.3. In comparison to the fixed-base structure, the maximum lateral deflection of the structure supported by hard soil, medium soil and soft soil increase by 1.25 times, 7.5times, and 23 times respectively( this comparison is done by neglecting foundation movement). For all three cases the base or the foundation is displaced horizontally by 0.2 cm, 5.8 cm and 9.6 cm for hard, medium and soft soil respectively. For further calculation which accounts story displacement value, this base or foundation movement should be subtracted from the lateral displacement value given in table 4.2.

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Table 4.2: Displacement of the structure for both fixed and flexible base model subjected for Hollywood storage earthquake.

<b>Lateral deformation (cm)</b>				
Story number	Fixed base	Flexible base Hard soil	Flexible base Medium soil	Flexible base soft soil
Story 1	0.0477	0.43	6.43	10.52
Story 2	0.14911	0.52	6.54	11.85
Story 3	0.27305	0.59	6.64	12.94
Story 4	0.40474	0.65	6.73	13.80
Story 5	0.53065	0.69	6.81	14.83
Story 6	0.64069	0.72	6.88	15.97
Story 7	0.73193	0.75	6.95	17.15
Story 8	0.79691	0.81	7.02	18.40
Story 9	0.843	0.95	7.06	19.72
Story 10	0.89106	1.06	7.1	21.10
Story 11	0.96321	1.2	7.2	22.49

Effects of soil-structure interaction on the seismic response of the building considering  
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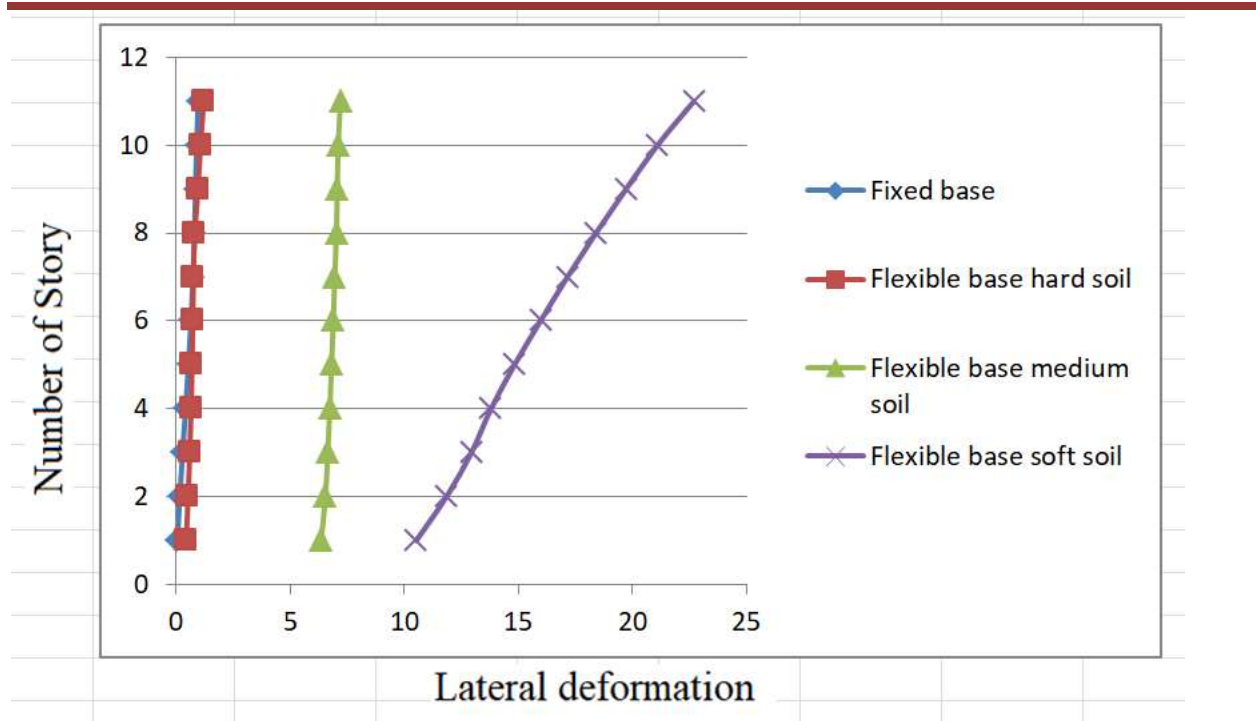


Figure 4.2 Maximum lateral deformation of the building for all base conditions.

