

SEISMIC ASSESSMENT OF BUILDINGS ON CAST IN PLACE PILES WITH
RIGOROUS MODELLING OF SOIL-PILE-STRUCTURE INTERACTIONS

A THESIS SUBMITTED TO
THE GRADUATE SCHOOL OF NATURAL AND APPLIED SCIENCES
OF
MIDDLE EAST TECHNICAL UNIVERSITY

BY
MÜGE ÖZGENOĞLU

IN PARTIAL FULFILLMENT OF THE REQUIREMENTS
FOR
THE DEGREE OF DOCTOR OF PHILOSOPHY
IN
CIVIL ENGINEERING

FEBRUARY 2025

Approval of the thesis:

**SEISMIC ASSESSMENT OF BUILDINGS ON CAST IN PLACE PILES
WITH RIGOROUS MODELLING OF SOIL-PILE-STRUCTURE
INTERACTIONS**

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ABSTRACT

SEISMIC ASSESSMENT OF BUILDINGS ON CAST IN PLACE PILES WITH RIGOROUS MODELLING OF SOIL-PILE-STRUCTURE INTERACTIONS

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Doctor of Philosophy, Civil Engineering
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February 2025, 138 pages

Construction of pile-supported high-rise buildings and large-span bridges has been steadily increasing due to rising demands in housing and transportation. Consequently, understanding the response of pile foundations under dynamic loads has become an important issue in the design of many structures. Modeling and the analysis of the soil-pile-structure systems with the consideration of SSI effects is a demanding task. Material nonlinearities of soil and structural elements, cyclic effects on the soil and pile group effects should be considered for realistic simulations. The application of the seismic loads, while considering these issues, is also a significant challenge for the dynamic analysis of the pile-supported structures. Given the challenges in these simulations, the design and assessment of these structures are generally conducted with force-displacement curves representing equivalent soil behavior in static and dynamic analysis. The kinematic and inertial interaction are considered with approximate techniques. Modeling this complex behavior with simplified sub-structuring for kinematic and inertial interactions requires a set of assumptions. The implications of these assumptions and simplified modeling techniques on the demand predictions for the pile and superstructure are also not clear. The goal of this study is to explore the dynamic behavior of a typical building

structure supported by pile foundations, with a focus on soil-structure interaction (SSI) effects, and to extract practical insights from the results. A detailed 3D soil-pile (and structure) continuum model is used to simulate the response of these systems under dynamic loads. Soil medium, multiple piles and 2D frame structure are modeled together simulating the complex nonlinearity, including cyclic effects and residual displacements. The seismic loads are applied to the system using a local domain decomposition technique proposed by Lokke and Chopra (2018). The soil, pile group and the superstructure is modelled with different modeling approaches as linear, nonlinear alternatives. The demands on the pile and structure are then derived from a series of seismic analyses and compared to assess the impact of different modeling approaches.

Keywords: Pile Foundations, Soil Structure Interactions, Nonlinear Dynamic Analysis

ÖZ

ZEMİN-KAZIK-YAPI ETKİLEŞİMLERİNİN KAPSAMLI MODELLENMESİYLE YERİNDE DÖKME KAZIKLAR ÜZERİNDEKİ BİNALARIN SİSMİK DEĞERLENDİRMESİ

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Doktora, İnşaat Mühendisliği
Tez Yöneticisi: Prof. Dr. Yalın Arıcı

Şubat 2025, 138 sayfa

Kazık destekli yüksek binaların ve geniş açıklıklı köprülerin inşası, konut ve ulaşım taleplerindeki artış nedeniyle sürekli olarak artmaktadır. Bu nedenle, kazık temellerin dinamik yükler altındaki tepkisinin anlaşılması, birçok yapının tasarımında önemli bir konu haline gelmiştir. Zemin-yapı etkileşimi (SSI) etkilerini göz önünde bulundurarak, zemin-kazık-yapı sistemlerinin modellenmesi ve analizi özel çaba gerektiren bir süreçtir. Gerçekçi simülasyonlar için zemin ve yapısal elemanların malzeme doğrusal olmayanlıkları, zemin ve kazık grubu üzerindeki döngüsel etkiler dikkate alınmalıdır. Bu sorunları göz önünde bulundurarak sismik yüklerin uygulanması, kazık destekli yapıların dinamik analizi açısından da önemli bir zorluktur. Bu tür simülasyonlardaki zorluklar göz önünde bulundurulduğunda, bu yapıların tasarımı ve değerlendirilmesi genellikle statik ve dinamik analizlerde eşdeğer toprak davranışını temsil eden kuvvet-yer değiştirme eğrileri kullanılarak yapılmaktadır. Kinematik ve atalet etkileşimleri yaklaşık tekniklerle dikkate alınmaktadır. Bu karmaşık davranışı kinematik ve atalet etkileşimleri için basitleştirilmiş alt yapılandırmalarla modellemek, bir dizi varsayım gerektirir. Bu

varsayımların ve basitleştirilmiş modelleme tekniklerinin kazık ve üst yapı üzerindeki talep tahminleri üzerindeki etkileri de net değildir. Bu çalışmanın amacı, kazık temellerle desteklenen tipik bir bina yapısının dinamik davranışını, zemin-yapı etkileşimi (SSI) etkilerine odaklanarak incelemek ve sonuçlardan pratik çıkarımlar elde etmektir. Dinamik yükler altında bu sistemlerin tepkisini simüle etmek için ayrıntılı bir 3D zemin-kazık (ve yapı) sürekli modeli kullanılmaktadır. Zemin ortamı, çoklu kazıklar ve 2D çerçeve yapı, döngüsel etkiler ve kalıcı yer değiştirmeler de dahil olmak üzere karmaşık doğrusal olmayanlığı simüle edecek şekilde birlikte modellenmektedir. Sismik yükler, Lokke ve Chopra (2018) tarafından önerilen yerel bölge ayrıştırma tekniği kullanılarak sisteme uygulanmaktadır. Zemin, kazık grubu ve üst yapı, doğrusal ve doğrusal olmayan alternatiflerle farklı modelleme yaklaşımlarıyla modellenmektedir. Kazık ve yapı üzerindeki talepler, bir dizi sismik analizden elde edilmekte ve farklı modelleme yaklaşımlarının etkisini değerlendirmek amacıyla karşılaştırılmaktadır.

Anahtar Kelimeler: Kazıklı Temeller, Zemin Yapı Etkileşimleri, Doğrusal Olmayan Dinamik Analiz



To My Grandmother

ACKNOWLEDGMENTS

I would like to express my deepest gratitude to my supervisor, Prof. Dr. Yalın Arıcı, for his guidance, advice, constructive criticism, encouragement, and insight throughout the research.

I am sincerely thankful to Prof. Dr. Kağan Tuncay and Assoc. Prof. Dr. Mustafa Tolga Yılmaz for their valuable suggestions and comments during the study. My appreciation extends to the members of my thesis examining committee, Prof. Dr. Sami Oğuzhan Akbaş and Assist. Prof. Dr. Tuba Eroğlu Azak, for their willingness to help, as well as for their constructive feedback and contributions to this study.

I am grateful to my esteemed instructors, Prof. Dr. Özgür Kurç and Prof. Dr. Ayşegül Askan Gündoğan, for their support and understanding throughout this process.

I would like to sincerely thank my friends for their continuous support, understanding, and companionship throughout this journey. The time I spent with them was always enjoyable and meaningful to me.

Finally, I am deeply grateful to my whole family—especially my sister, my father, and my grandmother—for their endless love, patience, and encouragement. Most of all, I lovingly remember my late mother, whose memory continues to inspire and guide me every day. Their unwavering belief in me, even during the most challenging moments, has been a constant source of motivation and strength.

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CHAPTER 1

INTRODUCTION

1.1 General

The response of a structure to seismic loads depends on the interaction of a triple system: the structure itself, the foundation supporting the structure, and the ground surrounding the foundation (FEMA, 2009). Seismic soil-structure interaction (SSI) is defined as the collective response of the structure-foundation and the soil medium to a specific incident motion. The response of a structure, in case of the theoretical condition of a rigid foundation and rigid soil strata, corresponds to the behavior of the structure in the absence of the SSI effects.

Seismic interaction, as such is classified into two parts, namely, the inertial and the kinematic interaction mechanisms. The inertial interaction refers to the arising displacements on the soil-foundation system induced by the inertial forces acting on the superstructure. During seismic excitation, inertia on the superstructure results in to increase base shear, moment and torsional effects on the foundation. Forces caused by inertia produce deflections and rotations at the soil-foundation interface. These deflections cause to energy dissipation by the means of radiation damping and hysteretic soil damping (NEHRP 2012). Inertial interaction also recognizes the movement of some soil/rock mass with the structure, leading to an increase in flexibility (periods) and energy dissipation (damping) of the system compared to an identical counterpart on fixed base: the demand on the structure is therefore changed.

Kinematic interaction, on the other hand, is the effect of the system on the foundation motion itself (or vice-versa), system changing the behavior of the ground strata compared to the free field vibration. The term "free field motion" is used for the motions on or through the soil in the absence of the superstructure and foundation.

In other words, existence of the foundation and the structure affects the input motion for the seismic analysis. The kinematic interaction is mainly caused by the base slab averaging, wave scattering and the embedment effects (NEHRP 2012).

Structural modeling and analysis of a superstructure under dynamic loads with the aforementioned SSI effects is a challenging problem. Several techniques have been suggested in research studies and design codes to take into account the SSI features in these analyses. The information regarding the SSI applications in research work are very technical and difficult to understand, while the guidance in the design codes and standards is rudimentary for practical engineers.

Response prediction of the structures having pile foundations constitutes an important part of the soil-structure interaction issues since the construction of pile supported structures like oil-gas platforms, wind turbines, high rise buildings and large span bridges has been steadily increasing with the increase in the housing, transportation and energy production demands. While the response of soil-pile-structure systems has been an actively studied in the last 4 decades, the recognition of seismic effects and the need for proper design and evaluation procedures considering SSI is more recent.

The need for the proper representation of the soil-pile response originated mainly from the energy sector, with the design of such systems for oil platforms as the main concern (API, 1987). The initial concern was the static lateral loading, from the structure or pile cap and its effect on the flexibility of the system. The traditional design approach based on these studies is still continued for buildings and urban infrastructure. Seismic loads are generally treated with identical models in the customary procedures. On the other hand, seismic loading is different in the sense that loading and the system properties are interrelated leading to a dynamic interaction between the structure, foundation and the embedment media. Recognizing these complexities, and considering the lack of documentation and uncertainties in this area, the goal of this study is to explore the detailed behavior under seismic conditions.

1.2 Scope and Objective

The primary objective of this study is to evaluate the dynamic response of a representative building structure supported by pile foundations, with a specific focus on the influence of soil-structure interaction (SSI) effects. The research aims to provide a comprehensive understanding of how soil and structural nonlinearities affect the engineering demands of such systems, including displacements and internal forces.

In addition to investigating the effects of nonlinear behavior in both the soil and the superstructure, the study also examines the role of cap beam stiffness in influencing the structural response. Cap beam stiffness is a critical factor in load transfer mechanisms between the superstructure and pile foundation, and its impact on dynamic behavior under seismic loading remains an area requiring further exploration. Within this scope;

- A detailed three-dimensional model of a soil strip, along with a two-dimensional frame representing the building structure supported by a pile foundation, has been developed. The composite system is analyzed under varying conditions to assess the effects of soil nonlinearity, structural nonlinearity, building height, cap-beam rigidity, and seismic intensity.
- The impact of nonlinearity on engineering demands has been observed through different modeling approaches such that elastic soil with elastic superstructure, nonlinear soil with elastic superstructure, and nonlinear soil with nonlinear superstructure - considering both elastic and nonlinear properties.
- The effect of cap beam stiffness has been investigated using nonlinear soil-elastic structure models with varying cap beam elastic modulus.
- Two separate building models with different numbers of stories are used in the analyses. This allows to observe the nonlinearity effects with respect to building height.

- The soil has been modeled using a finite element approach, with transmitting boundaries defined at the edges. Seismic loading has been applied by defining effective earthquake forces at these boundaries (Joyner and Chen, 1975), using the procedure provided in Lokke and Chopra (2018).
- Twenty-four different ground motions recorded on stiff soil have been selected based on the criteria of moment magnitude, shear wave velocity, and distance to the causative fault. The selected records have been scaled using eleven different scale factors to achieve an average spectrum PGA value ranging between 0.1 and 0.6 g, with increments of 0.05 g. Using the selected records and scale factors, each model has been analyzed with 264 different earthquake scenarios. A total of 2640 dynamic analyses have been carried out.

The novel contributions of this study for seismic analysis of pile foundation structures:

- The use of a direct analysis method for modeling the combined soil, pile foundation, and superstructure has enabled the simultaneous representation of the nonlinearities in both the soil and the superstructure, which is not a common modeling and analysis approach in engineering practice.
- The effective earthquake forces has been successfully calculated and applied to the system using the technique specified in the aforementioned study, which facilitated more realistic simulations of seismic loading.
- The effects of soil and structural nonlinearity, as well as the influence of cap beam stiffness have been explored within the framework of a comprehensive soil-foundation-structure model. This study aims to contribute to the existing body of knowledge by providing further insights into these interactions.

The limitations of this study are presented as follows:

- To reduce the computational burden, the three-dimensional soil-pile foundation and superstructure system has been simulated by a 3D soil strip model and a single frame of a typical building, with three bays.
- The three dimensional system has been subjected to unidirectional dynamic loads, vertical motions are not considered.

This thesis is structured into six chapters. Chapter 2 provides a comprehensive review of the literature on seismic analysis methodologies for pile foundation structures, alongside experimental studies investigating the behavior of piles under cyclic loading. Chapter 3 details the modeling approaches and analysis methodologies employed in this study, while Chapter 4 is dedicated to the description of the case study. The outcomes of the dynamic analyses for the Mid-Rise building structure are thoroughly presented and discussed in Chapter 5. Finally, Chapter 6 concludes the thesis by summarizing the key findings and implications of the research.

CHAPTER 2

LITERATURE RESEARCH

Modeling and analysis of soil-pile foundation and structure systems under dynamic loads present a multifaceted challenge due to the intricate interactions between system components, along with nonlinear effects. The complexity of the problem mainly arises from the material nonlinearities of soil and structural elements, geometric nonlinearities like separation and gapping phenomena at the soil-pile interface or soil cave-in, and the degradation or hardening of soil strength and stiffness. To address these challenges, various analytical and numerical methods have been developed, ranging from simplified approaches to highly detailed and sophisticated models, alongside research on the response of soil and pile foundations under dynamic loads. This section provides a summary of research on the behavior of soil and pile foundations under cyclic/dynamic loads and includes the modeling and analysis methods for pile foundation structures. First, the analysis methods and the current practice existing in several design codes will be explained and then, findings regarding to the experimental and analytical studies for pile behavior under cyclic loading will be summarized.

2.1 Analysis of Structures with Pile Foundations

Dynamic analysis of pile foundation structures with SSI considerations is performed by following the sub-structuring or direct analysis approaches. In sub-structuring method, the complete system of soil, pile, and structure is divided into two separate models: the soil-foundation model and the foundation-superstructure model. On the other hand, in a direct analysis method, the soil-foundation-pile and the structure is built within the same model and analyzed in a single step. For this reason, the SSI effects are taken into account inherently. In addition, since the system is analyzed as

a complete model, the response analysis can be carried out by defining the nonlinear features of the soil and structural elements. Linear, equivalent-linear or complex nonlinear models may be used for modeling of the foundation material behavior. The use of equivalent linear soil properties is more common, as defining the soil with nonlinear features, in addition to the overall size of the model and the presence of the complex geometric nonlinearities, increases the computational burden significantly. From a computational difficulty perspective direct solution of the SSI problem is not preferred in engineering practice (NEHRP, 2012).

In sub-structuring analysis methods, the complete system is analyzed using two distinct models including the soil-foundation-pile and foundation-structure separately. For rigorous analytical solutions, the implementation of sub-structuring method requires the assumption of linear behavior for soil and the structural components due to the use of superposition approach (Ostadan, 2006). The sub-structuring involves a combination of the analysis of the foundation system (with the kinematic interaction) and the response analysis of the structure modelled with the simplified foundation model using the foundation input motion.

For an adequate consideration of SSI effects in sub-structuring approach, the followings are required: (i) assessment of free-field soil motions and corresponding soil material properties; (ii) determination of transfer functions to convert free-field motions to foundation input motions; (iii) inclusion of springs and dashpots to represent the stiffness and damping at the soil-foundation interface; and (iv) response analysis of the combined structure-spring/dashpot system applying the foundation input motion.

The use of sub-structuring with combination of results from linear and nonlinear analysis is an approximation to the expected solution conducted for decreasing the computational burden. The sub-structuring analysis method can be found advantageous in terms of handling the systems separately, focusing on the most significant parts of the problem in each step and continuity with the general practice

in building design regarding the superstructure and sub-foundation design separately.

Direct analysis and the sub-structuring methods can be carried out by different discretization methods with different degrees of complexity depending on the requirements of the analysis and the computational tools. In other words, soil-pile foundation part of the complete system can be represented in two ways, namely Beam on Winkler Foundation (BWF) method and continuum based methods.

In continuum-based modeling approaches, the soil medium surrounding the piles is represented as a continuous medium. Appropriate material properties are assigned to this medium, along with proper interface elements at the soil-pile surface and transmitting boundaries at the limits of the soil. On the other hand, Beam on Winkler Foundation (BWF) method, soil layers are represented by a combination of spring and dashpot elements at each depth. Continuum based approach and Beam on Winkler Foundation method are illustrated on a complete model of soil-pile-structure system in the Figure 2.1. In the following sections, basic information and findings regarding to these two analysis methods are summarized.

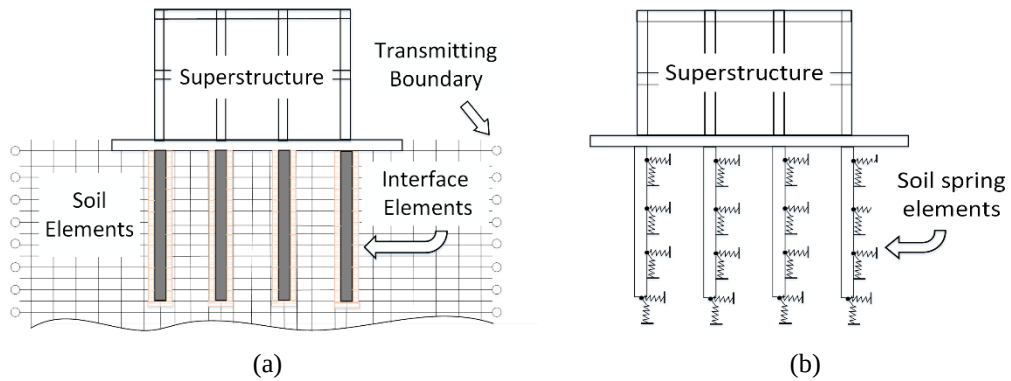


Figure 2.1. Simple illustration for (a) continuum modeling approach and (b) discrete element approach

2.1.1 Beam on Winkler Foundation Approach

The Beam on Winkler Foundation (BWF) method is widely used for modeling soil-pile interaction by representing the soil as a series of uncoupled one-dimensional spring elements along the pile length in both vertical and lateral directions. The BWF modeling approach requires the definition of the behavior of spring elements, i.e. the relations between the soil resistance-pile displacement in lateral (p-y curves) and in vertical (t-z and Q-z curves) directions, properly. The BWF procedure is commonly called as also the “P-y method” taking the name of the relationship between the soil reaction (p) and the lateral pile deflection (y). The modulus between the lateral displacement and the soil resistance is called the subgrade reaction modulus which depends on the foundation size, depth, and the soil type. A representative p-y curve is shown in the Figure 2.2. The subgrade reaction modulus is constant for the small deflections and decreases with the increase in the deflection.

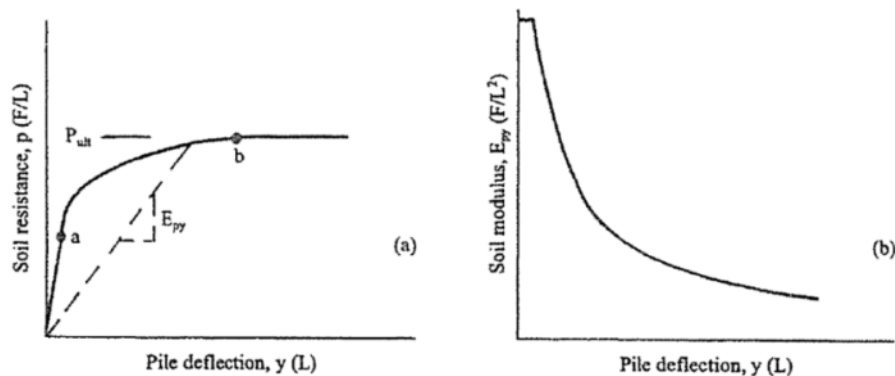


Figure 2.2. (a) Representative P-y curve and (b) Soil modulus-pile deflection
(adopted from Reese and Van Impe, 2001)

The soil resistance-pile displacement relations, particularly the p-y curves are obtained from the back analyses of the results of monotonic and cyclic loading tests and particular for the considered depth of the pile and for the soil type. For the lateral static analysis, static p-y curves have been developed for cohesive (Matlock 1970,

Reese et al. 1975, Reese and Welch, 1975, Bushan et al. 1979) and cohesionless soil (Reese et al. 1974, Cox et al. 1974) over the years.

While static p-y curves are well-established and widely used in the analysis of pile foundations under lateral static loading, the development of dynamic p-y relationships for lateral dynamic analyses remains an active area of research. The p-y curve approach suggested by API (1989) has been widely used as a backbone curve for analyzing laterally loaded piles under both static and dynamic conditions due to its simplicity. However, the API approach has been identified with significant limitations particularly for dynamic loading scenarios such as high initial modulus (Boulanger et al. 1999, Finn and Fujita 2002).

The behavior of soil-pile systems under dynamic conditions differs significantly from static conditions due to the complex effects of cyclic and dynamic loading. Under cyclic loading conditions, the relationship between soil resistance and lateral deflection evolves due to the repetitive application of forces. In Figure 2.3, the static and cyclic p-y curves obtained from full scale tests carried out by Reese et al. (1975) in stiff clay are given for different depths. The decrease in the soil resistance can be obviously seen in the cyclic loading case.

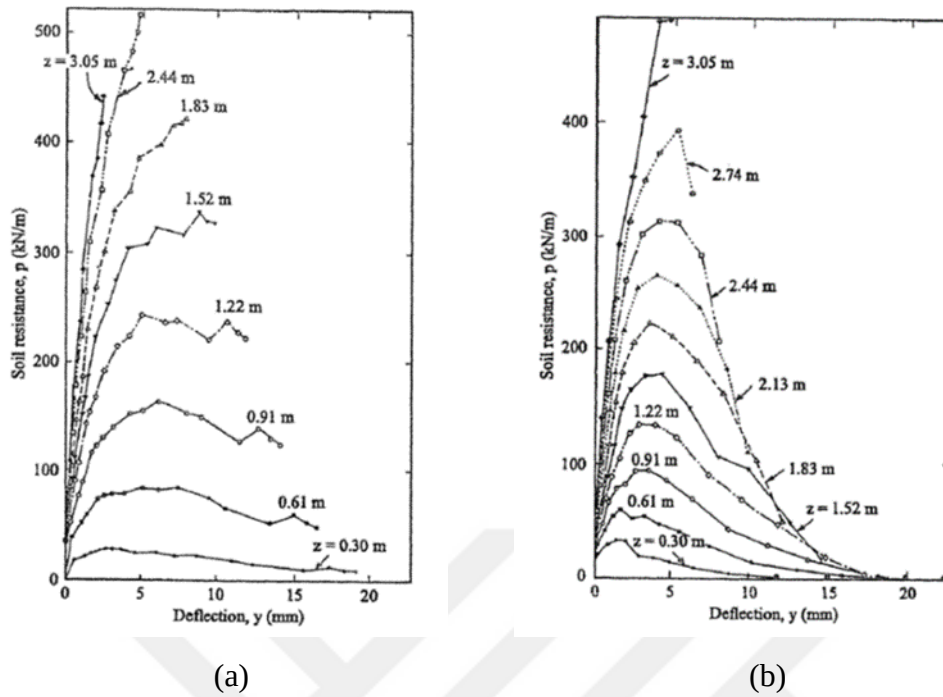


Figure 2.3. P-y curves under static (a) and cyclic (b) loading tests in stiff clay
(adopted from Reese et al. 1975)

In addition to variations in soil resistance due to cyclic loading, critical factors such as nonlinear hysteretic behavior, damping mechanisms, and the phenomenon of gapping at the soil-pile interface profoundly influence the dynamic response of soil-pile systems. Original BWF model does not consider the nonlinear features of the soil-pile interaction during cyclic (dynamic) loadings. As a result, dynamic p-y curves have been a subject of extensive research for several decades, with researchers proposing various idealized models to capture these complex interactions.

The foundational research on dynamic p-y curves is attributed to Matlock et al. in the 1970s. Their work proposed modifications to static p-y curves by incorporating the influence of cyclic loading, which can reduce the stiffness and strength of clay. Later, Matlock et al. (1979) incorporated the Beam on Non-linear Winkler Foundation (BNLWF) model into a computer program named SPASM for the dynamic analysis of offshore structures. These approaches established the basis for

further advancements in dynamic p-y formulations through both experimental and numerical studies.

Full-scale field tests (Ting et al., 1987; He et al., 2017) and small-scale pile tests (McVay et al., 1994; Rosquoët et al., 2004; Rovithis et al., 2009; Yang et al., 2011; Hong et al., 2013; Yoo et al., 2012, 2013; Lee et al., 2019) have been conducted extensively to investigate the response of pile foundations under cyclic and dynamic loading conditions. These studies have significantly contributed to understanding the key factors influencing soil-pile interaction, including soil type, pile flexibility, loading characteristics, and degradation effects under repeated loading cycles. Full-scale field tests provide direct insights into the behavior of piles under in-situ conditions, incorporating the complexities of soil stratification, groundwater effects, and large-scale boundary conditions that cannot be fully captured in laboratory settings. For instance, Ting et al. (1987) and He et al. (2017) examined the response of piles subjected to lateral cyclic loading in different soil profiles, offering valuable data for validating numerical and analytical models. On the other hand, small-scale model tests have allowed researchers to conduct controlled parametric studies, isolating specific influencing factors such as cyclic degradation, stiffness reduction, and energy dissipation mechanisms.

The experimental investigations have contributed to refining empirical and theoretical frameworks for soil-pile interaction under cyclic and dynamic loads. Based on the experimental studies semi-experimental dynamic/cyclic p-y curves have been derived by fitting experimental data to mathematical functions. This curve-fitting process is guided by fundamental soil mechanics principles and engineering judgment, ensuring that the resulting relationships remain physically meaningful. The experimentally obtained p-y curves settle a comprehensive understanding on the soil-pile interaction under cyclic (and dynamic) loading and constituted a reference basis for dynamic p-y relations.

As an alternative to these experimental based dynamic p-y curves, semi-theoretical methodology for developing cyclic p-y curves has also been explored. In this

approach, a mathematical model -often referred to as a p-y "macro-element"- is formulated based on theoretical considerations. The p-y macro elements consists of spring and dashpot components which represents the components of soil-pile interactions during dynamic loadings such as nonlinear hysteretic behavior, damping and gapping at the soil-pile interface. The model parameters are calibrated using experimental data from full-scale tests, centrifuge experiments, or even high-fidelity 3D numerical simulations when available. Validation against independent test results is a crucial step in ensuring the reliability of such models before they can be applied in engineering practice. There are a number of p-y macro element models in literature (Nogami et al., 1992, Boulanger et al. 1999, Curras 2000, Gerolymos and Gazetas 2005, Taciroglu et al. 2006, Allotey and El Naggar 2008, Heidari and El Naggar, 2016).

The BWF models often require application of displacement loading at the fixed end of the p-y element: the uncertainty in the determination of free field motions using 1D site response analysis for use in these models can be significant (Boulanger et al., 1999). The effect of the frequency of inertial loading on the dynamic loading was considered for the generation of dynamic p-y curves in El Naggar and Bentley (2000); recent studies showed dynamic p-y curves to be affected significantly by the relationship between the natural frequency of the soil-pile system and the loading frequency, the amplitude of the acceleration as well as the confining stress in addition to these factors (Lim and Jeong, 2018).

Phasing of the inertial and kinematic effects is an important question especially for substructuring approach (with BWF models) to work for prediction of demands on pile systems. Phasing depends on the pile foundation stiffness, ground motion characteristics and superstructure period (Boulanger et al., 2007). In-phase behavior observed in some tests (centrifuge models (Brandenberg et al., 2005; Chang et al., 2005) and large-scale shaking table tests, (Tokimatsu et al., 2005)) is attributed to structures with natural periods of 0.8 s or less in soil profiles whose equivalent natural periods in the range of 1 to 2 s. Inertial and kinematic effects can be out-of-phase if for example the structure's natural period was larger than the ground's

natural period (Tokimatsu et al., 2005). Out-of-phase behavior is also possible: Boulanger et al. (2007) suggests the simultaneous application of ultimate lateral ground displacements and an appropriate fraction of peak inertial loads to predict the dynamic phasing of the inertial and kinematic effects underlining the phase effect.

In BWF method, representation of soil behavior with spring elements ensures that the model remains relatively compact, thereby facilitating computational efficiency. Due to the computational ease of the method, the traditional approach to modeling the soil-pile system, the Beam on Winkler Foundation (BWF) approach, is the general norm of practice in the industry. In addition, since the soil resistance-pile displacement relations, particularly the p-y curves, are obtained from the results of static and cyclic loading tests, these curves indirectly take into account the geometrical nonlinearities appearing in the soil-pile interface which may be a compelling task for the finite element modelling (Gerolymos et al., 2009).

However, BWF models have some disadvantages. Since the spring elements used in BWF method are independent from each other, the method is not found sufficient to handle the response of three dimensional continuum systems. Moreover, the p-y curves may not be representative for every loading scheme, soil conditions and the pile properties etc. For this reason, the need for calibration of the modeling parameters for a particular site characteristics and geometry of the system is considered as another disadvantage of such models. As presented above, the use of BWF models and sub-structuring are deficient in predicting the acceleration response for the superstructure, they are still commonly used due to the acceptable prediction of the bending moment distribution along the pile (Rahmani et. al., 2012). The well-known difficulty in modeling group effects as well as interaction along the depth of the pile should also be noted.

2.1.2 Continuum based Modeling Approach

Continuum-based modeling of soil-pile-structure systems has gained increasing attention in recent decades, particularly due to advancements in computational technology. While the Beam on Winkler Foundation (BWF) method has been widely used for many years, continuum-based methodologies have become more prevalent since the 2000s. These models employ numerical formulations such as the finite element method, the boundary element method (Padron et al., 2007; Liang et al., 2013; Stacul and Squeglia, 2017), and the finite difference method (Stevens and Krauthammer, 1988). Among these, the finite element method (FEM) is most commonly applied, incorporating both equivalent linear and advanced constitutive soil models.

In finite element analysis, the soil surrounding the pile is discretized in three dimensions. Therefore unlike the BWF method, continuum modeling provides representation of the three-dimensional nature of the interaction between the pile and the surrounding soil necessitating substantial computational effort. While computational cost remains a major challenge, analytical difficulties persist. Geometric nonlinearities at the soil-pile interface require advanced contact elements to accurately modeling the pile-soil separation and gap formation. The use of advanced material models based on plasticity theory enables the representation of nonlinear soil behavior by capturing the stress-strain response under different loading conditions. Constitutive models often necessitate intricate calibrations with non-physical parameters, complicating their application to simulate hysteresis and softening behavior accurately. Moreover, the need for case-specific model calibration limits the widespread application of continuum-based modeling to large-scale infrastructure projects rather than standard engineering design and evaluation processes. Comprehensive simulations from installation to seismic loading often necessitate different computational tools at different stages, adding to the difficulty. Additionally, effective simulation of wave propagation requires the incorporation of absorbing boundary conditions. To address these complexities, both simplified and

detailed continuum models have been developed for analyzing the dynamic behavior of pile foundations under cyclic and seismic loading.

Several studies have demonstrated the advantages of continuum-based modeling in capturing complex soil-pile interactions. Giannakos et al. (2012) validated a finite element model for the cyclic lateral response of single piles and pile groups in dry dense sand using centrifuge test data, assessing key aspects such as pile-soil interaction, plastic shakedown, and pile group efficiency. Their findings highlighted the capability of 3D models to represent system densification effects under cyclic loading. Similarly, Wang et al. (2014) developed a 3D finite element model to simulate the seismic response of piles in liquefiable ground, incorporating a plasticity-based liquefaction framework validated against centrifuge shaking table tests. Their study examined the influence of pile cap effects, lateral spreading, and non-liquefiable crust layers on pile behavior, underscoring the importance of capturing complex interactions that are often overlooked in conventional p-y models. Expanding this work, Wang et al. (2016) emphasized the significance of pile-group interactions, which are not accounted for in beam-on-Winkler-foundation (BWF) models, noting the scarcity of field data and theoretical solutions for pile groups with varying spacing configurations. The effects of pile-cap stiffness and embedment on inertial and kinematic loading phase relationships was investigated, offering insights that contrasted with findings from Tokimatsu (2005).

Cheng and Jeremic (2009) conducted finite element simulations to analyze the response of single piles embedded in liquefiable soil. Their study incorporated staged loading procedures and an elasto-plastic soil model to enhance liquefaction response accuracy. Similarly, Jeremic et al. (2009) utilized finite element modeling to develop a high-fidelity numerical model of a bridge system under seismic loading, employing the Domain Reduction Method (Bielak et al., 2003) for improved earthquake wave propagation simulation. Chu and Truman (2004) investigated the effects of pile configuration and spacing on pile head response using a dynamic finite element model incorporating soil nonlinearity and an infinite element boundary for energy

dissipation. More recently, Tipsunavee and Arangjelovski (2021) examined the seismic response of high-rise buildings on pile foundations in soft soil, incorporating a flexible wall barrel container as an absorbing boundary condition to align with calibrated shake table experiments. The study of Çetindemir and Zülfikar (2024) presents a numerical validation study of fully coupled nonlinear seismic soil–pile–structure interaction (SPSI) models. The research aims to address critical numerical modeling issues and validate new OpenSees elements against centrifuge test results.

It should also be noted that continuum-based modeling has significantly advanced soil-pile-structure interaction analysis by providing a simulation platform to apply traditional methods to the problem. Lok et al. (2000) introduced a hybrid finite element approach that integrates the nonlinear BWF model into the finite element software GeoFEAP (Bray et al., 1995) to simulate soil-pile-superstructure interaction in a coupled manner. Their study demonstrated that coupled finite element methods provide a more accurate representation of soil-pile-structure interaction compared to uncoupled approaches. Ozutsumi (2003) proposed a simplified finite element framework that transforms three-dimensional soil-pile interaction into a two-dimensional representation using soil-pile interaction springs. Hussien et al. (2011) validated this simplified approach for seismic soil-pile-structure interaction (SSPSI) analysis in the frequency domain, confirming its effectiveness in capturing inertial and kinematic loading effects.

Collectively, these studies highlight the ability of continuum-based approaches to capture stress distributions along piles, pile-group effects, radiation damping, kinematic interactions, and directional loading effects, reinforcing their increasing preference over traditional modeling techniques. Despite the challenges and disadvantages associated with continuum-based modeling, this approach is increasingly favored over traditional p-y models due to its ability to capture critical factors influencing soil-pile response, including soil constitutive behavior, pile properties, soil-pile interface interactions, loading schemes, boundary conditions, and soil-structure interaction (SSI) effects. Advances in computational capabilities

have further contributed to the feasibility of three-dimensional (3D) continuum analysis, making it a more practical alternative compared to a decade ago.

The following sections summarize code based methods and practical approaches in literature for modeling and analysis of soil-pile systems.

2.2 Current Practice for Pile Foundation Structures

2.2.1 Practical Approaches in Literature for Predicting Pile Response

As the continuum analysis for prediction of pile-structure interaction is rather costly, it is seldom used in practice, especially for building structures. Consequently, a range of practical approaches for the prediction of demands on piles have been suggested in the literature. These approaches have not been codified and appear to be used at the discretion of the practicing engineer.

A general approach can be found in Poulos (2013) focusing on computing the additional loads arising from the seismic excitations rather than the overall structural response of the soil-pile-structure system. Several simple methods for computing the inertial and kinematic loads separately are overviewed. Inertial forces are generally obtained as a factor of the axial load acting on the pile (site specific or code based spectrum). For inertial loading, elastic analysis based Randolph (1981) is suggested for the calculation of maximum bending moment for both the free-head and fixed-head piles. For the estimation of the maximum moment developing from the kinematic loading, two distinct approaches are presented (1. Dobry and O'Rourke (1983), Nikolaou et al. (2001) and 2. Tbesh and Poulos (2001)) Simple formulas are used to estimate pile moments in (1) while free field site response analysis is used to compute the kinematic demands by a pseudo static approach in (2). The maximum displacement values obtained from the free field analysis is defined along the pile which is, at the same time, the procedure recommended by 2018 Turkish Earthquake Code as Method-III.

Group effects are usually considered with reduction factors in the analyses. Under inertial loading, this well-known effect is modeled by reducing the stiffness of piles within the group and decreasing the overall group capacity (Poulos and Davis, 1980 and Fleming et al., 2009). However, under kinematic loading, group effects tend to be beneficial, inner piles within a group tend to be subjected to smaller forces and moments developed by ground movements than outer piles. Dynamic group interaction involving the complete behavior of pile-foundation system is complex and can increase or decrease the demand on piles depending on the frequency of the loading and the spacing between the piles. For practical design, the use of an SSI solution by Gazetas (1991) is advised simplifying the group as an equivalent pier with the corresponding impedances.

2.2.2 EuroCode 8

The Eurocode – 8 (2004) explicitly indicates the conditions in which the SSI effects are required to be considered. The SSI effects are taken into account in the design of structures in which the second order effects are significant, for structures having massive or deep seated foundations, for slender-tall structures and for structures built on very soft soils. The kinematic and inertial interaction forces are advised to be taken into account without providing modeling guidelines. However, the kinematic interaction is required to be considered as long as the specific circumstances related to soil profile, seismic zone and the importance level of the structure occur simultaneously. Discrete or continuum models can be used with no specific suggestions. Proper handling of the flexural stiffness of the pile, soil reactions along the pile under cyclic loading effects, pile group effects and the boundary condition at the pile-structure connection are required. The piles are suggested to be designed to remain elastic but plastic hinge formation at their heads is permitted for certain conditions.

2.2.3 Japanese Standards

Japanese guidelines proposed for concrete structures (JSCE, 2007) requires the consideration of SSI effects if the interaction effects are significant, but it does not clearly indicate the necessary conditions for including the interaction effects. The code prescribes direct modeling of the soil and structure using finite element method and conducting time history analysis with the consideration of the nonlinearity in the structural members and the soil for seismic response analysis (Maki et al. 2008; Sharma et al., 2018).

The standards proposed for highway bridges, railway facilities and building foundations provide detailed instructions and specifications for the design of piles embedded in liquefiable and lateral spreading grounds. The kinematic and inertial interaction forces are considered by using BWF models for both the liquefiable and the lateral spreading ground cases. However, the specifications differ from each other in terms of estimating the kinematic effects, coefficient of subgrade reactions and the reduction factors estimated for the coefficient of subgrade reactions of the soil (Uchida and Tokimatsu, 2006). For liquefiable soils, in the Specifications for Highway Bridges (JRA, 2002), the kinematic effect on piles are not considered but the soil subgrade reaction is reduced depending on the several parameters such as the safety factor for liquefaction, liquefaction strength ratio, depth of the soil and the earthquake level. On the other hand, the Design Standard for Railway Facilities (RTRI, 1999) considers these (kinematic) forces by describing an additional inertia force at the top of the pile as well as using reductions for the coefficient of subgrade reaction. In the Recommendations for Design of Building Foundations (AIJ, 2001), kinematic effects are considered using the accumulated shear strain profile given for the liquefiable soil layer. The subgrade reaction (reduced as before) is assumed to be identical for inertial and kinematic loadings.

2.2.4 American Petroleum Institute (API) Recommendations

In the API Working Stress Design (2000) guidelines, the general rules and concerns for both the discrete element and the continuum model approaches are expressed. The guidelines do not provide detailed information for modeling of kinematic or inertial interaction. The use of discrete model approach i.e. modeling the soil with uncoupled spring elements, is more emphasized. The main concern is expressed as ensuring the proper representation of the nonlinear soil response. API provides simple formulations for p-y, t-z, Q-z backbone curves for BWF modeling, concerns related to loading effects incorporated into the backbone curves. In practice, these p-y curves are referred by most of the design guidelines and applied for the various pile types. The API curves are not found successful to represent the major effects of soil-structure interaction (Rahmani et al., 2018) as well as the seismic loading conditions (Rovithis et al., 2009; Yoo et al., 2012). This fact is attributed to not only the inadequate description of the (p-y) curves but also the simulation of the soil medium with uncoupled soil springs.

2.2.5 AASHTO LRFD Guideline

In AASHTO LRFD Seismic Bridge Design Guideline (2011), the foundation modeling method (FFM) is determined according to the seismic design criteria and the site class for different foundation types. For many cases, rigid foundation assumption is deemed adequate in Seismic Bridge Design Guideline, no suggestions regarding simulation of kinematic/inertial interaction is provided. The pile footings with pile caps are modelled with foundation springs with fixed supports at an estimated depth or modelled with soil springs. In other AASHTO documents (non-seismic, 2010, 2017) the kinematic and inertial effects of soil-pile interaction are explained without modeling suggestions and the use of equivalent cantilever beam approach, BWF approach and continuum model approach are provided as options all of which are based on the elastic behavior assumptions. The use of nonlinear

subgrade reaction as proposed by API recommendations is deemed important for piles exposed to high lateral loading and embedded in soft clay/sand.