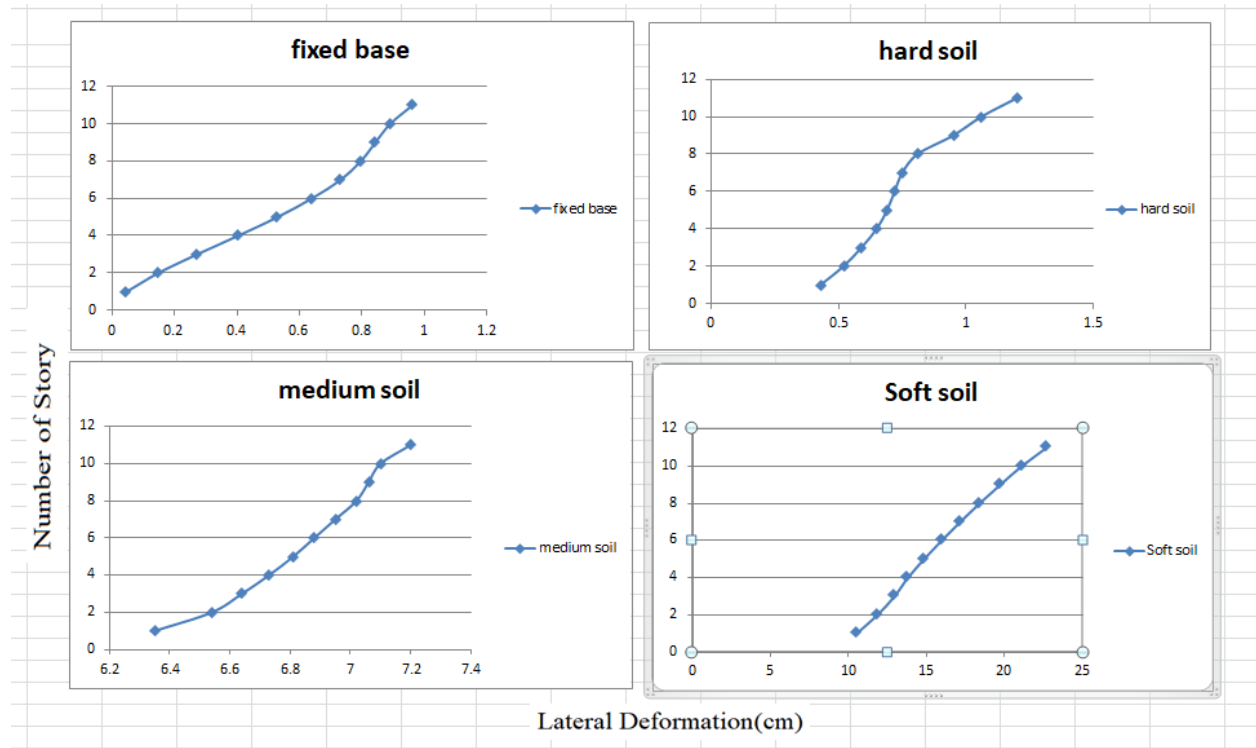


Figure 4.3 Maximum lateral deformation of the building for each base condition separately

#### 4.4 Inter-story drift

The inter-story drifts are defined as the difference between the lateral deflections of two adjacent stories divided by the height of that story.

Inter-story drifts are the most commonly used damage parameters, and based on FEMA (BSSC)[51] performance level describe the state of structure after being subjected to a certain hazard level and are classified as: fully operational, operational, life safe, near collapse, or collapse. This five qualitative performance levels are related to the corresponding quantitative maximum inter-story drifts (as a damage parameter) of  $<0.2\%$ ,  $<0.5\%$ ,  $<1.5\%$ ,  $<2.5\%$  and  $>2.5\%$ . Table 4.3 presents the corresponding maximum inter-story drift for the different types of soil. From this table maximum story drift value for fixed base, hard soil, medium soil and soft soil models is  $0.043\%$ ,  $0.046\%$ ,  $0.063\%$ ,  $0.533\%$  respectively. Based on this result it is clear that the building under study is fully operational stage for fixed base, hard soil, and medium soil case, but for soft soil case it is changed to life safe stage. The building considered in this study is hospital (vital importance for civil protection), so this type of building should be operational for the design earthquake. As we see from the drift result the soil flexibility particularly soft soils introduce the building to life safe stage from fully operational stage. This indicates the soil flexibility has significant impact on the seismic response of the building. Using the value of Table 4.3 the inter-story diagrams are plotted for better assessment as shown in figure 4.4.