2.2.6 Turkish Building Earthquake Code (2018)

The new Turkish Earthquake Code (2018) classifies the dynamic analysis procedures for the buildings with pile foundations as direct analysis and sub-structuring approaches and the steps for the analysis are described in details. Considering the direct analysis and the sub-structuring approaches, the Turkish Earthquake Code, presents three distinct methods one of which is selected according to the soil class, seismic design level and the height class of the building. In all options kinematic and inertial interaction is taken into account. Method 1 requires 3D nonlinear continuum analysis, while method 2 requires a similar analysis (involving hysteresis for foundation) using the BWF approach. Method 3 is a simplification of method 2 in which a push-over analyses involving the envelopes of displacements are used for kinematic interaction instead of providing time histories obtained from the free field analysis at each spring level.

All options allow for the separate modeling of the superstructure and the foundation system, conducting kinematic interaction and inertial interaction analysis separately and combining the results. For (p-y) relationships of BWF elements, commonly used curves in Matlock (1970), Reese et al. (1975) and API recommendations are referred. For all of the three sub-structuring methods, the internal forces and the deformations in the piles are calculated by summing the absolute values of results obtained from the kinematic and the inertial interaction according to the Turkish Earthquake Code. In case the internal forces in the superstructure obtained from the analysis with the consideration of inertial interaction are lower than those obtained from the analysis without inertial interaction consideration, the effects of inertial interaction are not to be taken into account in the design of the superstructure.

2.3 Summary

The analysis of soil-pile systems is predominantly carried out using either the Beam on Winkler Foundation (BWF) or continuum-based modeling techniques. Various researchers have investigated the response of the soil-pile systems under dynamic loadings and soil-structure interactions with pile foundations exploring both simplified and detailed approaches. While the beam-on-Winkler-foundation (BWF) method offers a practical framework for such analyses, its limitations in capturing three-dimensional effects and accurately representing the dynamic behavior of soil-pile systems under seismic conditions have been widely recognized. In this study, a continuum-based modeling approach is utilized to examine soil-pile-superstructure interactions.

The response of a superstructure depends on the triple interaction of soil, foundation and the superstructure itself as indicated before. In order to simulate this interaction, the complete system can be modeled by sub-structuring or direct analysis approaches. Both analysis approaches have advantages and disadvantages overcoming to each other. Direct analysis approach provides more detailed, high-fidelity simulations due to straightforward incorporation of nonlinear effects and a more accurate representation of soil-structure interaction.

In this study, a representative, “complete” model consisting of soil, pile foundation and superstructure components is constituted. The important features of this “complete” model of the soil-pile-structure system can be defined and summarized as follows:

- **Constitutive Model for Soil:** Soil is a non-uniform substance exhibiting material and geometric nonlinearity under cyclic/dynamic loading. For this reason, the constitutive model described for the soil layers should be representative for the facts such as change in stiffness and hysteretic response in terms of realistic estimations of structural demands.

- **Seismic Load Application:** The dynamic load definition is another important item for the dynamic analysis of a complete system. The seismic waves propagate through the soil containing the pile system. A special procedure is required to be followed for a proper application of the loads in the analytical model.
- **Initial Condition of the Soil:** Initial stress conditions of the soil is necessary to be defined before performing the dynamic load analysis. Therefore, the complete model should be analyzed under gravity loads prior to conducting the dynamic analysis.
- **Boundary Conditions:** Since a two-stage analysis is conducted, consisting of gravity load and dynamic load analyses, the boundary conditions around the soil is required to be defined accordingly.

CHAPTER 3

MODELING AND ANALYSIS METHODOLOGY

Direct analysis with proper dynamic boundary conditions is the most rigorous and accurate approach to get realistic system demands as shown in Chapter 2. This study aims to develop a comprehensive and highly realistic model that integrates the soil, pile foundation, and superstructure components. These features are represented in an analytical model built using OpenSees - an object-oriented, open-source software framework developed by McKenna and Fenves (2001). OpenSees is a powerful tool for nonlinear analysis in terms of multitude of the element and material definitions existing in its library. This diversity allows for the accurate representation of model components within the OpenSees domain. Furthermore, OpenSees has been widely utilized in academic research, and its library features have been extensively validated. Another advantage of OpenSees is its capability to perform successive analyses, changing boundary conditions. For pre- and post-processing of OpenSees files, MATLAB was used. For pre-processing a MATLAB code creating the analysis files for various dimensions and features of pile and soil automatically was prepared.

The MATLAB code generates all necessary OpenSees files, including those defining the model geometry, boundary conditions, output specifications, and analysis settings. The code allows to generate a three-dimensional rectangular soil medium, with or without piles with the necessary information such that nodes, elements, material model, mass, gravity loads and boundary conditions. According to the soil-pile model information, a separate OpenSees script is generated for the superstructure model, ensuring consistency in node and element numbering with the soil model. Based on the generated model geometry information, the same MATLAB code automatically determines which results will be recorded in the analysis. During a single analysis, a sequential procedure is followed: first, the

gravity load is applied, then the boundary conditions are updated, and finally, the seismic loads are introduced. This process is repeated for each analysis and the transition between the analyses is done automatically within the OpenSees file. Details regarding to the semi-automated analysis process are provided in the following items:

- **Model Formation:** The soil domain consists of a single homogeneous layer, where users can define the dimensions, number of piles, pile embedment depth, and total pile length. The pile cross-section is restricted to circular section, but its diameter can be specified in the code. The soil mesh is composed of brick elements, with adjustable element size. Soil and pile elements with the connection details, mass and vertical loads, soil boundary conditions and viscous damper elements to be required for dynamic analysis are defined in the model files.
- **Constitutive Model for Soil:** The input parameters for the soil material model selected in this study are defined in the MATLAB code and written to the OpenSees model files. These parameters can be updated directly within the OpenSees file. The selected constitutive model consists of both elastic and nonlinear phases. The material phase is updated during different analysis stages.
- **Seismic Load Estimation:** Seismic loads are applied to the system at the side and bottom boundaries of the soil which requires model geometry information. First, 1D soil column analysis is carried out and using the output of this analysis, the seismic loads are computed for each boundary node with a separate MATLAB code. Seismic load estimation will be explained in detail in the following sections.
- **Boundary Conditions:** The analysis is performed by first gravity load and then the seismic load application consecutively. However, the boundary conditions are different in both analysis case. In the gravity load analysis, bottom of the soil medium is fixed in vertical direction while the sides are fixed in horizontal directions. During seismic load application, the viscous dampers at

the sides and bottom are defined. The update in the boundary conditions are carried out sequentially inside the analysis files created by MATLAB.

- **Analysis Scheme:** After 3D finite element model of the soil medium (with piles), superstructure model and the seismic load files are created, then first the gravity analysis is performed with the appropriate boundary conditions. In gravity load stage constitutive model for soil is first defined in its elastic phase, followed by the nonlinear phase, ensuring that the initial stress conditions are properly established. Before proceeding to the seismic load analysis, the boundary restraints are removed, and the reaction forces obtained from the gravity load analysis are applied to ensure model stability. Additionally, viscous dampers are introduced at the side and bottom nodes. During seismic analysis, the material remains in the nonlinear phase.

In the following sections, characteristics of the created analytical model to be created and the analysis methodology are explained.

3.1 Finite Element Discretization of Structure, Piles and Underlying Soil Layers

A three dimensional model of a building structure and underlying foundation with piles is used in this work. A frame-bay of this structure, resting on a three-dimensional foundation domain is analyzed for simplification and reduction of the computational cost. The complete system consists of a frame structure resting on a mat foundation which is supported by a pile-group of 4 piles embedded in a three-dimensional soil strip. A three-dimensional view of the system analyzed is given in the Figure 3.1.

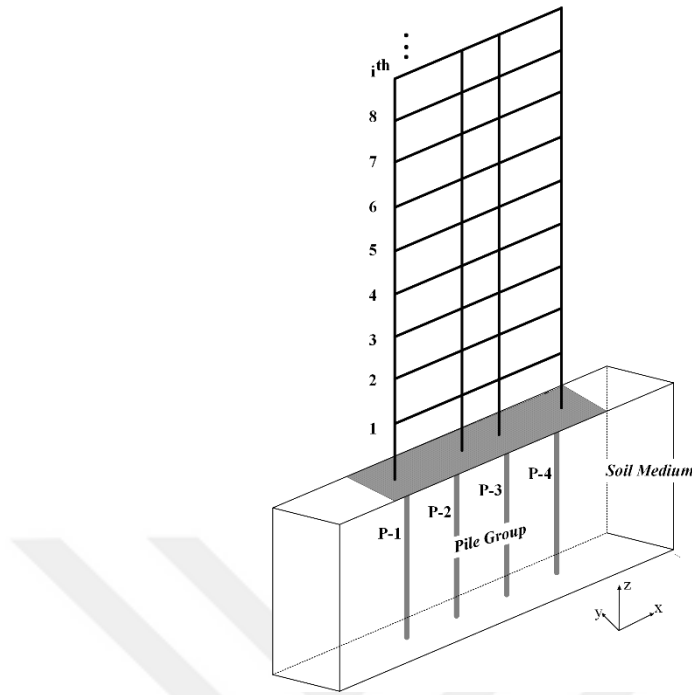


Figure 3.1 3D model of complete soil, pile foundation and structure system

The soil stratum surrounding the piles is modeled as a continuous medium, represented by a single-layer configuration in order to avoid increasing the model complexity with more than one soil layers. It is discretized using eight-node brick elements, with an element height of 2 meters. The dimensions of the elements in other directions are determined based on their specific locations within the model. To enhance the accuracy, a finer mesh is applied in the vicinity of the pile group, while a coarser mesh is used in regions farther from the pile group, considering the overall model dimensions and the duration of the analysis.

The mat foundation corresponding to the soil strip width and supported by the piles is referred to as the "cap beam". This element - equivalent to the tributary length of the system - serves as a representative mat foundation for the building. The piles, cap beam, and structure components are modeled using frame elements. The height of each pile element matches that of the soil elements to ensure compatibility, as the piles and soil elements are interconnected at corresponding heights. A full-contact condition is assumed between the pile elements and the nodes of the soil medium.

The cap beam is discretized based on the connection nodes of the piles and columns. The superstructure is modeled using single frame elements to represent its structural components.

3.2 Soil Model and Properties

As elaborated in the preceding chapter, the soil exhibits a material structure that undergoes variations in strength and stiffness when subjected to cyclic dynamic loads. Consequently, accurately simulating these changes during cyclic loadings is crucial for realistically estimating the demands on the piles and the superstructure.

OpensSees material library involves various types of constitutive models for continuum elements. Among these material models, a model reflecting the effects of cyclic loads but can also be defined with general engineering parameters is more preferable compared to materials characterized with a large number of parameters requiring significant calibration effort. To this end, the material model called “PressureIndependMultiYield” has been selected to represent the behavior of clays overlying the hard soil. The “PressureIndependMultiYield” material exhibits elastic-plastic behavior with plasticity manifesting only in the deviatoric stress-strain relationship (Yang et al., 2005). The volumetric stress-strain response remains linear-elastic and does not depend on the deviatoric response. This material is suitable for modeling the monotonic or cyclic response of substances whose shear characteristics are not influenced by changes in confinement, such as organic soils or clay under quick (undrained) loading scenarios. In the Figure 3.2, shear stress-strain behavior of the material is provided.

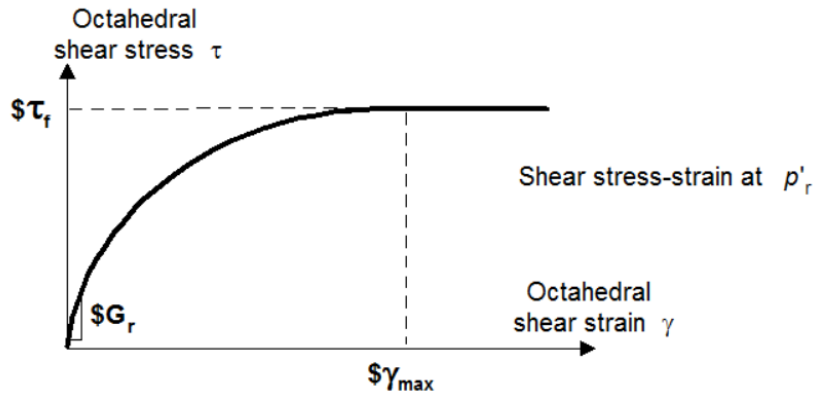


Figure 3.2. Shear stress-strain curve of “PressureIndependentMultiYield” material
(adopted from Yang et al. 2005)

During the application of gravity and any static loads, the material behaves in a linear elastic manner. In the following dynamic (rapid) loading phases, the stress-strain response becomes elastic-plastic. For this reason, the material stage is needed to be updated during the analysis. The reference values for from soft to stiff clay are given in the Figure 3.3. The parameters defined in the first column are the required parameters and explained as follows:

Rho: Saturated soil mass density

Ref. Shear Modulus (G_r): Low-strain shear modulus, specified at a reference mean effective confining pressure

Ref. Bulk Modulus (B_r): Reference bulk modulus, specified at a reference mean effective confining pressure

Cohesion (c): Apparent cohesion at zero effective confinement

Peak Shear Strain (γ_{max}): An octahedral shear strain at which the maximum shear strength is reached, specified at a reference mean effective confining pressure

Friction Angle (Φ): Friction angle at peak shear strength in degrees, optional (default is 0.0)

Ref. Pressure (p'_r): Mean effective confining pressure at which G_r , B_r , and γ_{max} are defined, optional (default is 100. kPa).

Pressure Dependent Coefficient (d): A positive constant defining variations of G and B as a function (Equation 3.1 and 3.2) of instantaneous effective confinement (p') (default is 0.0). If friction angle is zero, then d is reset to 0.0.

$$G = G_r \left(\frac{p'}{p_r} \right)^d \quad (3.1)$$

$$B = B_r \left(\frac{p'}{p_r} \right)^d \quad (3.2)$$

Parameters	Soft Clay	Medium Clay	Stiff Clay
rho	1.3 ton/m ³ or 1.217x10 ⁻⁴ (lbf)(s ²)/in ⁴	1.5 ton/m ³ or 1.404x10 ⁻⁴ (lbf)(s ²)/in ⁴	1.8 ton/m ³ or 1.685x10 ⁻⁴ (lbf)(s ²)/in ⁴
refShearModul	1.3x10 ⁴ kPa or 1.885x10 ³ psi	6.0x10 ⁴ kPa or 8.702x10 ⁴ psi	1.5x10 ⁵ kPa or 2.176x10 ⁴ psi
refBulkModu	6.5x10 ⁴ kPa or 9.427x10 ³ psi	3.0x10 ⁵ kPa or 4.351x10 ⁴ psi	7.5x10 ⁵ kPa or 1.088x10 ⁵ psi
cohesi	18 kPa or 2.611 psi	37 kPa or 5.366 psi	75 kPa or 10.878 psi
peakShearStra (at p _r '=80 kPa or 11.6 psi)	0.1	0.1	0.1
frictionAng	0	0	0
pressDependCoe	0	0	0

Figure 3.3. Reference values for “PressureIndependMultiYield” material parameters (adopted from Yang et al. 2005)

The material parameters of soft clay, as provided in Figure 3.3, are preferred as input parameters to capture the nonlinear soil behavior during dynamic analysis and its effect on soil-structure interaction representing a critical case for pile-supported buildings.

3.3 Modeling of Seismic Loading

Introducing dynamic loads into the system is a crucial aspect of seismic load analysis for a comprehensive soil-foundation-structure system. The process involves calculating earthquake forces, defining appropriate boundary conditions, and following a staged analysis approach, such as performing transient analysis after gravity load application. Seismic activity originates in the soil and propagates toward the foundation and structure, making it essential to accurately estimate the resulting loads and their impact using a structured methodological approach (Wolf, 1985).

The Wave Input Method (WIM), originally introduced by Joyner and Chen (1975), effectively simulates wave propagation and absorbs scattered waves, making the calculations more realistic. (WIM) employs a viscous boundary and a vertically incident SV wave, where boundary forces are determined by both the incoming waves and the viscous boundary conditions. The loading procedure, as generalized in the Lokke and Chopra (2018) was implemented in this study, computing the effective earthquake forces to be applied at the transmitting soil boundaries. According to the procedure, the incident (upward propagating) motion is obtained first, by deconvolution of the surface motion. After finding the incident motions, i.e. history at the bottom of soil strata, the effective earthquake forces at the bottom of the soil is estimated using the following formulation:

$$P_f = 2 \times c_f \times \dot{r}_i \quad (3.3)$$

where $\dot{r}_i$ is the velocity of the incident motion, c_f is the coefficient for the viscous damper defined at the bottom boundary. The free field tractions (forces) are not required for the estimation of the effective earthquake forces at the bottom surface.

In order to compute the side effective forces, a separate free field analysis is needed to be carried out to obtain the side tractions. From the 1D free field analysis of the soil column, the side velocities and forces are recorded to use in the three dimensional finite element model of the soil.

The side effective forces are computed from the equation given below:

$$P_f = R_f + c_f \times \dot{r}_f \quad (3.4)$$

where $\dot{r}_f$ is the velocity history obtained from the 1D free field analysis, c_f is the coefficient for the viscous damper defined at the side boundary and R_f is the free field side tractions.

3.3.1 Boundary Conditions

Boundary conditions of the model is defined in two different ways in the successive gravity and dynamic analyzes. As indicated previously, before dynamic analysis, to take into account the initial stresses in the soil, first, gravity load analysis is performed, then the dynamic analysis is carried out.

In gravity load analysis, at side boundaries, soil nodes are fixed in horizontal directions, but released in vertical direction to represent the settlement of the soil due to the own weight. At the bottom of the soil, vice versa is defined, i.e. bottom nodes are fixed in vertical direction and released in horizontal directions. After gravity load analysis, the model is analyzed under seismic loads.

Seismic analysis of an idealized 3D or 2D continuum models of a soil medium requires definition of transmitting boundaries at the side and bottom surfaces of the soil. Designating viscous damper elements at the side and bottom boundaries to the soil domain is the one approach for establishing the absorbing boundaries. In this study, the transmitting boundaries are preferred to be described by using those kind of elements. Dashpots in horizontal and vertical directions are assigned to the sides and bottom of soil by estimating the appropriate coefficients. At this stage, the supports at the sides are removed and reaction forces obtained in the gravity load analysis are assigned to sustain the equilibrium. However, the supports at the bottom are maintained to prevent potential instability caused by discrepancies in the total vertical forces due to the rounding differences and other factors. In the Figure 3.4,

2D illustration of the boundary conditions around the soil for gravity and dynamic load analysis stage is provided.

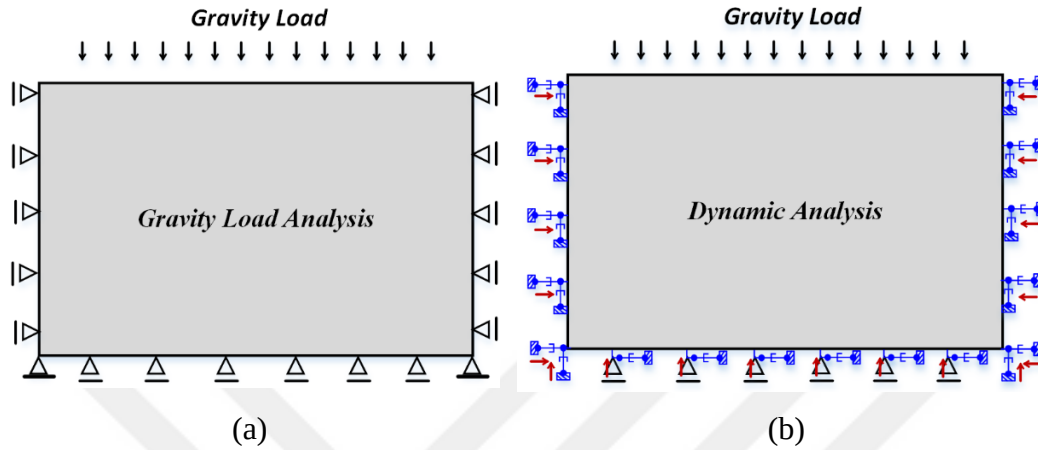


Figure 3.4. Boundary conditions for (a) Gravity load analysis (b) Dynamic load analysis

In this study, Lysmer-Kuhlemeyer (1969) dashpot element is used for defining the absorbing boundaries of the soil medium. The dashpot coefficients are determined as the product of the mass density, shear wave or primary wave velocity (according to the dashpot direction) of the underlying layer and the corresponding area of the soil element.

3.3.2 Verification of Dynamic Load Estimations

The procedure is verified for a 3D soil layer with elastic properties. The free field analysis is carried out on the 1D soil column using OpenSees software and effective earthquake forces are estimated for the three dimensional counterpart model. From each analytical model, surface velocity and displacement histories are acquired. Since the dynamic analyses of the complete model is going to be performed after gravity analysis, the effective earthquake force estimation procedure is applied with the gravity loads consideration.

With this purpose an arbitrary ground motion is selected for verification of seismic load application. The ground motion record (the first motion, record from Manjil Earthquake, details provided in the Table 3.1) is selected for the analyses. In the Figure 3.5, the soil column and the side view of the 3D model of the soil medium are provided. The 1D soil column has been analyzed with the selected ground motion by applying the velocity time history at the point where the dashpot element is assigned. From the free field analysis of the soil column, velocity time histories and side forces are obtained to estimate the effective earthquake forces for the three dimensional model of the soil medium. Using these forces, the dynamic analysis of the 3D soil model is conducted.

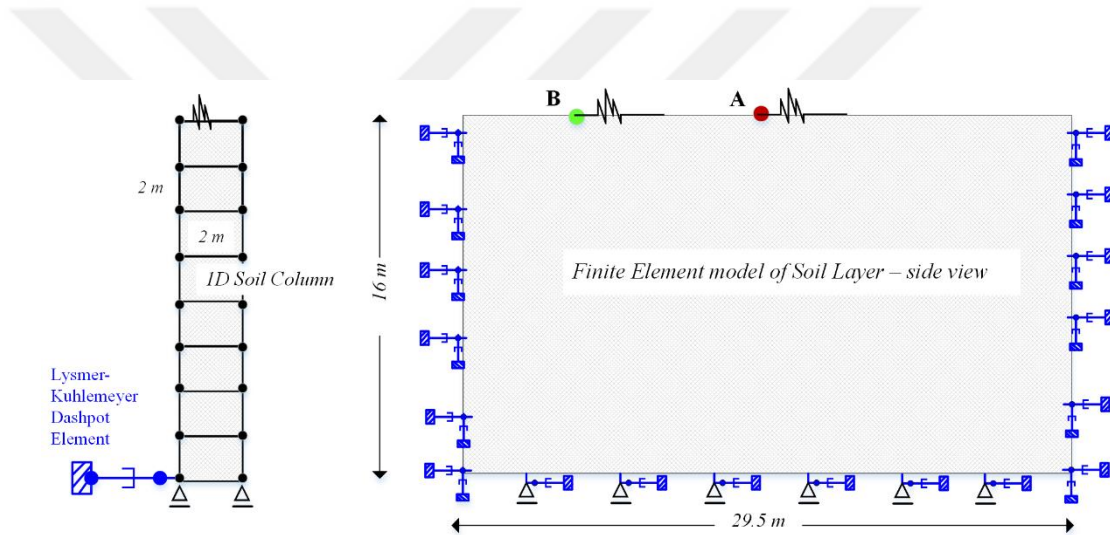


Figure 3.5. 1D soil column and 2D demonstration of soil medium with the control points

Velocity and displacement time histories are obtained at the surface on two different points (point A and point B). Figures 3.6 and 3.7 illustrate how the velocity and displacement time histories obtained from 3D analysis match with the 1D soil column analysis results, demonstrating the accurate estimation of the effective earthquake forces for a 3D soil finite element model. The results at point A and B are very close to each other and match well with soil column analysis.

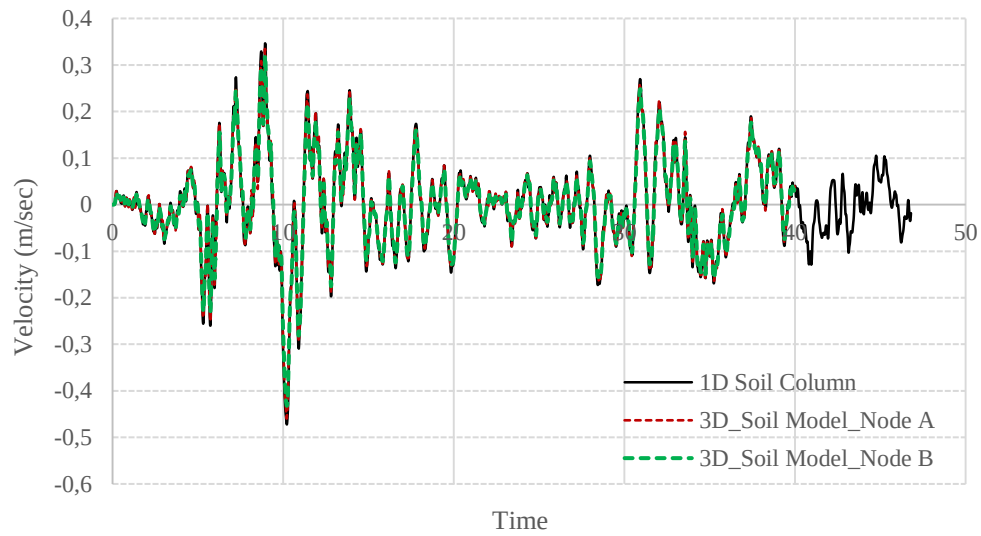


Figure 3.6. Velocity history at the surface of clay layer, 1D soil column vs. 3D finite element model

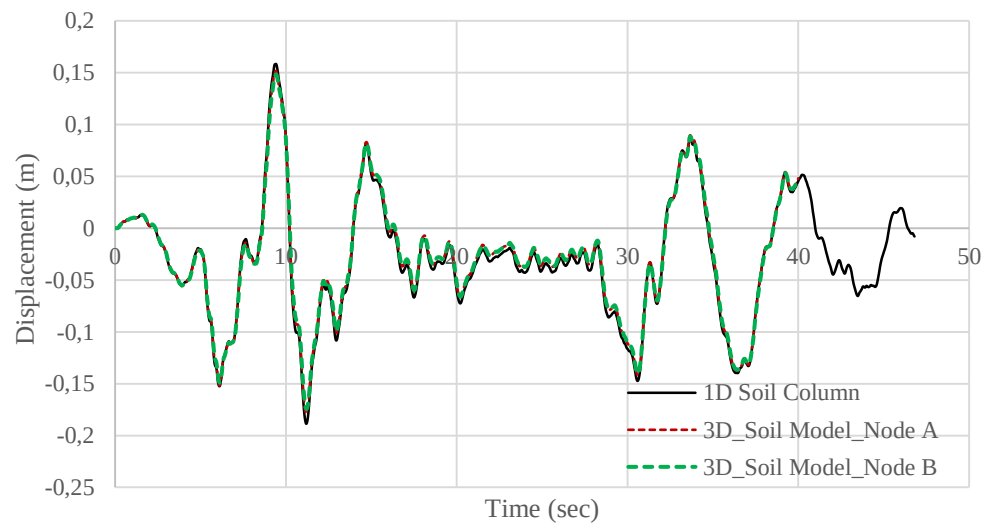


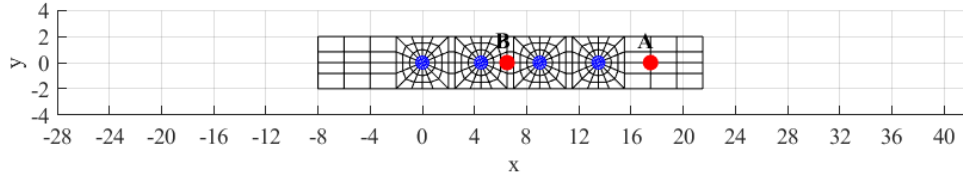
Figure 3.7. Displacement history at the surface of clay layer in the models, 1D soil column vs. 3D finite element model

3.3.3 Determination of the Soil Model Size

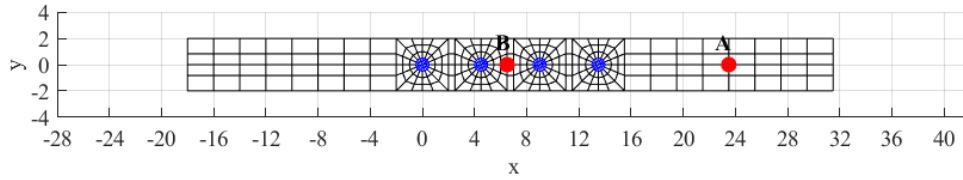
In a semi-unbounded domain, effectively simulating wave propagation requires not only the implementation of absorbing boundary conditions but also careful consideration of the model size. While absorbing boundaries, such as Perfectly Matched Layers (PMLs) or dashpot boundaries, are designed to reduce artificial wave reflections, their effectiveness is influenced by the proximity of these boundaries to the region of interest. If the soil domain is too small, seismic waves may reach the absorbing boundaries prematurely, potentially causing partial reflections or scattering due to insufficient energy dissipation within the boundary. This can distort the wave propagation in the area of interest and reduce the accuracy of the simulation. Conversely, a larger domain allows the waves to propagate more naturally before interacting with the boundaries, reducing the likelihood of such artifacts. However, increasing the domain size significantly also increases computational costs, as more elements or nodes are required to model the soil, leading to longer analysis times. Therefore, an optimal domain size must strike a balance: it should be large enough to allow realistic wave propagation and effective boundary absorption while remaining computationally efficient.

To determine this optimal size, several soil models with varying domain dimensions are compared, evaluating the extent to which boundary reflections are minimized and wave behavior is accurately captured. The dimensions of the soil domain has been selected considering the findings of this comparison.

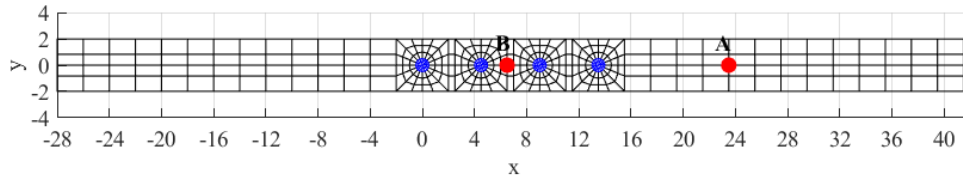
For the comparison, three different soil model sizes have been used based on the length of the strata in the loading direction, specifically 29.5 m, 49.5 m, and 69.5 m. The models consists of 3258, 3708 and 4158 nodes, respectively. The top view of the soil models are shown in Figure 3.8. Using an arbitrary ground motion, the acceleration response spectrum curves on the soil surface have been compared with the response spectrum curve obtained from the 1D soil column analysis. By this way, both soil model size to be selected is evaluated and the earthquake force estimation procedure is validated one more time.



(a) Small size soil model – Top view



(b) Medium size soil model – Top view



(c) Large size soil model – Top view

Figure 3.8. Compared soil models

The ground motion recorded on “Gebze” station during Kocaeli Earthquake (1999, Turkey) on rock type site class ($V_{s30}=792$ m/s) is used for the analyses of the finite element models (*file name: RSN1161_KOCAELI_GBZ000.AT2*). The acceleration time history of the ground motion is given in the Figure 3.9.

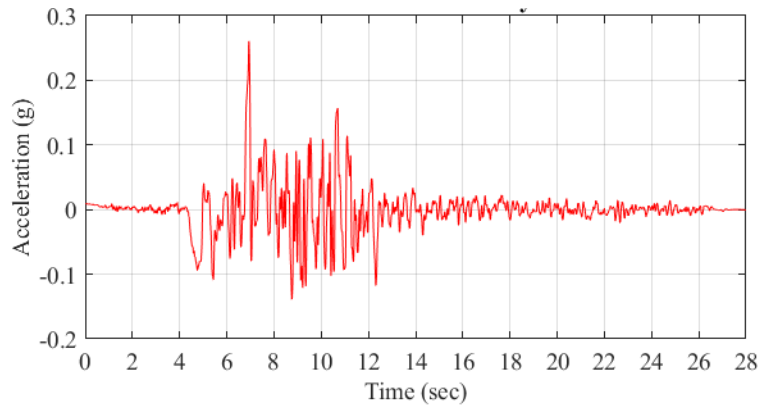
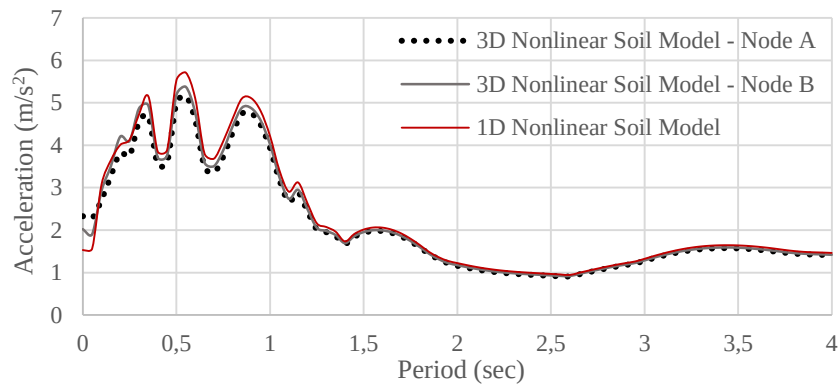


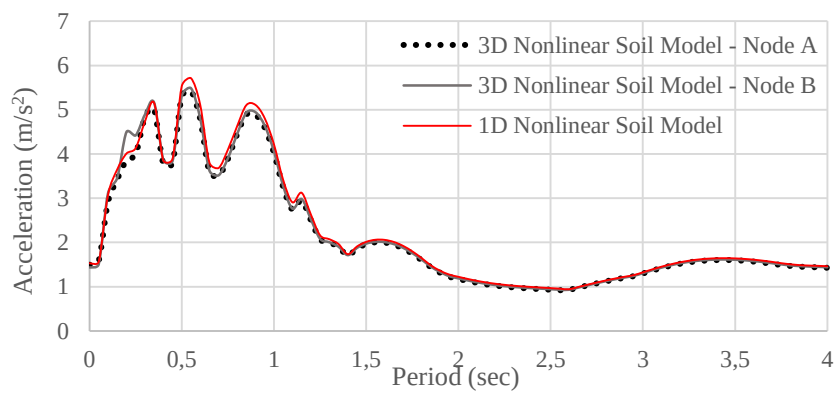
Figure 3.9. Acceleration time history of the selected arbitrary ground motion

The acceleration response at the surface level for the selected earthquake record was first obtained for the 1D soil column model, with a height of 16 m. The pressure independent constitutive model (PressureIndependentMultiYield) was used in accordance with the Figure 3.3, soft clays. The response spectrum for the surface motion from the 1D model is treated as the benchmark result which is then compared to the results from the 3D models. After the time history analysis of the finite element models, response spectra for the acceleration histories were obtained at two control points: the midpoint of the soil domain and the interface between the far-end pile and the boundary surface and compared with the benchmark spectrum derived from the 1D soil column analysis.

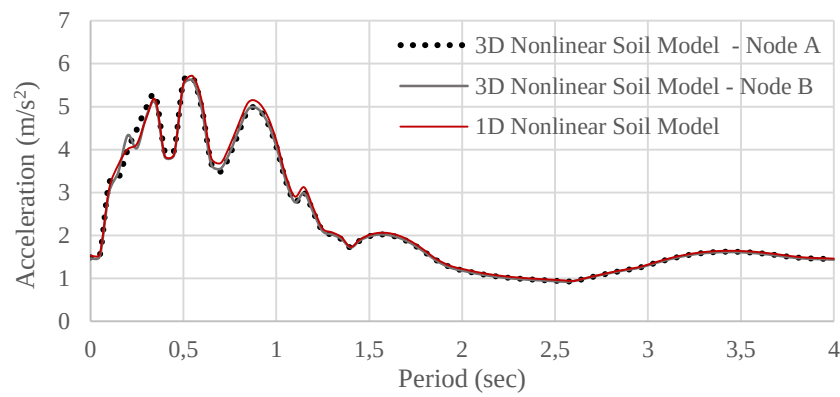
The comparison of the response spectra from 1D analysis and 3D models are shown in Figure 3.10, for models with increasing sizes. For larger domain sizes, the response spectra gets similar to the one obtained from the soil column analysis. In addition, in all figures the response spectra at the midpoint of the soil surface is better matched with the soil column analysis result compared to the spectrum close to the soil boundary as expected.



(a) Small domain (~30m wide)



(b) Medium domain (~50m wide)



(c) Large domain (~70m wide)

Figure 3.10. Acceleration spectrum, comparison of 1D soil model with 3D soil models

The figures indicate that as the soil domain size increases, the response spectra converges toward the curve obtained from the 1D soil column analysis as anticipated. In all models, the response spectrum aligns well with the 1D soil column analysis results for periods exceeding 1 second. While the largest soil model demonstrates the closest agreement with the 1D analysis, the smaller soil model is deemed sufficiently accurate, particularly when considering the increased computational effort required for larger models. This approach highlights the interplay between boundary effectiveness and domain size, emphasizing that even with advanced boundary techniques, the size of the model remains a critical factor for reliable seismic analysis.

3.3.4 Determination of the Soil Model Element Height

The element size in the finite element modeling of soil media is a critical parameter for accurately simulating wave propagation. Proper selection of the element size ensures the fidelity of the numerical model in capturing the dynamic behavior of the soil under seismic or vibrational loading. Kuhlemeyer and Lysmer (1973) established fundamental criteria for selecting an appropriate element size to minimize numerical dispersion and ensure accuracy in finite element analysis. According to their findings, the relationship between the element size and the wavelength of the wave is of primary importance. Specifically, the maximum element size should not exceed one-eighth to one-tenth of the wavelength (λ) associated with the highest frequency wave component of interest. This criterion provides that the finite element mesh is capable of accurately representing the waveforms, avoiding potential distortions or numerical inaccuracies due to under-resolution. The wavelength is determined by the relationship between the shear wave velocity (V_s) and the maximum frequency (f) of the wave, expressed as:

$$\lambda = \frac{V_s}{f} \quad (3.5)$$

To ensure accurate wave propagation modeling, the maximum element size (Δx) should satisfy the following relationship:

$$\Delta x \leq \frac{\lambda}{8} \quad (3.6)$$

Non-compliance with these criteria can result in pronounced numerical dispersion, characterized by inaccuracies in the representation of wave speeds and amplitudes, thereby compromising the reliability of the simulation outcomes. Additionally, finer discretization, while improving accuracy, increases computational cost. Consequently, the determination of the element sizes in the finite element model necessitates a careful balance between achieving precision and maintaining computational efficiency.

A model with 16m height and 2m element size, was used to assess the smallest wave length of the soil medium. Time history analysis with white noise input was used to accurately determine the frequency response of the system. 1D soil column model with elastic and nonlinear soil properties are shaken with white noise ground motion to test the system in a wide frequency range. The theoretical observable maximum frequency for this model with $H=16\text{m}$ and 2m linear element size with elastic soil of $E=40000\text{ kPa}$, is 6.55 Hz from the equations 3.5 and 3.6.

The white noise acceleration data with unit 1g maximum acceleration is, generated with normal distribution and provided in the Figure 3.11. To investigate the soil response under various excitation levels, the generated white noise record has been scaled to produce maximum acceleration amplitudes of 0.005g, 0.01g, 0.05g, 0.1g, 0.5g, and 1g. These scaled records have been then used to analyze the soil column's behavior under different loading conditions.

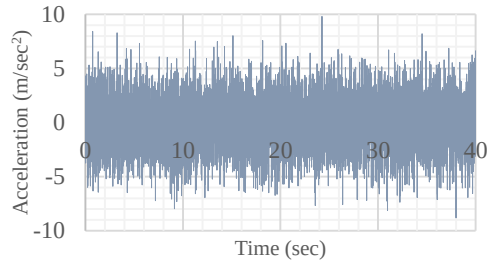
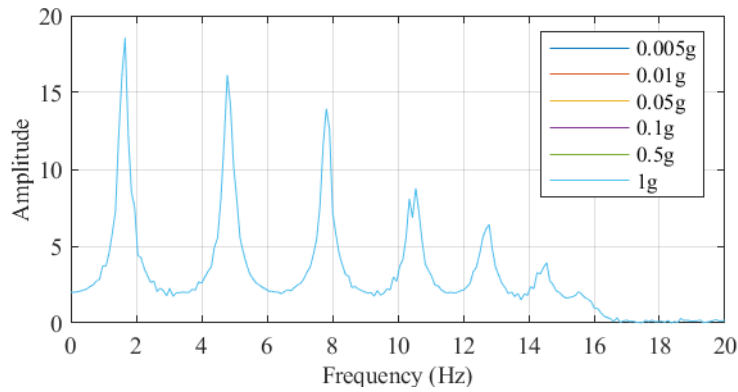
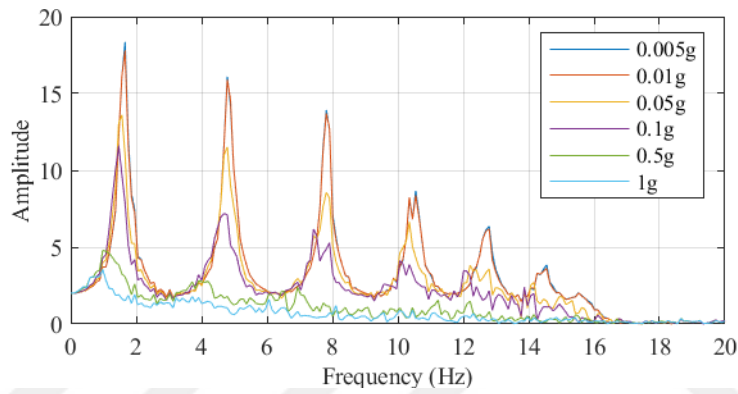


Figure 3.11. Acceleration (a) time history of the generated arbitrary white noise

The transfer function between the input velocity and the output velocity, was calculated by dividing Fast Fourier Transform (FFT) of output with the input velocity for each scaled white noise input. In Figure 3.12, the transfer functions are plotted for elastic and nonlinear soil models, respectively, at different shaking levels. At certain frequencies, the transfer functions exhibit significantly larger amplitudes, corresponding to the natural periods (or natural frequencies) of the system. These natural periods are influenced by the properties of the soil, such as stiffness and mass density. For the elastic soil model shown in Figure 3.12a, the natural periods remain constant regardless of the excitation amplitude. In contrast, for the nonlinear model, increased excitation amplitude typically reduces stiffness, shifting the natural periods to lower values.