Effects of soil-structure interaction on the seismic response of the building considering  
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Table 4.3. Inter story drift ratio of the structure for both fixed and flexible base subjected for Hollywood storage earthquake

Inter-story drift ratio				
Story number	Fixed base	Flexible base	Flexible base	Flexible base
		Hard soil	Medium soil	soft soil
Story 1	0.000159	0.000367	0.000433	0.001733
Story 2	0.000338	0.0003	0.000633	0.004433
Story 3	0.000413	0.000233	0.000333	0.003633
Story 4	0.000439	0.0002	0.0003	0.002867
Story 5	0.00042	0.000133	0.000267	0.003433
Story 6	0.000367	0.0001	0.000233	0.0038
Story 7	0.000304	0.0001	0.000233	0.003933
Story 8	0.000217	0.0002	0.000233	0.004167
Story 9	0.000154	0.000467	0.000133	0.0044
Story 10	0.00016	0.000367	0.000133	0.0046
Story 11	0.000241	0.000467	0.000333	0.005333

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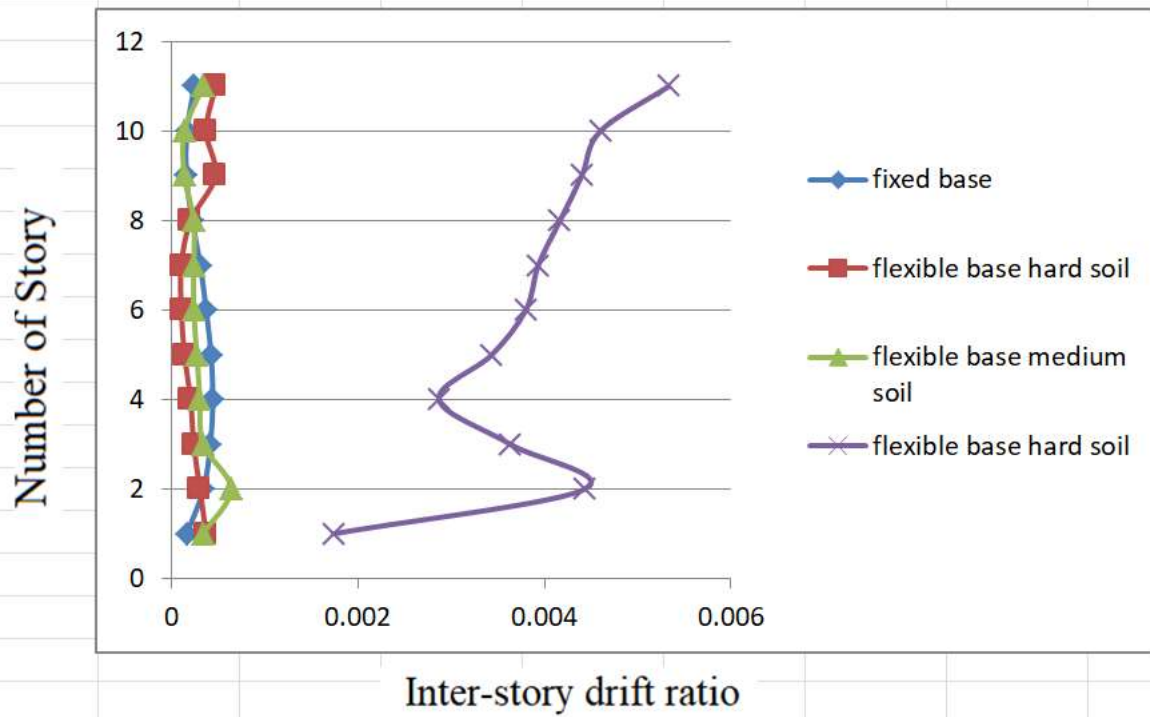