(a) Elastic model, 1D soil column



(b) Nonlinear model, 1D soil column

Figure 3.12. Transfer function for nonlinear model of 1D soil column

The amplitude of the transfer function drops to nearly zero after a certain frequency, implying that the system does not respond significantly to input signals at higher frequencies. The high frequencies correspond to the small wavelength motions and may not effectively propagate through thick soil layers.

Evaluation of the transfer function indicates that the maximum frequency resolvable by the soil model is 16 Hz. However, engineering applications mostly focus on lower frequency ranges, as higher frequencies often have less impact on overall structural response. Considering the computational demands of the model, the significant nonlinearity of the soft soil and the preceding results with good frequency content until 16 Hz, an element size of 2 meters was determined to adequately represent the spatial resolution while maintaining computational efficiency.

As a second criterion for the element size stems from the nature of finite element analysis. In finite element analysis, the time step must be small enough to ensure that a wave front progresses sequentially through the nodes. If the time step is too large, the wave front could reach multiple nodes simultaneously, violating wave propagation principles and causing instability in the solution. Therefore, there is a relation between the element size (Δx) and time step (Δt) such that:

$$\Delta t \leq \frac{\Delta x}{c} \quad (3.7)$$

$$c = \sqrt{\frac{E}{\rho}} \quad (3.8)$$

where c is the wave velocity, E is the Modulus of Elasticity and ρ is the unit mass. For $E=40000$ kPa and $\rho=1.3$ ton/m³ along with the 2m element size, the maximum time step to be used in the dynamic analysis is estimated as 0.011 seconds. In the dynamic analysis 0.0025 time step is used which satisfies the maximum Δt criterion.

3.4 Ground Motion Selection

In time history analyses, selected ground motion set has an important role in the analysis since the demands will vary according to the characteristics of the ground motion. Earthquake records have been selected according to the following criteria from PEER Ground Motion database and arithmetically scaled in the acceptable scaling range in this study.

- **Average shear wave velocity at the upper 30 m (V_{s30}):**

For 1D soil column analysis with non-linear properties, ground motions chosen from rock were deconvolved. The ground motions recorded on rock are chosen in this study for this analyses and one-half of the motions is used in convolution through non-linear soil. By this way, deconvolution process and the issues from

these calculations are avoided. Motions were hence chosen from PEER database from sites with 540 – 800 m/s average shear wave velocity at the upper 30 m (V_{s30}).

- **Moment magnitude (M_w):**

The larger magnitude events were selected as representative of risk for which these structures are designed for. The moment magnitude of chosen motions are between 6.0 and 8.0.

- **Projection of the closest distance (Joyner-Boore Distance – R_{jb}):**

The shortest distance from the site where the ground motion is recorded to the surface projection of the rupture plane has been selected to be between 10 and 40 km. This range is chosen to eliminate the pulse effect of the motions while preserving the intensity of the event.

Considering the above criteria, close to 100 ground motions have been found in the PEER Database. A set of 24 records have been constituted by selecting the motions recorded as much as possible from different earthquake events (17 out of 24 records are distinct events). In addition, diversity in terms of earthquake formation mechanism has been taken into account while comprising the motion set. The selected ground motions are summarized in Table 3.1 along with their important features. The first components of the motion sets were used in the dynamic analysis. The response spectra of the selected motions are given in Figure 3.13. The acceleration response spectrum obtained from the clay surface is provided in the next chapter in Figure 4.13.

Table 3.1 Selected ground motions for dynamic analyses

	Event	Year	Magnitude	Mechanism	Rjb (km)	Rrup (km)	Vs30 (m/sec)
1	Manjil_ Iran	1990	7.37	strike slip	12.55	12.55	723.95
2	Loma Prieta	1989	6.93	Reverse Oblique	12.04	18.41	713.59
3	Northridge-01	1994	6.69	Reverse	12.39	18.36	545.66
4	Chi-Chi_ Taiwan	1999	7.62	Reverse Oblique	26	26	704.64
5	San Fernando	1971	6.61	Reverse	13.99	19.3	602.1
6	Hector Mine	1999	7.13	strike slip	10.35	11.66	726
7	New Zealand-02	1987	6.6	Normal	16.09	16.09	551.3
8	Kozani_ Greece-01	1995	6.4	Normal	14.13	19.54	649.67
9	Irpinia_ Italy-01	1980	6.9	Normal	22.54	22.56	561.04
10	Chalfant Valley-02	1986	6.19	strike slip	14.97	18.31	585.12
11	Coalinga-01	1983	6.36	Reverse	25.98	27.46	648.09
12	Duzce_ Turkey	1999	7.14	strike slip	25.78	25.88	782
13	Sitka_ Alaska	1972	7.68	strike slip	34.61	34.61	649.67
14	Chi-Chi_ Taiwan-03	1999	6.2	Reverse	13.04	14.66	614.98
15	Chi-Chi_ Taiwan-04	1999	6.2	strike slip	17.63	17.7	542.61
16	Chi-Chi_ Taiwan-05	1999	6.2	Reverse	31.33	32.27	573.04
17	Loma Prieta	1989	6.93	Reverse Oblique	17.92	18.33	663.31
18	San Fernando	1971	6.61	Reverse	30.7	30.7	667.13
19	Morgan Hill	1984	6.19	strike slip	14.83	14.84	729.65
20	N. Palm Springs	1986	6.06	Reverse Oblique	16.55	17.03	659.09
21	Griva_ Greece	1990	6.1	Normal	32.84	33.29	551.3
22	Big Bear-01	1992	6.46	strike slip	34.43	35.41	659.09
23	Northridge-01	1994	6.69	Reverse	20.77	21.36	602.1
24	Chi-Chi_ Taiwan	1999	7.62	Reverse Oblique	10.96	10.96	544.74

The duration of the selected ground motions is adjusted based on the Arias Intensity of the entire seismic event for faster time history analysis. Specifically, the ground motion recordings are truncated at the point where 99.5% of the total Arias Intensity has been reached, resulting in a shortened version of the record. The acceleration, velocity and displacement time histories for each motion are provided in the Appendix.

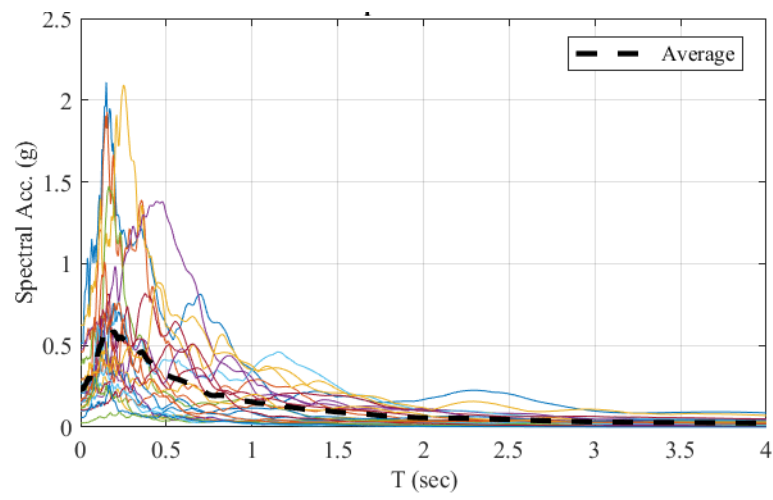


Figure 3.13. Acceleration response spectrum of the selected ground motions

CHAPTER 4

SEISMIC ANALYSIS OF COMPLETE MODEL

In this study, the important features of the soil-pile-structure interaction response are included in the analytical model for the dynamic analyses to obtain a realistic representation of structure and pile response in this complex setting. The important features regarding to the analytical model of soil-pile and structure system have been expressed as the constitutive model of soil, representation of soil-pile interface behavior, seismic load application, stage analysis process and boundary conditions.

In the following section each part of the complete system is introduced regarding to the model geometry and material properties.

4.1 Model Properties

4.1.1 Structural System Modeling

Two mid-rise buildings with different numbers of floors are analyzed. To represent typical mid-rise structures, 2D frame models with 5 and 13 stories are used for the superstructure. These models have been developed based on the 12-story frame structure from the study by Alici and Sucuoğlu (2015), with modifications made to the number of floors and section dimensions. The original building design in that study adheres to TS 500 (2000) and TEC (2007) using capacity design principles. Typical floor plan of the building is shown in Figure 4.1a.

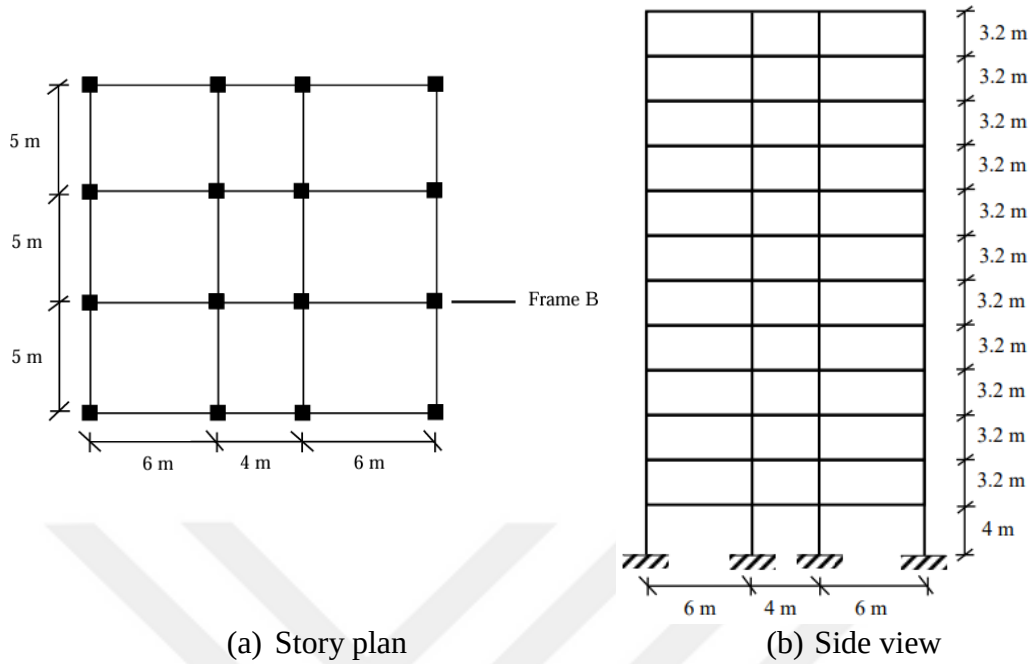


Figure 4.1. 2D Model of 12-story symmetric building (Adopted from Alıcı and Sucuoğlu (2015))

In this 3D building plan, frame B is modeled as the 2D frame model supported by the pile foundation and the continuum model of the soil strata. The frame system is modeled to exhibit two-dimensional (2D) behavior within a three-dimensional (3D) environment by constraining the out-of-plane degrees of freedom, specifically translation in the y-direction and rotations about the x- and z-axes. To achieve this, the soil and foundation components of the system are modeled in 3D, while the frame itself is restricted to 2D behavior. In OpenSees, the spatial dimension and the number of degrees of freedom must be defined at the outset, and the model is configured accordingly. Consequently, it is not possible to combine 3D and 2D models within a single simulation framework. To address this limitation, the frame model is defined in 3D but with its out-of-plane degrees of freedom constrained to ensure realistic behavior. This approach ensures that the frame behaves as intended within the 3D environment while maintaining computational consistency. In the Figure 4.2, the frame models with the soil medium and the pile foundation are provided.

elements differ between the support and span regions to account for the tension and compression forces. For example, in a 30x55 cm section, there are 4 Φ 22 reinforcement bars at the top of the section in the support region, while in the span region, there are 4 Φ 22 reinforcement bars at the bottom of the section. However, in the present study, a single cross-section configuration is preferred for each element, without changing between support and span regions.

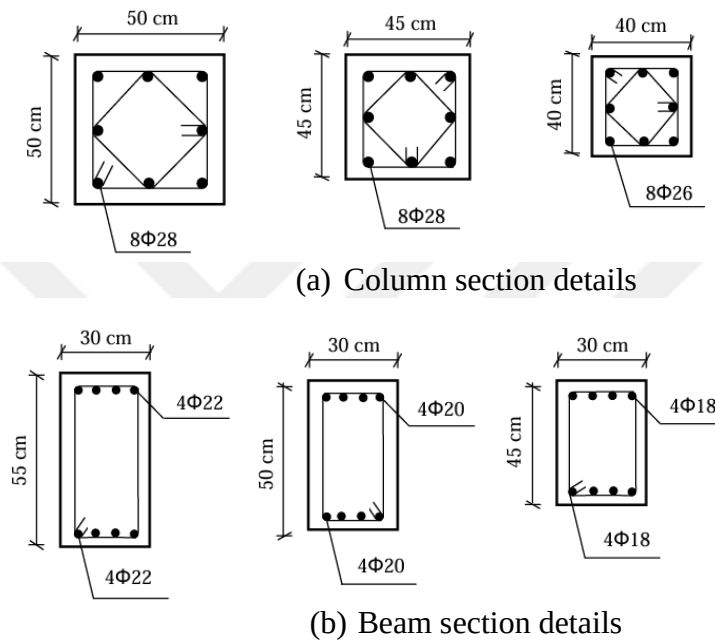


Figure 4.3. (a) Column and (b) Beam Section details (Adapted from study of Alıcı and Sucuoğlu (2015))

The dimensions of the structural sections are adjusted based on the number of floors in the building. Material properties are identical to Alıcı and Sucuoğlu (2015). The beam and column dimensions for the 5- and 13-story buildings are modified as follows:

- The 5-story building has beam cross-sectional dimensions of 30 x 45 cm and column cross-sectional dimensions of 40 x 40 cm. The cap beam height is set at 60 cm, determined based on the number of stories. The slab thickness, live load, and material properties are identical to those of the reference building.

- The 13-story building has been developed by adding an additional floor to the top of the 12-story structure described in the referenced study. The beam and column cross-sections of the 13th story are identical to those of the preceding story, with beams measuring 30x45 cm and columns 40x40 cm.

Each of the 2D frame structures in different number of stories (5-story and 13-story models) are modeled in two different ways, including elastic and nonlinear properties. In elastic model of the superstructure, beams, columns and cap beam elements are modelled with the line elements with elastic properties. In order to define cracked section stiffness, the gross moment of inertia values of beams and columns are multiplied by 0.4 and 0.6, respectively. Elastic material properties are provided in the Table 4.1. Young's modulus and shear modulus are necessary for elastic frame element definitions.

Table 4.1 Material properties for beam, column and cap beam elements of the elastic model of superstructure

Youngs Modulus	(kPa)	25000000.00
Shear Modulus	(kPa)	10416666.67
Poissons Ratio	-	0.2
Unit Weight	(kN/m ³)	25.0

For the nonlinear model of the superstructure, beam and column elements are modeled using the "Beam with Hinges" element available in the OpenSees library. In the "Beam with Hinges" element, plastic hinge regions are defined at the start and end locations of the element. The plastic hinge length (L_p) for beams and columns have been taken as the half of the section dimension (h) in the active direction, which is the height of the sections for beams ($L_p = 0.5 h$). For the plastic hinge regions, bilinear moment-curvature behavior is defined for beam elements. This behavior is then converted into a moment-rotation ($M-\theta$) representation using the plastic hinge length, enabling the plastic hinge to capture inelastic deformations through rotational

response. The moment-curvature relations obtained using Response2000 software are compared to idealized curves in Figure 4.4.

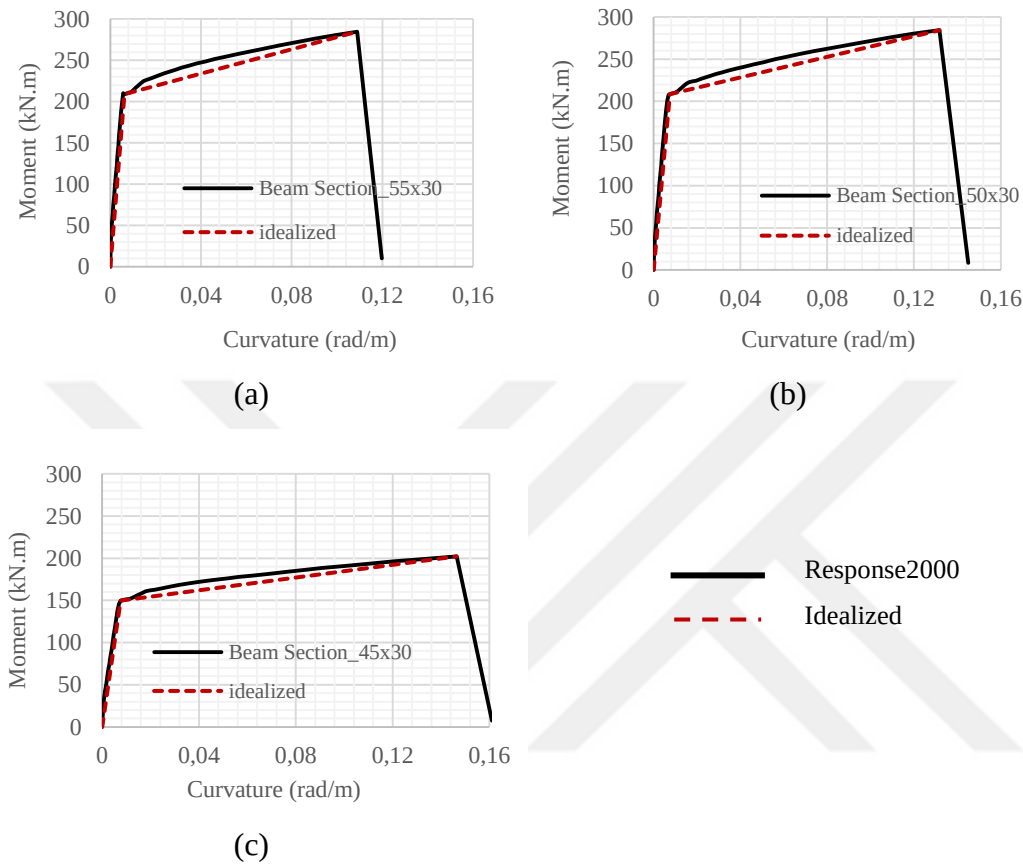


Figure 4.4. Moment curvature diagrams for beam sections (a) 55x30, (b) 50x30 and (c) 45x30

Plastic hinge regions for column elements are characterized as fiber sections capturing material nonlinearities through a fiber discretization approach. The cross-section consists of confined core concrete, unconfined cover concrete, and longitudinal reinforcement fibers. The Kent and Park (1971) model is used to define the confined concrete behavior, considering the effects of lateral confinement provided by transverse reinforcement, while the unconfined concrete follows the same model without confinement enhancement. Both concrete types are modeled using “Concrete01” existing in OpenSess library, while “Steel01” is used for the reinforcement rebar. The section is split as confined and unconfined concrete

regions. The core concrete part is discretized into 10×4 fibers in the excitation and non-excitation directions, respectively. This fiber-based approach allows for detailed evaluation of the moment-curvature relationship and nonlinear behavior under seismic loading conditions. Section analysis results for columns are illustrated in Figure 4.5. Outside the plastic hinge zones, structural components are represented by cracked section properties, consistent with the assumptions of the elastic superstructure model.

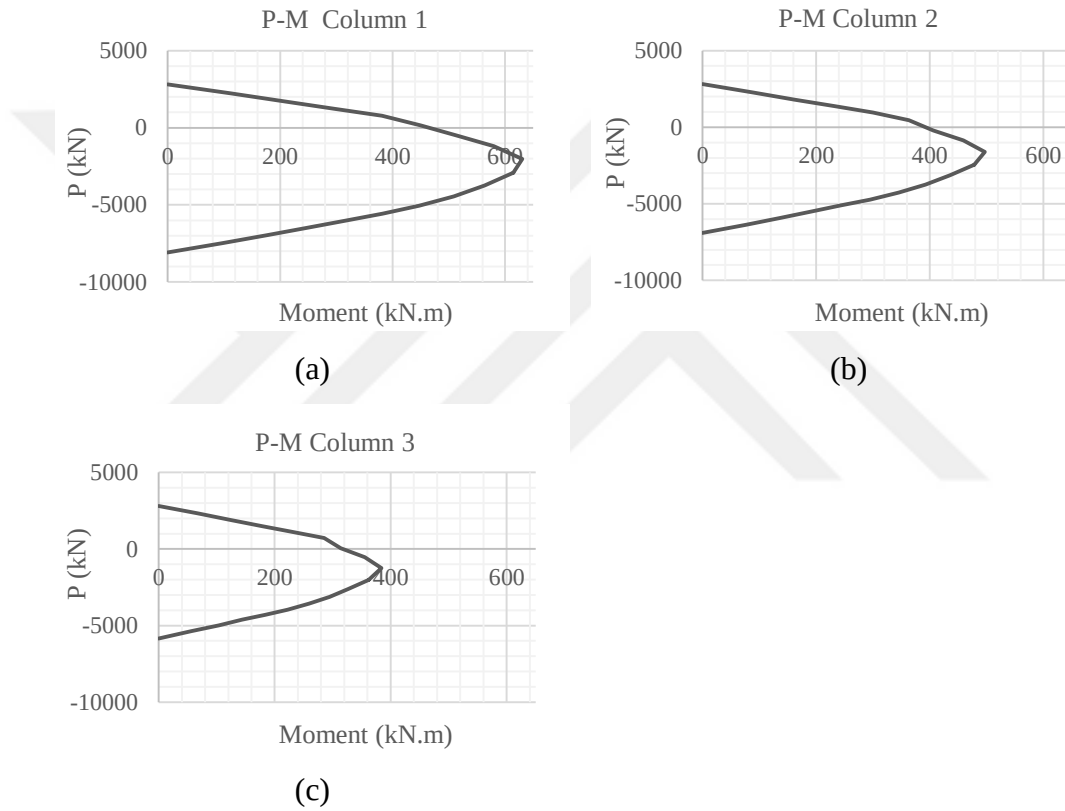


Figure 4.5. P-M interaction diagrams for column sections (a) 50x50 cm (b) 45x45 and (c) 40 x40

4.1.2 Soil System Modeling

The dimensions of the soil stratum is determined as 16 m in height, 4 m in width and 29.5 m in length. The mesh of the soil medium is constituted from eight node brick elements. Element heights are the same throughout the entire soil mesh and are

determined as 2m. The length and width of brick elements vary throughout soil medium. The top view of the soil mesh is provided in the Figure 4.6.

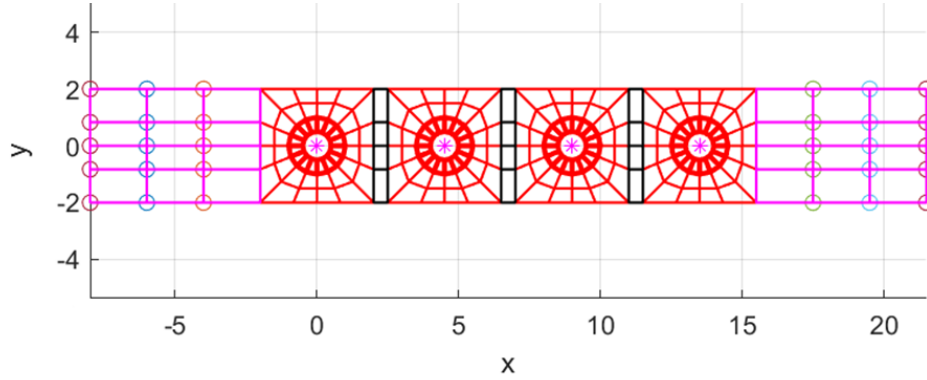


Figure 4.6. Top view of soil mesh

Soft clay properties of the “PressureIndependMultiYield” material model are used as the constitutive model for soil. Material properties for both soil layer and the underlying rigid rock layer are provided in Table 4.2 and Table 4.3.

Table 4.2 Material Properties for Clay Layer

Soil Medium		
Unit Mass	ton/m ³	1.3
Young's Modulus	kN/m ²	40000
Shear Modulus	kN/m ²	14285.71
Bulk Modulus	kN/m ²	66666.67
Poisson's ratio	-	0.4
Peak Shear Strain	-	0.1
Cohesion	kN/m ²	20
Shear Wave Velocity	m/s	104.83
Primary Wave Velocity	m/s	256.78

Table 4.3 Material Properties for Base Rock Surface

Rock		
Unit Mass	ton/m ³	2.5
Young's Modulus	kN/m ²	3800000
Shear Modulus	kN/m ²	1357143
Poisson's ratio,	-	0.25
Shear Wave Velocity	m/s	736.79
Primary Wave Velocity	m/s	1804.76

Boundary conditions around the soil medium has been defined according to the analysis type (gravity or dynamic analysis) as mentioned in the previous Chapter 3. Damper elements have been defined at the left and right sides of the soil mesh in all translational directions. However, since seismic loads have been purposed to be applied in only the horizontal (x) direction, the damper elements assigned in y direction will be ineffective. Similarly, at the bottom of the soil, damper elements have been defined in all translational directions. The front and back surfaces of the soil have been fixed in y-direction, no damper elements have been assigned on these surfaces. For dashpot coefficient estimation, at the bottom of the soil, the parameters of the underlying rock layer is used while those for the clay layer are used at the sides of soil. Similarly, damper elements are defined at the base of the piles where they rest on the rock layer.

4.1.3 Pile Foundation Modeling

The foundation model consists of a pile group comprised of 4 cast-in-place concrete piles supporting a mat foundation. The foundation component corresponding to the soil strip width and supported by the piles is referred to as the "cap beam." This element, with a width of 4 meters—equivalent to the tributary length of the system—serves as a representative mat foundation for the building. In other words, the cap beam functions as the mat foundation within the modelled soil strip. The cap beam

elements are defined at mid-height of the mat foundation, with rigid elements defined between the centerline of the cap beam and the pile tops, as well as between the cap beam and the column bases, to accurately represent the geometry of the system. The cap beam thickness was assumed 0.6 m and 1.5 m for the 5 story and 13 story buildings, respectively. The cap beam was modeled with cracked section modulus as typically expected for these structures. Cap beam nonlinearity is not considered in the analysis.

Piles are embedded into the soil layer completely and are assumed to be seated on the rigid/competent base boundary at the bottom of the model. Piles have been modeled with the beam elements through the center line with the section properties. In order to preserve the cross-section geometry, rigid elements have been defined from the end nodes of the beam elements at the center of the pile section to the soil surface. Circular section is used for the pile sections. The diameter of the circular pile section is determined as 1m and center to center distance between the piles are specified as 4.5m corresponding to 4.5 times of the diameter in order to limit the group action.

Piles are modeled with elastic material properties using cracked section properties with 25.0 GPa Young's Modulus, 0.2 Poisson's ratio and a unit weight of 23.6 kN/m³. Elastic material properties for concrete section is given in the Table 4.4. Concrete and steel material strengths are assumed as 25 MPa and 420 MPa, respectively, for the capacity calculations. The cross-section of the pile with 16 ϕ 32 S420 reinforcement and 4cm clear cover is provided in Figure 4.7. The axial load-moment interaction diagram for the pile section is given in the Figure 4.8.

Table 4.4 Material Properties for Pile

Young's Modulus	(kPa)	25000000.00
Shear Modulus	(kPa)	10416666.67
Poisson's Ratio	-	0.2
Unit Weight	(kN/m ³)	23.6

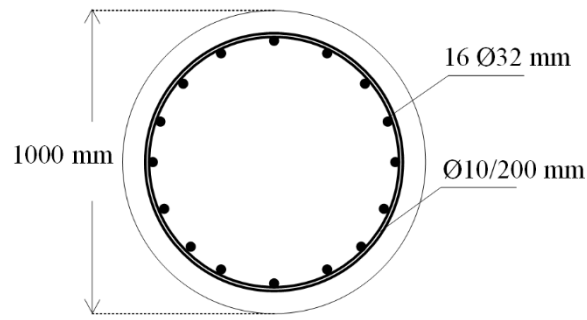


Figure 4.7. Preliminary design for pile section

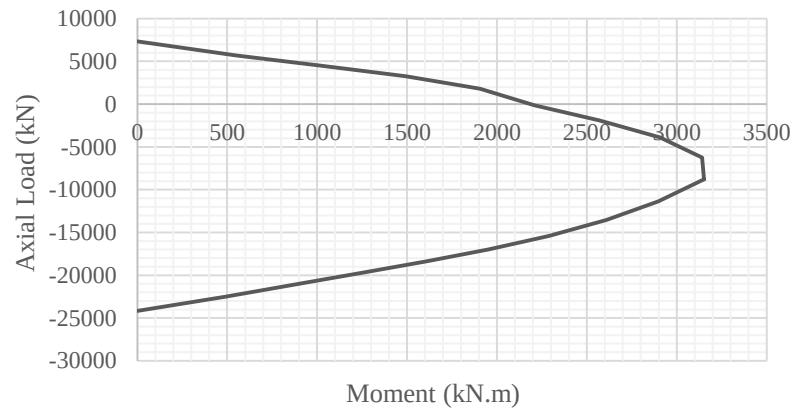


Figure 4.8. P-M interaction diagram for pile section

Piles are connected by a cap beam with dimensions of 4.0 m in width and 20 m in length. The cap beam height is specified as 1.5 m for the 13-story building model and 0.6 m for the 5-story model. Cracked section stiffness for the piles and cap beam is defined by multiplying the gross moment of inertia values by 0.6 and 0.4, respectively.

In the next section, the mass and Eigen modes of the modeled structures are presented for the fixed base systems.

4.2 Structure Only, Gravity and Modal Analysis

Before seismic analysis of the complete system, gravity analysis with dead and reduced live loads was carried out in order to obtain the dynamic properties of the superstructure and the whole model.

Modal analysis and gravity load analysis of the fixed base model of the superstructure have been performed before the dynamic analysis. Fixed support was provided at the top of the piles. Consistent mass approach was used in the analysis. Total structure mass with reduced live load (used in dynamic analysis) and the total weight of the building with full of live load (used in gravity load analysis) is given in the Table 4.5.

Table 4.5 Total mass and weight of the buildings

Building	Mass (ton)	Weight (kN)
Five-Story Structure	336	3582
Thirteen-Story Structure	884	9421

Natural periods of the 5-story frame model with cracked section properties are provided in Table 4.6 while the mode shapes for the first four modes are shown in Figure 4.9. The table also includes the natural periods of the 5-story frame model incorporating the soil-foundation system. The Eigen analysis of the complete soil-pile-structure system is performed with soil boundaries restrained in all directions.

Table 4.6 Natural periods, 5-story frame and 5-story frame with soil domain

Mode	Only Structure	Structure with Soil Domain
1	1.317	1.337
2	0.408	0.411
3	0.220	0.223
4	0.141	0.144

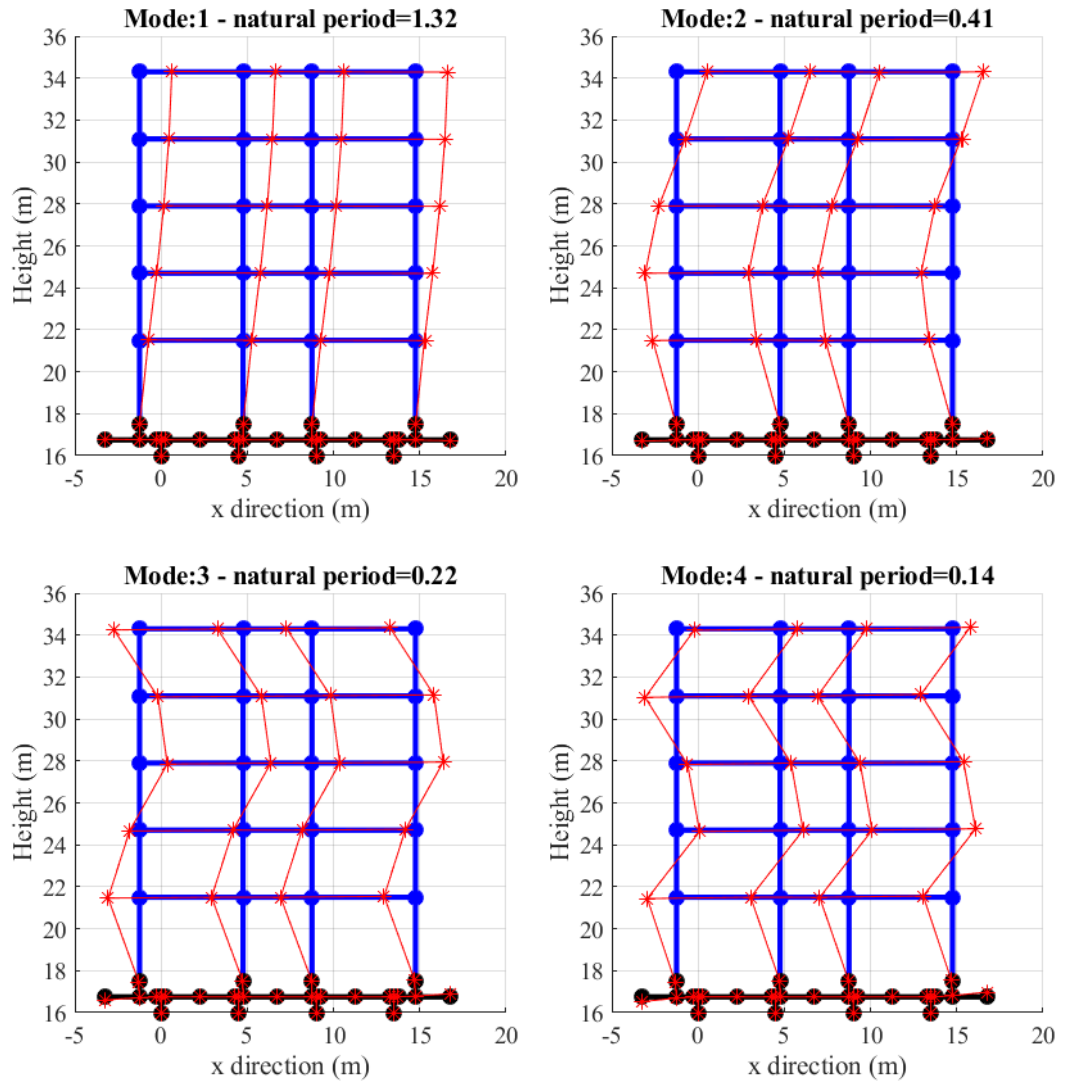


Figure 4.9. Mode shapes of the 5-story frame with cracked section properties

Similarly, the natural periods for the 13-story frame model with and without soil-pile foundation system is provided in the Table 4.7. The mode shapes are given in the Figure 4.10.

Table 4.7 Natural periods, 13-story frame and 13-story frame with soil

Mode	Only Structure	Structure with Soil Domain
1	2.618	2.647
2	0.947	0.949
3	0.547	0.549
4	0.375	0.376

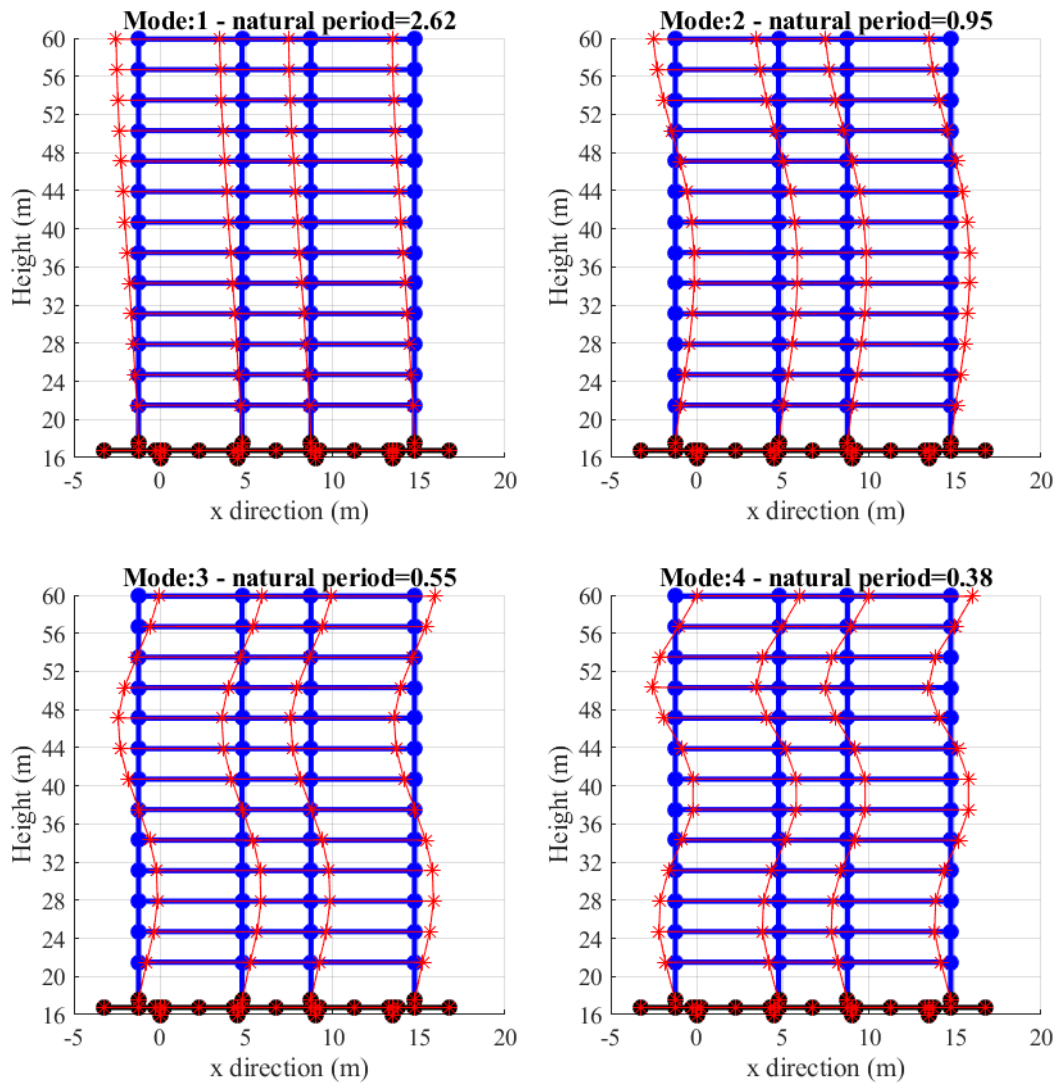


Figure 4.10. Mode shapes of the 13-story frame with cracked section properties

4.3 Soil Only, Free Field Analysis

The buildings rest on a 16m soil layer with a pile foundation. Before detailed analysis of the complete system, site response analysis on the 16m soil column was conducted with the selected ground motions. The natural period of the soil column is estimated as 0.61 sec using the following equation:

$$T_s = \frac{4 \times H}{V_s} \quad (4.1)$$

where T_s is fundamental natural period, H is the total thickness and V_s is the shear wave velocity of the soil (Richart et al., 1970).

The model used to this end consists of 2m square four-node elements and solved in OpenSees. The ground motions recorded on the rock surface is deconvolved to the bottom of the rock layer by multiplying the histories with 0.5. Then this record is applied at the bottom of the soil layer as a velocity time history at the point where the Lysmer-Kuhlemeyer damper element is assigned (Figure 4.11).