Figure 4.4 Inter story drift ratios of all base condition

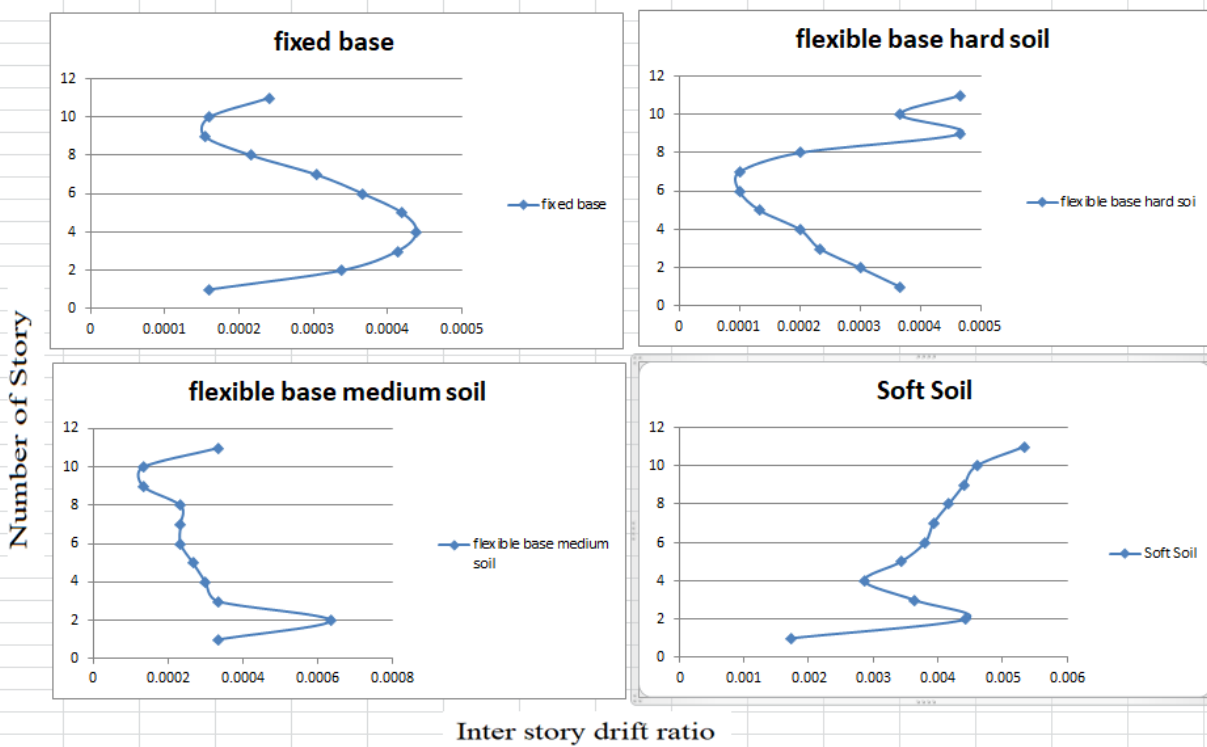


Figure 4.5 Inter-story drift ratio for each base condition separately

#### 4.5: Effect of SSI on bending moments in beams

The effect of SSI on bending moment variations along beams is assessed through systematic comparisons of results obtained for the fixed-base and flexible-base building models on selected beams. The comparisons are made for end bay beams. In reference to Figure 4.6, the comparisons are made along axis 'A' in between axis '1' and '2'. The axes and bays are selected as considerable differences in bending moments (BM) and shear forces (SF) between the fixed-base and flexible-base. The differences in BM in beams become increasing while going down the story levels from top to bottom. So, the comparisons are made for selected beams of the lowest story.

The BMDs along the selected beams as obtained from Midas software for both the flexible-base and fixed-base models for different cases are plotted together for better assessment and comparison of the effects of SSI on the structures.