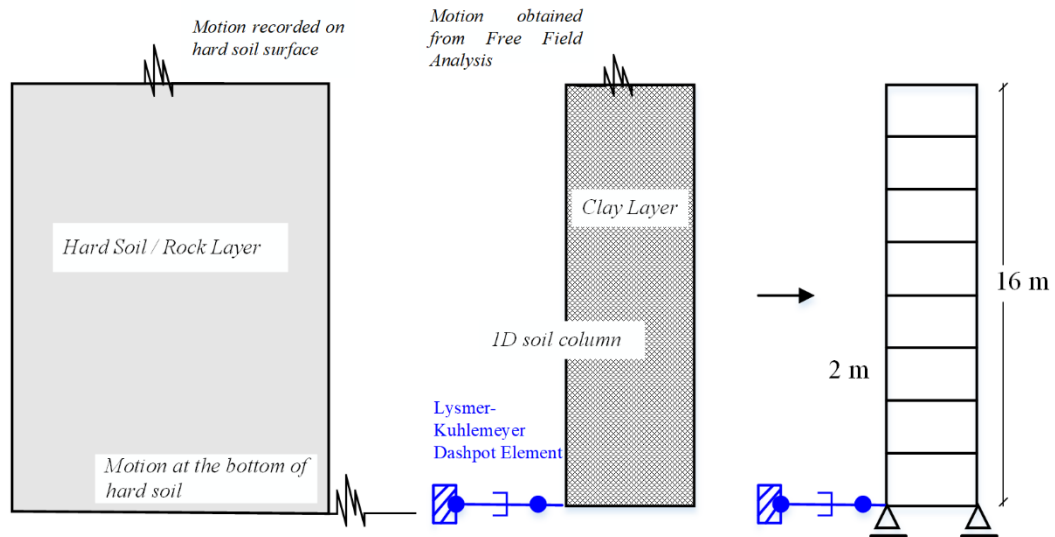


Figure 4.11. Schematic representation of free field analysis

The acceleration, velocity, displacement at each depth of the soil column is obtained from the free field analyses of the selected motions. Force histories at each node is also obtained and used together with the velocity time histories for the estimations of effective earthquake forces used in the analysis of the complete soil-pile-structure system. The maximum and minimum acceleration, velocity and displacement values obtained on the surface as a result of the free field analysis are summarized in the Table 4.8. The absolute maximum values of the acceleration and displacement histories are provided to show the intensity of the selected records on the clay layer in the Figure 4.12.

Table 4.8 Maximum and minimum acceleration, velocity and displacement values at the ground surface

Motion ID	Acceleration (m/s²)		Velocity (m/s)		Displacement (m)	
	Max	Min	Max	Min	Max	Min
1	2.904	-2.807	0.347	-0.472	0.158	-0.189
2	2.639	-2.818	0.240	-0.201	0.077	-0.081
3	2.881	-3.234	0.382	-0.313	0.130	-0.030
4	2.687	-2.312	0.409	-0.443	0.379	-0.446
5	2.588	-2.803	0.177	-0.170	0.033	-0.017
6	1.828	-1.957	0.364	-0.369	0.181	-0.208
7	2.009	-2.257	0.261	-0.298	0.086	-0.078
8	1.970	-1.852	0.130	-0.113	0.016	-0.021
9	2.187	-1.917	0.156	-0.159	0.034	-0.033
10	1.135	-1.071	0.080	-0.081	0.018	-0.029
11	1.423	-1.528	0.273	-0.191	0.047	-0.058
12	0.538	-0.537	0.118	-0.094	0.099	-0.100
13	1.053	-1.124	0.138	-0.132	0.173	-0.075
14	1.785	-1.502	0.267	-0.235	0.128	-0.069
15	1.653	-1.502	0.145	-0.109	0.076	-0.074
16	1.539	-1.330	0.083	-0.066	0.008	-0.008
17	1.288	-1.508	0.194	-0.211	0.063	-0.056
18	1.802	-1.486	0.094	-0.085	0.010	-0.011
19	1.120	-1.019	0.056	-0.059	0.009	-0.008
20	1.380	-1.278	0.049	-0.038	0.004	-0.004
21	1.225	-1.274	0.148	-0.154	0.015	-0.016
22	0.958	-0.764	0.051	-0.042	0.005	-0.006
23	1.800	-1.816	0.143	-0.132	0.027	-0.052
24	1.384	-1.700	0.334	-0.305	0.188	-0.060

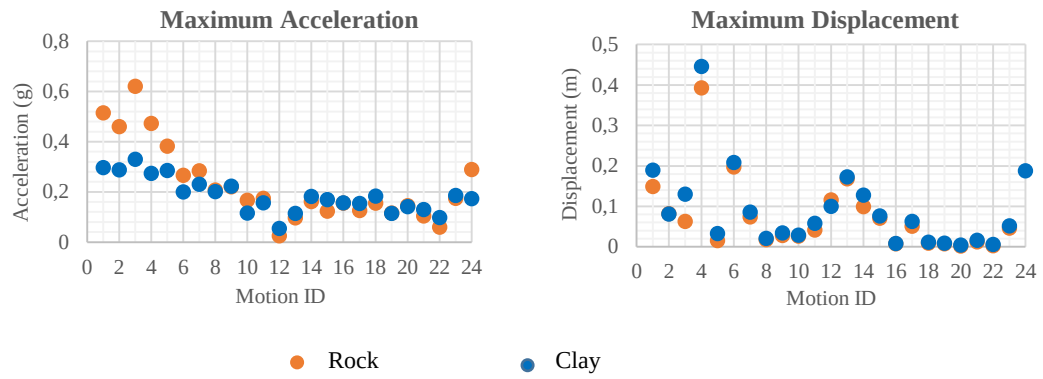


Figure 4.12. Absolute maximum of (a) accelerations and (b) displacements on clay surface and rock surface

Response spectrum curves are obtained from the surface acceleration histories using SeismoSignal. The response spectra for the surface shaking are presented in Figure 4.13 with their (arithmetic) average spectrum.

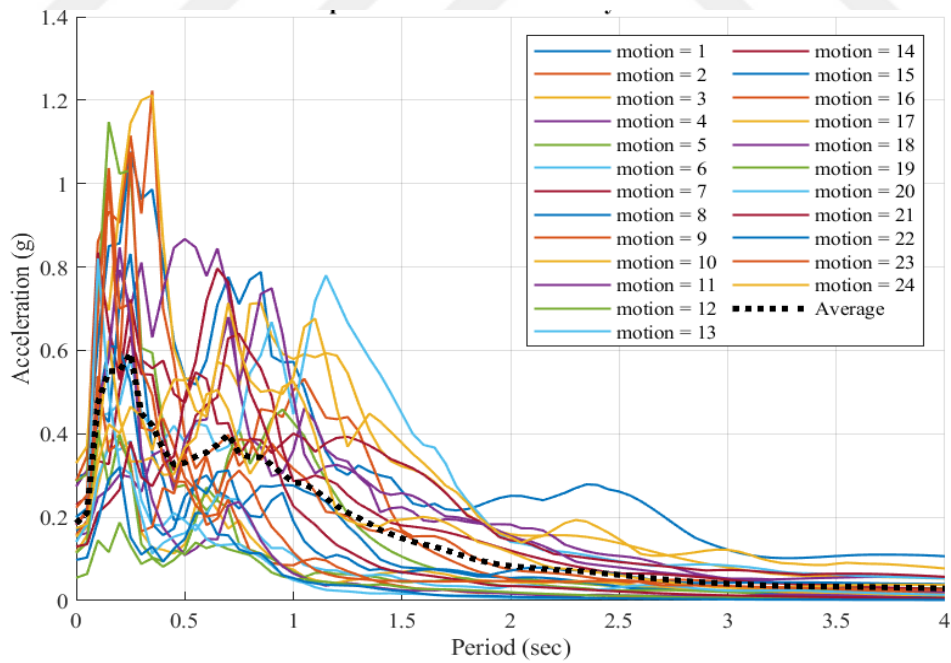


Figure 4.13. Response spectrum on the surface of clay layer

Response spectra are also obtained from the input ground motions on the rock surface, i.e., the time histories of the original ground motion set. The response spectra of the rock motion and the surface shaking on clay layer are compared in Figure 4.14 separately for all motions. The average of the input set and the surface ground motions on the clay layer are compared in Figure 4.15 provided with the first mode natural periods of the frame models with and without soil medium.

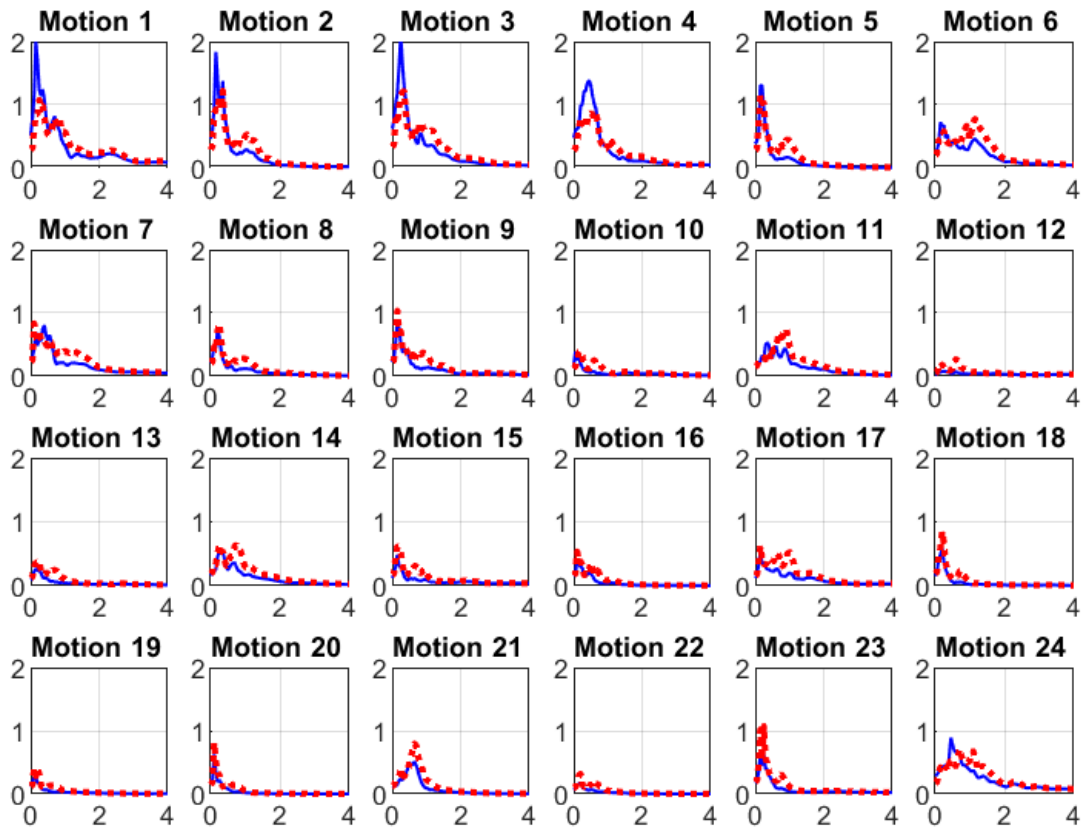


Figure 4.14. Acceleration spectrum on rock and clay surface for all motions

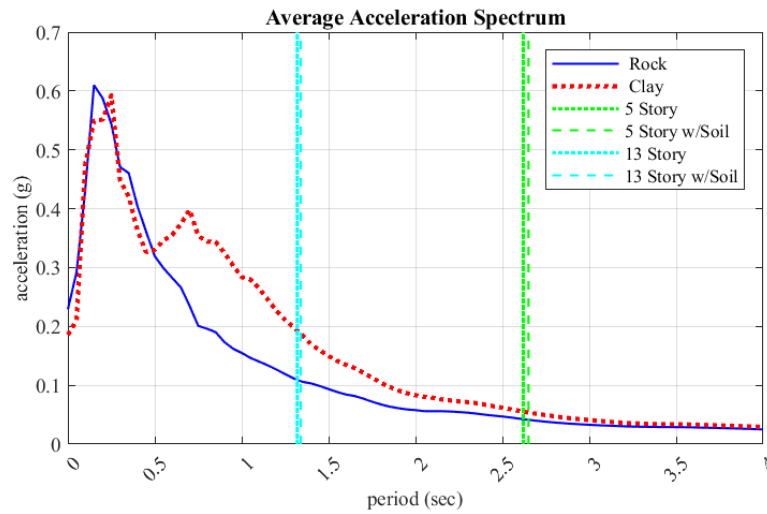


Figure 4.15. Average acceleration spectrum on rock and clay surface for the selected motion set

For considering other hazard levels, the ground motion set was simply scaled for the Peak Ground Acceleration (PGA) which is the PGA of the average is approximately 0.23g. Ground motion scale factors are determined such that the PGA falls between 0.1g and 0.6g in 0.05g intervals. In the Table 4.9 the scale factors according to the target PGA of rock outcrop and the scale factor ID is expressed.

Table 4.9 Scale factors for the target PGA for rock

Scale Tag	Target PGA for Rock	Scale Factors
1	0.1	0.435
2	0.15	0.652
3	0.2	0.870
4	0.23	1
5	0.3	1.304
6	0.35	1.522
7	0.4	1.739
8	0.45	1.957
9	0.5	2.174
10	0.55	2.391
11	0.6	2.609

CHAPTER 5

MID-RISE R/C BUILDINGS – SSI EFFECTS

The buildings supported by pile foundations were subjected to seismic loads as outlined in Chapter 2. Twenty four (24) ground motions are used in the analysis to consider ground motion variability. Each motion was scaled by 11 different scale factors, resulting in a total of 264 dynamic load cases per analytical model. In order to explore the impact of soil-structure interaction (SSI) on the system's dynamic response, different modeling strategies were employed:

- Structural model: Simulations incorporated either linear elastic or nonlinear structural behavior to capture different response characteristics.
- Soil model: The soil medium was represented using either linear elastic or nonlinear material models to assess the effect of soil softening.
- Pile response: The piles were modeled assuming linear elastic behavior.

The pile group and cap beam were consistently modeled with elastic but cracked section properties. These combinations of assumptions allowed for a detailed investigation into the influence of interaction effects and nonlinearities on the system response. The analytical models were classified into the following cases:

- Group 1: Both the soil medium and the superstructure were modeled with elastic material properties.
- Group 2: The soil medium was represented with nonlinear material properties, while the moment-resisting frame representing the superstructure was modeled with elastic but cracked section properties.
- Group 3: Both the soil medium and the superstructure were characterized by nonlinear material behavior, providing a fully nonlinear representation of the system.

The model cases are demonstrated in the schematics given in Figure 5.1.

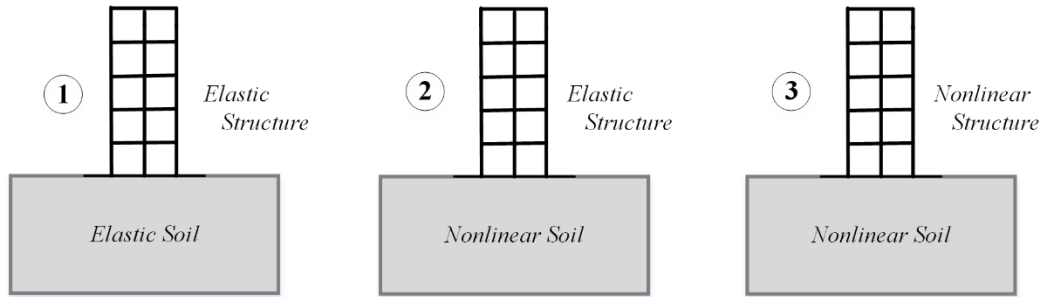


Figure 5.1. Modeling approaches of the complete soil-pile-structure system

These modeling approaches were applied to both a 5-story and a 13-story building model. Consequently, a total of 1,584 dynamic analyses were performed, considering two structural configurations and three model cases, for 264 dynamic analysis. To further evaluate the influence of cap beam stiffness, additional analyses were conducted by varying the cap beam properties. Specifically, uncracked and rigid cap beam configurations were tested within Group 2, resulting in an additional 1,056 dynamic analyses. This brought the total number of simulations to 2,640, providing a comprehensive assessment of soil and structural nonlinearity effects on seismic response. In the following table, the summary of the analysis are provided.

Table 5.1 Summary of dynamic analyses

Buildings	5 Story	13 Story
Group 1 (Cap Beam stiffness: $0.4 \times E$)	264	264
Group 2 (Cap Beam stiffness: $0.4 \times E$, E and $10 \times E$)	3 x 264	3 x 264
Group 3 (Cap Beam stiffness: $0.4 \times E$)	264	264
Total	2640	

To thoroughly capture the dynamic behavior and key response characteristics of the system, a range of engineering demand parameters (EDPs) were extracted from the analytical results. These EDPs included maximum and minimum displacements,

rotations, velocity, and acceleration histories in both the piles and the superstructure. Additionally, shear force and moment values were recorded for columns and piles to assess the load transfer behavior.

The subsequent sections present a detailed analysis of the results based on selected key output parameters, highlighting the important factors influencing the overall structural response.

5.1 Analysis Results

Among the various demand parameters, base shear, drift ratio and the pile moments are the selected response parameters for performance evaluation.

To provide a comprehensive overview of the analyses, the results from Group 1, Group 2, and Group 3 are presented collectively. Base shear, drift ratio, and pile moments obtained from each analysis group are plotted against the corresponding spectral acceleration (S_a) values. The S_a values are determined at the first-mode period of the fixed-base superstructure using the response spectrum curves of the original ground motions. By multiplying the spectral acceleration values of 24 ground motions by 11 scaling factors, the S_a values corresponding to each dynamic analysis and each demand parameter are obtained. Spectral acceleration values of the original motions at the first mode period of the structures are given in the Table 5.2. In addition, in the Figure 5.2, spectral accelerations for each motion at the natural frequency of the 5 and 13 story buildings are plotted.

Table 5.2 Spectral accelerations of unscaled ground motions

5-Story Building (Period=1.32 sec)				13-Story Building (Period=2.62 sec)			
Motion	Sa (g)	Motion	Sa (g)	Motion	Sa (g)	Motion	Sa (g)
1	0.210	13	0.036	1	0.170	13	0.031
2	0.184	14	0.130	2	0.028	14	0.046
3	0.257	15	0.047	3	0.067	15	0.047
4	0.141	16	0.027	4	0.081	16	0.007
5	0.070	17	0.107	5	0.014	17	0.043
6	0.361	18	0.024	6	0.067	18	0.011
7	0.205	19	0.024	7	0.064	19	0.009
8	0.065	20	0.009	8	0.024	20	0.003
9	0.103	21	0.054	9	0.042	21	0.011
10	0.036	22	0.016	10	0.033	22	0.002
11	0.148	23	0.032	11	0.049	23	0.036
12	0.040	24	0.255	12	0.023	24	0.097

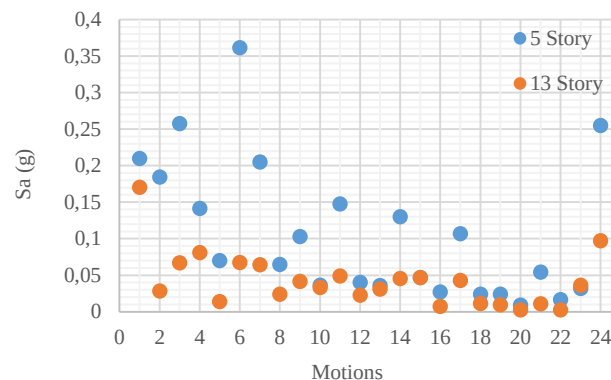


Figure 5.2. Spectral acceleration of input motions (rock surface) at 1st mode periods for 5-story and 13-story frame models

5.1.1 Base Shear

Base shear is a critical design parameter representing the total horizontal force exerted on the structure due to earthquake loads. The base shear time history during the analysis was determined by summing the shear forces at the bottom of all columns at each time step. The maximum base shear was then obtained as the peak

absolute value from this time history, representing the largest force induced by the applied ground motion. In the following figures base shear results are presented for the Group-1, 2 and 3 analysis sets. For a generic expression, base shear values are normalized with respect to weight of the structure excluding the cap beam. Figures 5.3 and 5.4, the weight normalized base shear are presented with respect to the spectral acceleration value of each motion, for 5 story and 13 story building models, respectively.