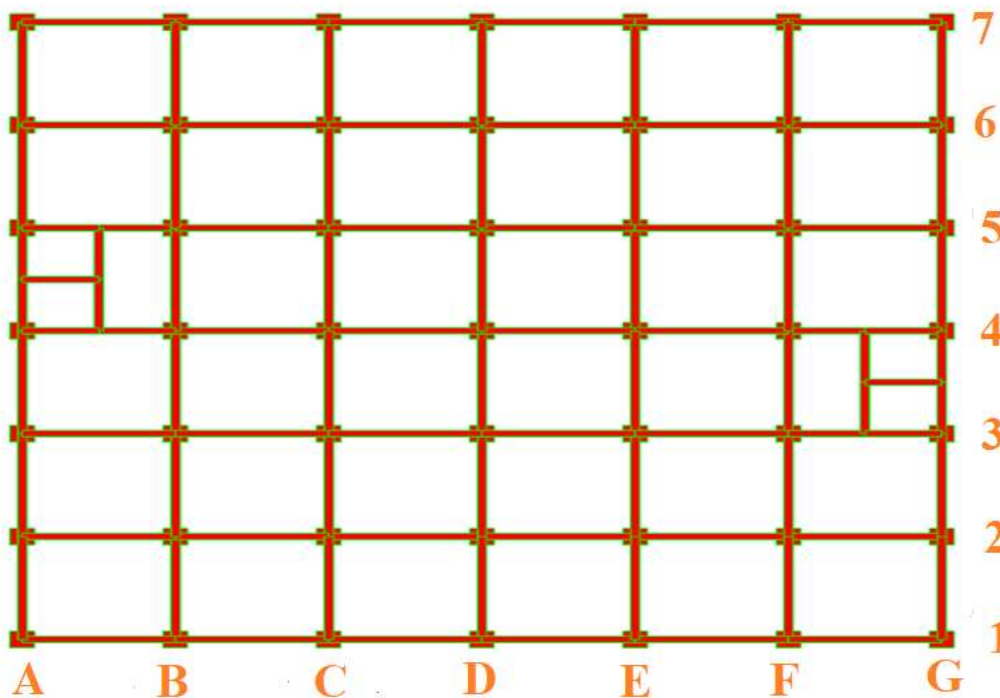


Figure 4.6: Frame layout

The bending moment diagram of the edge beam shown in Figure 4.7 illustrates the influence of various base conditions on structural behavior under lateral loading. The fixed base model exhibits a relatively consistent bending moment along the beam, with moderate values ranging between 25 and approximately 80 KNm. In contrast, the flexible base models demonstrate significant variation depending on the type of supporting soil. The flexible base with soft soil shows the most pronounced bending moment fluctuation, with extreme values reaching approximately 270 KNm at one end and 490 KNm at the other. This indicates a substantial increase in structural demand due to reduced stiffness and higher deformation at the base. The flexible base with medium soil also shows increased bending moment compared to the fixed base, though the values are more moderate, peaking around 170 KNm, suggesting intermediate flexibility and structural impact. Lastly, the flexible base with hard soil presents a bending moment trend slightly above that of fixed base, indicating minimal influence of base flexibility due to higher subgrade stiffness. This comparative analysis highlights that the softer the supporting soil, the greater the bending moment experienced by the edge beam, underscoring the importance of soil-structure interaction in structural analysis and design.

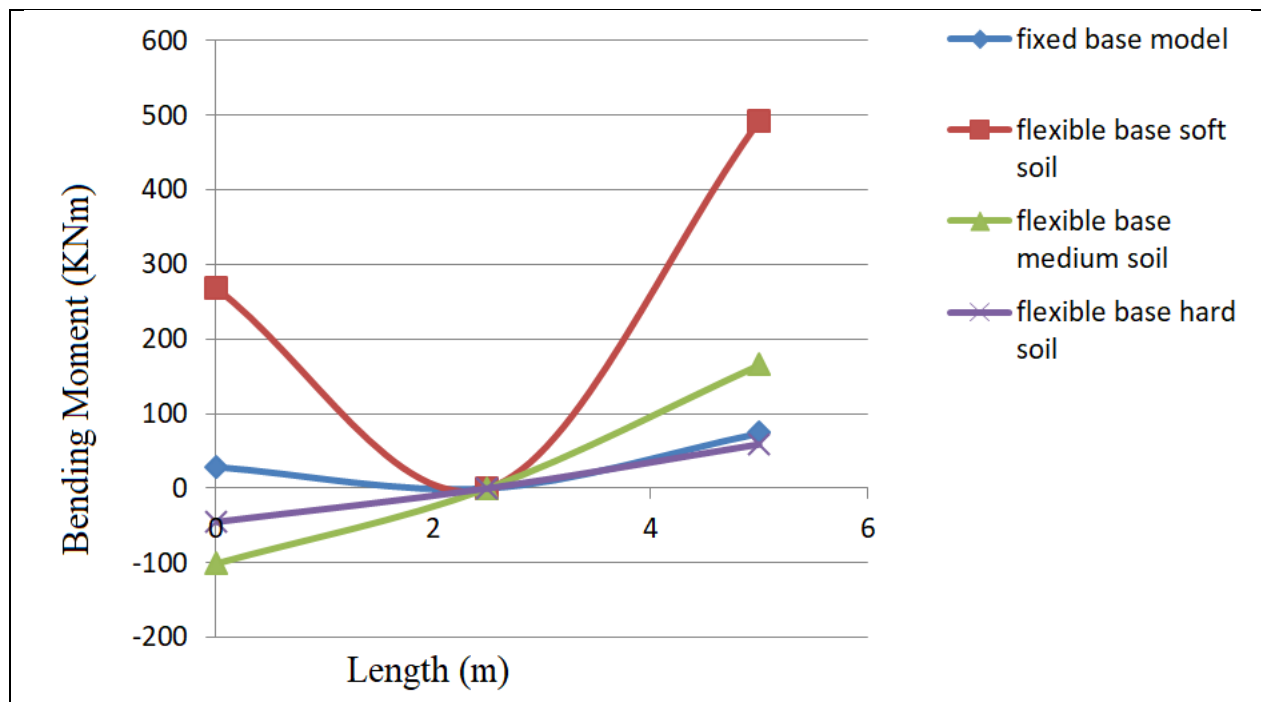


Figure 4.7 bending moment diagram of edge beam

#### **4.6 Effect of SSI on Shear Forces (SF) in beams**

The effect of SSI on shear force (SF) variations along length of beams is assessed through comparisons of Shear Force Diagrams (SFD) obtained for the fixed-base and flexible-base models. The beam elements selected for plotting SFDs is taken @ 5<sup>th</sup> story along axis 1 in between EF and FG reference to figure 4.6. The SFDs of fixed-base and flexible-base models of G+10 building are presented in Figures 4.8. The shear force diagram presented in Figure 4.8 illustrates the variation of shear force along axis 1, specifically between points EF and FG, under different base conditions. The fixed base model shows the lowest shear force values, remaining below 20 kN throughout the length, indicating minimal lateral force transfer due to the rigidity of the base. In contrast, the flexible base with soft soil exhibits the highest shear force values, ranging around 220-250 kN, reflecting a significant increase in lateral load effects due to the high deformability of the soil. This dramatic rise emphasizes the critical influence of soil softness on force distribution within the structure. The flexible base with medium soil shows moderately elevated shear forces, reaching approximately 70 kN, suggesting a balance between soil flexibility and structural resistance. Meanwhile, the flexible base with hard soil presents shear force values slightly higher than the fixed base but much lower than the soft soil case, staying around 35-50 kN. This indicates that stiffer soil conditions, limit the amplification of shear forces. Overall, the analysis clearly demonstrates that soil-structure interaction, particularly with softer soils, leads to increased shear demand on structural elements, which must be considered in the design and safety assessment of buildings.

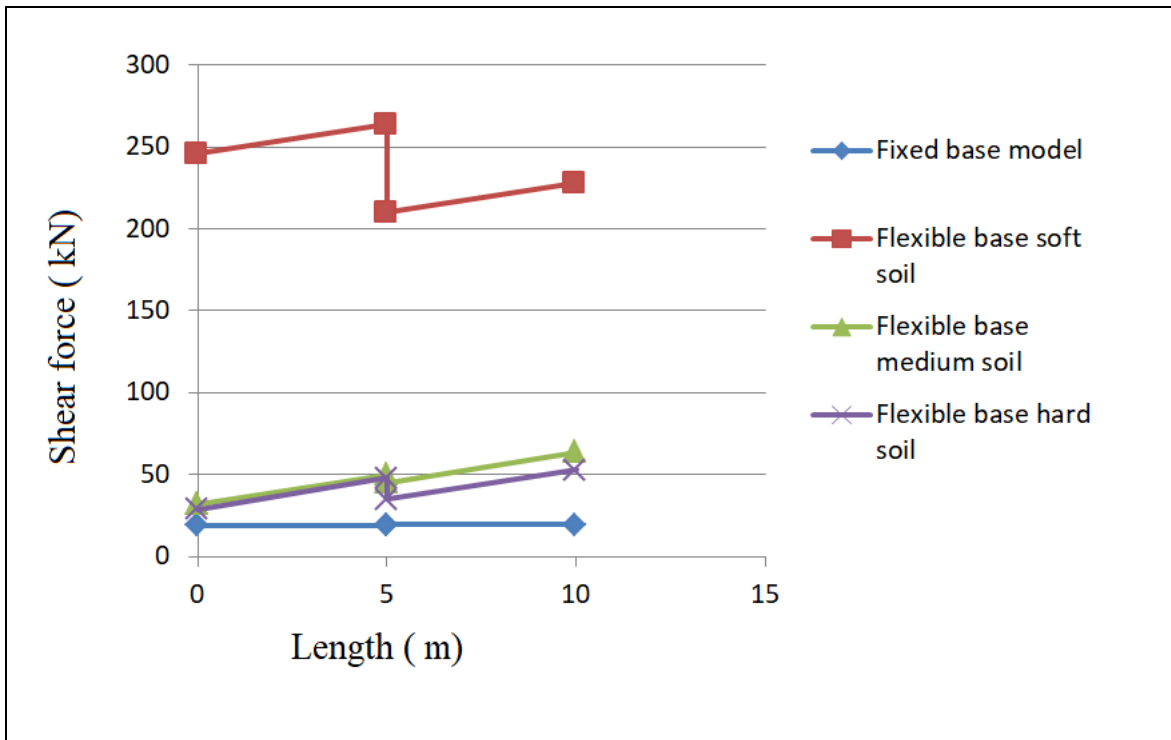


Figure 4.8 Shear force diagram along axis '1' in between EF and FG

#### 4.7 comparison of Study results with other published results

To validate the reliability of the presented results, the natural vibration period and the lateral displacement profiles obtained from this study are compared with published results by Kabtamu et al, Kanta et al, Pokharel et al, Jain and Parekar. Table 4.4 and 4.5 shows the comparison of story-wise period and displacements for the fixed-base and SSI scenarios.

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Table 4.4: Vibration Period comparison of this study with other literature

Published literature	Fixed Base	Hard Soil	Medium Soil	Soft Soil	Increase (%) ( Fixed to Soft)
Dynamic Analysis of Soil Structure Interaction Effect on Multi Story RC Frame((Kabtamu et al. 2021)  <b>Number of Story =12</b>	2.12 sec	–	–	3.5 sec	66 %
Seismic Response of RC Frame Building Considering Soil-Structure Interaction. (Kanta et al. 2021)  <b>Number of Story =10</b>	1 sec	1.65	2 sec	2.9 sec	190 %
This Study /paper	1 sec	1.49 sec	2.24	3.6 sec	260 %

Table 4.5: Top story displacement comparison of this study with other literature

Published literature	Fixed Base	Hard Soil	Medium Soil	Soft Soil	Increase (%) ( Fixed to Soft)
Dynamic Analysis of Soil Structure Interaction Effect on Multi Story RC Frame (Kabtamu et al. 2021)  <b>Number of story=12</b>	42 cm	–	–	78 cm	85.7%
Study on seismic performance of RC multi-story building considering soil-structure interaction effect (Kanta et al. 2021)  <b>Number of story =20</b>	20 cm	–	–	35 cm	75 %
Seismic Response of RC Frame Building Considering Soil-Structure Interaction. (Kanta et al. 2021)  <b>Number of Story =10</b>	1.5 cm	3 cm	7 cm	8.5 cm	466%
Effects of Soil Structure Interaction on RC Frame Building (Jain and Parekar 2018)  <b>Number of Story =7</b>	3cm	17 cm	35	65 cm	2066 %
<b>This Study /paper</b>  (The top story displacement presented here for this study accounts the foundation or base movement as discussed in section 4.3)	0.97 cm	1.12 cm	1.4 cm	12.9 cm	1228 %

## CHAPTER 5: CONCLUSIONS AND RECOMMENDATIONS

From the sensitivity study conducted through systematic comparisons of flexible-base models with the conventional fixed-base models for the different cases considered, the following major conclusions are made. Finally recommendations for future works are presented.

### 5.1. Conclusions

- The results indicate that the natural vibration period of the structure increases significantly when the soil flexibility is considered. The flexible base models exhibit longer periods compared to the fixed base model. Soft soils contribute the most to period lengthening, followed by medium soils, and finally hard soils.
- Lateral deformation increases with soil flexibility, particularly in soft soil conditions. Soft soil conditions lead to lateral deformations that are significantly higher than those observed in medium and hard soils. Fixed base models underestimate lateral deformations, posing potential risks in seismic design.
- Story drift ratios, which represent inter-story displacements, are substantially influenced by soil type. Soft soil conditions result in the highest drift ratios but Medium and hard soils show moderate increases, highlighting the varying influence of soil
- Bending moments and shear forces in beams are observed to be affected by consideration of the effects of soil flexibility. When soil-structure interaction effects are considered, both BM and SF are observed to increase with variable magnitudes along beams for all models. When the stiffness of soil decreases, both SF and BM will increase.

### 5.2. Recommendations for Future Studies

In most national seismic codes, consideration of soil-structure interaction effects in usual analysis and designs of buildings is not yet taken as a must to do procedure. So, it required to do many researches related to SSI that will motivate seismic codes and standards to consider the influence of soil flexibility on the seismic response of the building and other structures.

Accordingly for further investigation, the following ideas are recommended as important extensions of this study.

- The effects of soil flexibility on the building height variation are not covered in this study and can be taken as important extension of this study.
- This study covers only building models supported on raft foundation. So investigation of the influence of soil-structure interaction effects for models supported on another types of foundations is worth studying
- Building considered in this study is regular both in plan and elevation and considering the building irregularity is an important extension for this study.

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## Appendix A

Table A.1: Stability coefficient (Theta) of fixed base model building

Load Case	Story	Story Height (cm)	Vertical Load (kN)	Story Shear Force (kN)	Story Drift (cm)	Stability Coefficient (Theta)	Allowable Limit
EXP	11F	300	9827	791.7089	0.0929	0.0038	0.25
EXP	10F	300	19654	1627.3962	0.1087	0.0044	0.25
EXP	9F	300	29481	2379.5148	0.1265	0.0052	0.25
EXP	8F	300	39308	3048.0647	0.1445	0.0062	0.25
EXP	7F	300	49135	3633.0458	0.16	0.0072	0.25
EXP	6F	300	58962	4134.4582	0.171	0.0081	0.25
EXP	5F	300	68789	4552.3019	0.1754	0.0088	0.25
EXP	4F	300	78616	4886.5768	0.1706	0.0091	0.25
EXP	3F	300	88443	5137.283	0.1529	0.0088	0.25
EXP	2F	300	98270	5304.4205	0.117	0.0072	0.25
EXP	1F	300	108097	5387.9892	0.0524	0.0035	0.25

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Table A.2: Stability coefficient (Theta) of soft soil model building subjected for Hollywood storage earthquake

Story	Story Height (cm)	Vertical Load (kN)	Story Shear Force (kN)	Story Drift (cm)	Stability Coefficient (Theta)	Allowable Limit
11F	300	9827	817	1.6	0.19	0.25
10F	300	19654	1492	1.38	0.02	0.25
9F	300	29481	2038	1.32	0.23	0.25
8F	300	39308	2470	1.25	0.27	0.25
7F	300	49135	2800	1.18	0.32	0.25
6F	300	58962	3043	1.14	0.38	0.25
5F	300	68789	3211	1.03	0.42	0.25
4F	300	78616	3319	0.86	0.47	0.25
3F	300	88443	3379	1.09	0.49	0.25
2F	300	98270	3406	1.33	0.53	0.25
1F	300	108097	3412	0.52	0.56	0.25

Effects of soil-structure interaction on the seismic response of the building considering  
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Table A.3: Stability coefficient (Theta) of Medium soil model building subjected for Hollywood storage earthquake

Story	Story Height (cm)	Vertical Load (kN)	Story Shear Force (kN)	Story Drift (cm)	Stability Coefficient (Theta)	Allowable Limit
11F	300	9827	982	0.1	0.01	0.25
10F	300	19654	1924	0.04	0.0045	0.25
9F	300	29481	2687	0.04	0.0054	0.25
8F	300	39308	3290	0.07	0.011	0.25
7F	300	49135	3751	0.07	0.014	0.25
6F	300	58962	4090	0.07	0.018	0.25
5F	300	68789	4325	0.08	0.028	0.25
4F	300	78616	4409	0.09	0.044	0.25
3F	300	88443	4493	0.1	0.073	0.25
2F	300	98270	4530	0.19	0.22	0.25
1F	300	108097	4539	0.1	0.26	0.25

Effects of soil-structure interaction on the seismic response of the building considering  
Three Categories of soil

Table A.4: Stability coefficient (Theta) of hard soil model building subjected Hollywood storage earthquake

Story	Story Height (cm)	Vertical Load (kN)	Story Shear Force (kN)	Story Drift (cm)	Stability Coefficient (Theta)	Allowable Limit
11F	300	9827	1106	0.14	0.012	0.25
10F	300	19654	2168	0.11	0.011	0.25
9F	300	29481	3028	0.14	0.016	0.25
8F	300	39308	3707	0.06	0.009	0.25
7F	300	49135	4720	0.03	0.005	0.25
6F	300	58962	5102	0.03	0.006	0.25
5F	300	68789	5367	0.04	0.011	0.25
4F	300	78616	5536	0.06	0.023	0.25
3F	300	88443	5631	0.07	0.04	0.25
2F	300	98270	5673	0.09	0.08	0.25
1F	300	108097	5683	0.11	0.23	0.25