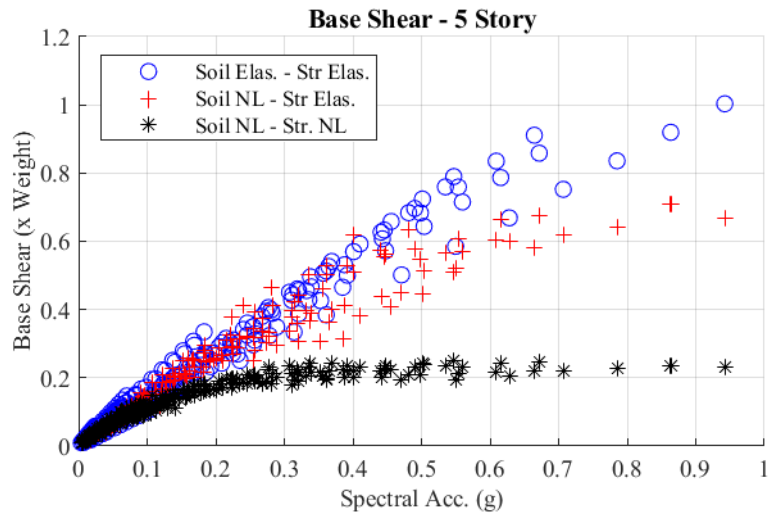


Figure 5.3. Weight normalized base shear with respect to S_a for the 5-story building

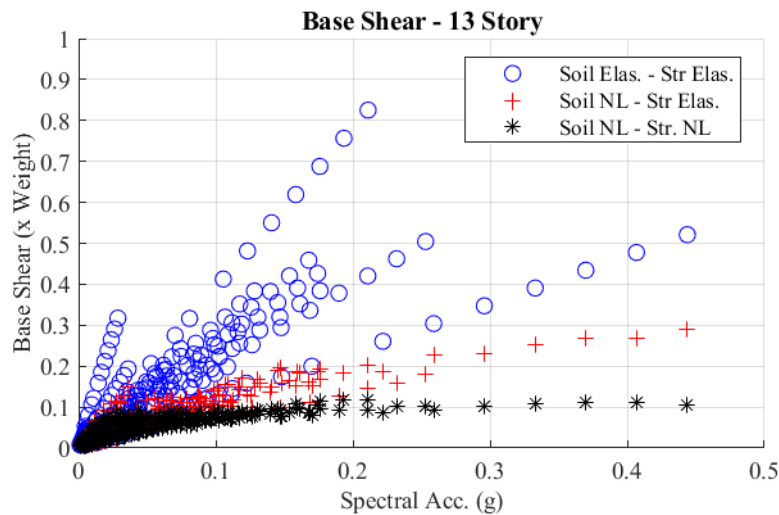


Figure 5.4. Weight normalized base shear with respect to S_a for 13-story building

Based on the results obtained from the fully elastic model, the base shear exhibits a linear relationship with increasing spectral accelerations. For all the ground motions, base shear exhibit an elastic response with a consistent slope, closely following a 45° line, which represents the first-mode response of the structure. As nonlinearity increases, first in the soil and then in both the soil and structure, the base shear response begins to plateau, indicating a shift from elastic behavior to a nonlinear regime. In the fully nonlinear case, where both the soil and structure exhibit nonlinearity, the load factor - defined as the ratio of base shear to the building's weight - stabilizes at approximately 0.2 for 5-story building.

For the 13-story building, a linear response is also observed in the fully elastic case; however, the dynamic response varies depending on the ground motion. This variation indicates that the characteristics of the ground motion significantly influence the results, primarily due to higher-mode effects, which are not as apparent in the analysis of the 5-story frame model. Similarly, as nonlinearity in the overall system increases, the base shear values tend to stabilize at a certain level. Unlike the 5-story building, the influence of soil nonlinearity on base shear is more pronounced. Specifically, nonlinear behavior begins to emerge at approximately 0.5g for the 5-story building, whereas for the 13-story building, it is observed at around 0.2g. In the 13-story building, the larger building mass results in greater inertial forces acting on the foundation and surrounding soil. Consequently, the soil medium is more likely to enter the nonlinear range compared to the 5-story counterpart. In the fully nonlinear case, the ratio approaches a steady value of approximately 0.12. The reference 12-story building model, developed by Alici and Sucuoğlu (2015), was designed in accordance with the Turkish Earthquake Code (TEC) 2007 spectrum for soil type Z3 with a PGA of 0.4g, considering a response reduction factor (R) of 8.

5.1.2 Drift Ratio

Drift ratio measures the relative lateral displacement between floors and serves as a direct indicator of deformation and potential structural damage during seismic

events. Therefore, it is a critical engineering demand parameter for assessing the seismic performance and overall stability of buildings. To provide a comprehensive overview of structural behavior before conducting detailed response evaluations, the average drift ratio along the building height is presented in Figure 5.5 and 5.6 for 5- and 13-story frames, respectively. Specifically, the drift ratio is estimated considering the displacement history of right-most columns from each analysis. Drift ratio at each story and for each ground motion “m” at a specific scale factor is estimated as the ratio of the displacement difference of consecutive stories to the story height, using the following equation:

$$Dr_{m,n,t} = \frac{dx_{n,t} - dx_{n-1,t}}{h} \quad (5.1)$$

where “dx” is the displacement at “nth” story at time step “t” and “h” is the story height. The absolute maximum drift ratio is determined for each motion “m” and for each story for a specific scale factor such that

$$Dr_{m,n} = \max\left(\text{abs}(Dr_{m,n,t})\right) \quad (5.2)$$

$Dr_{m,n}$ is the maximum drift ratio at nth story for a specific motion at a specific scale factor. Subsequently, the average drift ratio (Dr_n) at each story “n” is computed as the arithmetic mean of the results from 24 ground motion for each scale factor at each story as shown in Equation 5.3.

$$Dr_n = \text{Average}(Dr_{m,n}) \quad (5.3)$$

The results are plotted for different modeling approaches, including (i) elastic soil-elastic structure (Case 1), (ii) nonlinear soil-elastic structure (Case 2), and (iii) nonlinear soil-nonlinear structure (Case 3). To facilitate an effective comparison and assess the influence of soil and structural nonlinearity, the results are presented as Case 1 vs. Case 2 and Case 2 vs. Case 3 comparisons. This approach enables a direct evaluation of drift demands across different building heights, seismic intensities, and

modeling approaches, providing a preliminary characterization of the structure's global deformation pattern. The average drift ratio results are provided in the Table A.1 and A.2 in the Appendix, for both 5-story and 13-story building models.

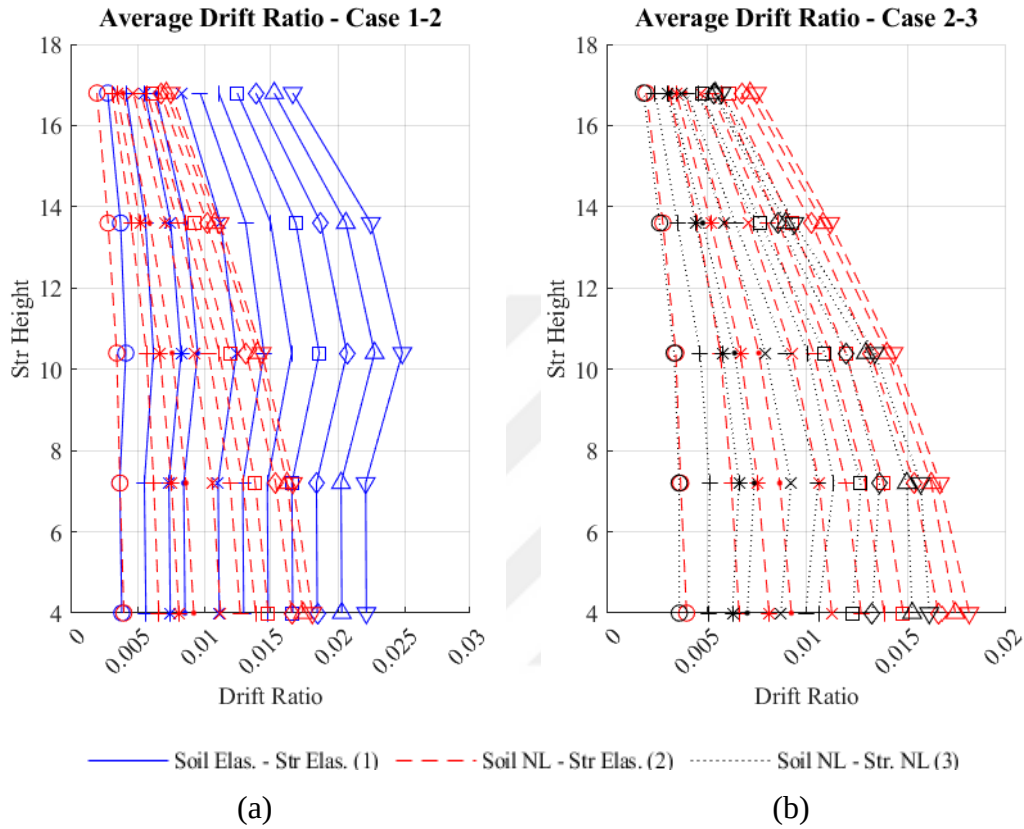


Figure 5.5. Average of drift ratio for each scale factor along the structure, 5-story frame

The average drift ratio values plotted for Case-1 vs Case-2 comparison in Figure 5.5a show that the soil nonlinearity decreases the drift ratio at all heights of the structure. The decrease at the upper stories are more significant compared to the lower stories. The soil nonlinearity adds significant damping on the system. In Figure 5.5b, Case-2 and Case-3 results are compared. The soil nonlinearity still exists in the models and the effect of structural nonlinearity is observed. Structural nonlinearity leads to a small reduction in drift ratio values. For a numerical representation of the variations in drift ratio, the relative change was estimated between Case 1-2 and Case 2-3

results. With this purpose, the average change in story drifts are estimated using the formulations 5.4 and 5.5. In the Table 5.3, the decrease in the average drift ratio is provided. Soil nonlinearity reduces the average drift ratio amount on the upper floors by up to 43% while the decrease in the average drift ratio is around 23%.

$$\Delta D_{R_{story, Soil NL}} = Average \left[\frac{(Case 2_{per scale} - Case 1_{per scale})}{Case 1_{per scale}} \times 100 \right] \quad (5.4)$$

$$\Delta D_{R_{story, Str NL}} = Average \left[\frac{(Case 3_{per scale} - Case 2_{per scale})}{Case 2_{per scale}} \times 100 \right] \quad (5.5)$$

Table 5.3 Average drift ratio change for 5-story building

Story	Case 1-2	Case 2-3
1	-2.18%	-18.78%
2	-7.63%	-10.58%
3	-27.65%	-10.81%
4	-37.50%	-16.30%
5	-43.44%	-19.61%
Average	-23.68%	-15.21%

Similarly, for the 13-story building model, the soil nonlinearity clearly decreases the drift ratio demands. Modeling the soil with elastic properties reveals higher in the drift ratio at all levels of the structure, highlighting the influence of damping. Unlike to the 5 story building case, the decrease in drift ratio is more significant at upper heights. The average variations in the drift ratio at each story are estimated using the formulations 5.4 and 5.5 and presented in the Table 5.4. According to the results, the difference in the average drift ratio along the height is at almost similar levels compared to the variations in the 5-story building model for Case 1-2 comparison. Structural nonlinearity generally reduces the drift ratio at higher floors, but not so much for the lower floors. It is important to note that the influence of structural nonlinearity is examined in scenarios where soil nonlinearity is also considered for

both the 5-story and 13-story models. In addition to the average absolute maximum drift ratio values, the time history results for Case-1,-2, and -3 at scale ID 4 which is the original ground motion set are provided in the Appendix to illustrate the overall behavior during the shaking event.

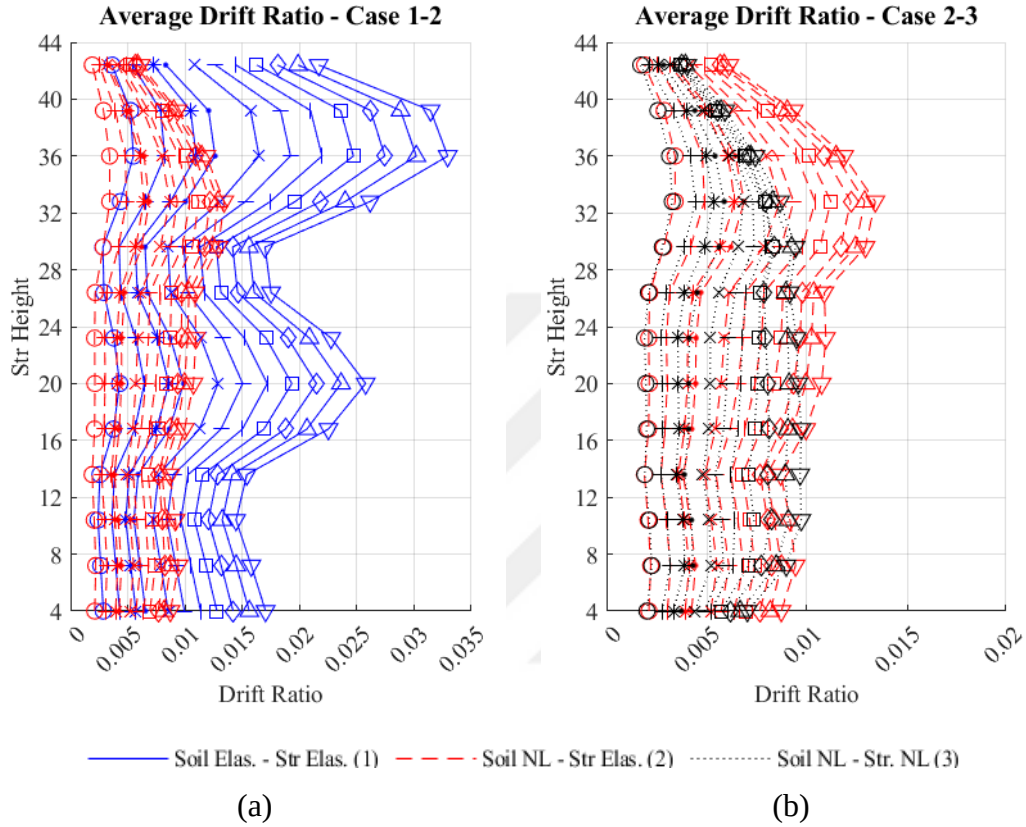


Figure 5.6. Average of drift ratio for each scale factor along the structure, 13-story frame

Table 5.4 Average drift ratio change for 13-story building

Story	Case 1-2	Case 2-3
1	-39.54%	-14.81%
2	-30.68%	-3.78%
3	-27.96%	1.74%
4	-36.75%	2.59%
5	-51.67%	-4.75%
6	-55.41%	-9.68%
7	-48.56%	-11.75%
8	-31.34%	-10.08%
9	-10.52%	-17.06%
10	-36.36%	-22.59%
11	-53.01%	-24.91%
12	-60.37%	-25.70%
13	-62.59%	-23.84%
Average	-41.91%	-12.66%

To examine the effect of each motion individually, drift ratio values at the first story are plotted in Figures 5.7 and 5.8 for the 5-story and 13-story building models, respectively. These figures show that some motions have little effect, even after scaling. Certain motions that result in higher drift ratios at the first story are common in both the 5-story and 13-story models. However, overall, the same set of motions tends to cause greater first-story drift ratios in the 5-story building compared to the 13-story building as the 5-story building experiences greater earthquake loads due to its shorter natural period. For the 13-story building, larger drift ratios are observed at the upper floors.

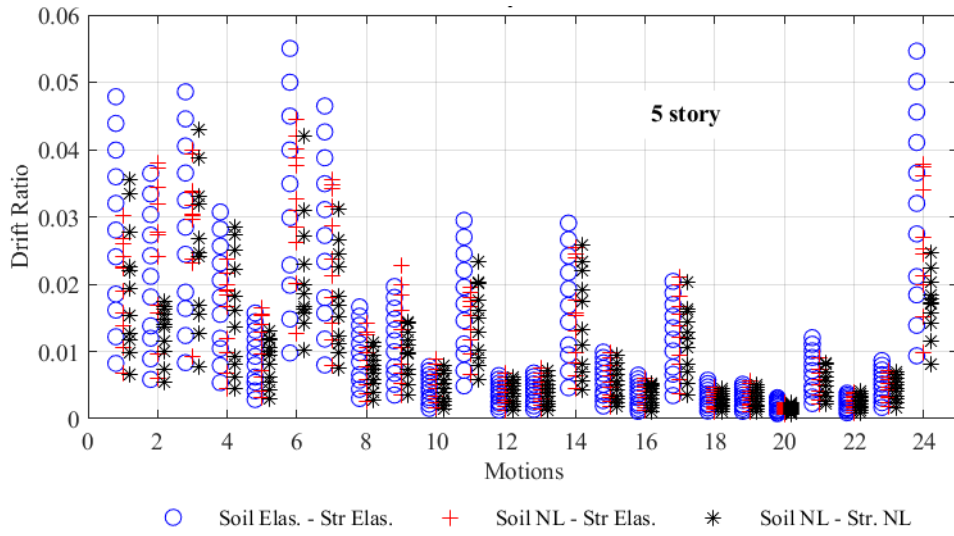


Figure 5.7. First story drift ratio for each motions, 5-story building model

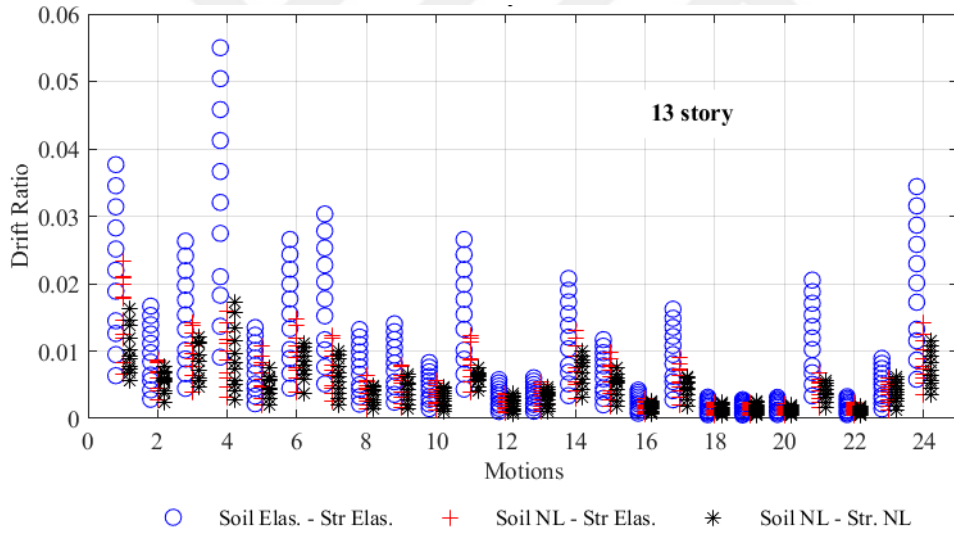


Figure 5.8. First story drift ratio for each motions, 13-story building model

The ground motions having the smallest and largest influence on the first story drift ratio are determined as the 6th and 20th motions for the 5-story building model and as the 4th and 19th motions for the 13 story building models. For this specific motions the drift ratio values are plotted along the building height for all analysis cases in Figure 5.9. For larger drift ratio values in the fully elastic model, the impact of soil

nonlinearity on the drift ratio is more pronounced. This effect is particularly observed when comparing the changes in drift ratio for motion 4 and motion 6 for both of the structures.

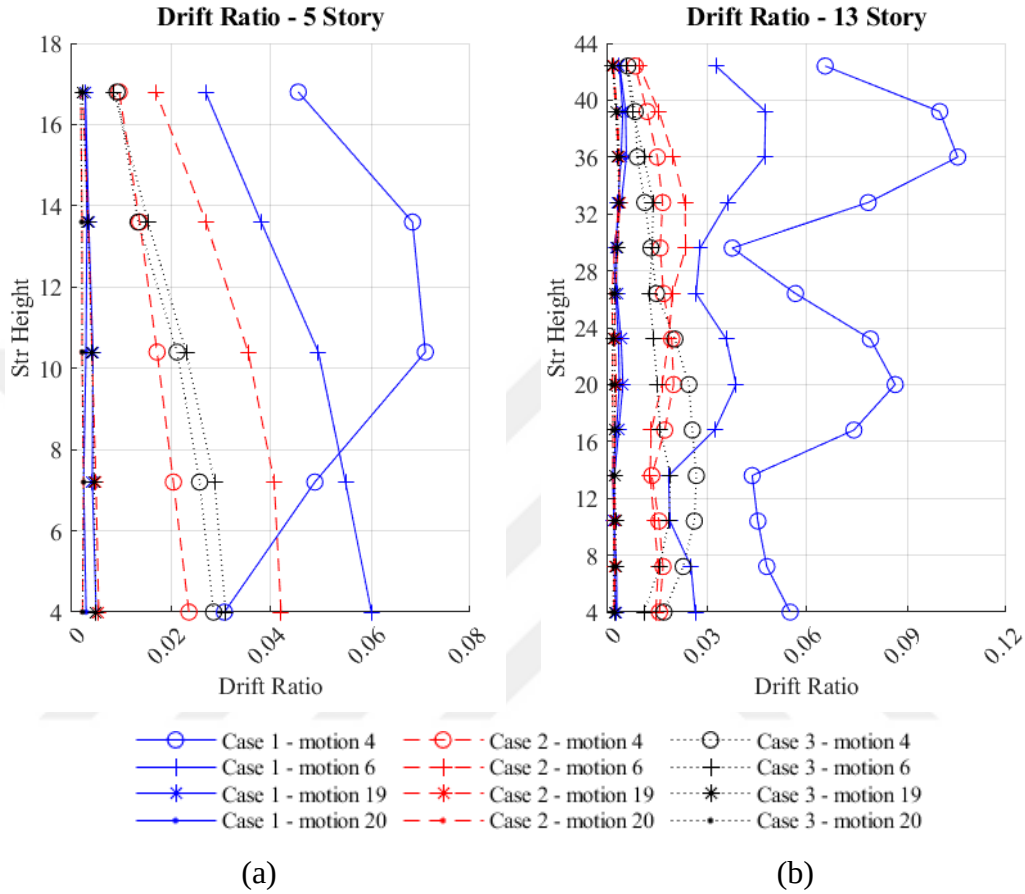


Figure 5.9. Drift ratio along the structure for the smallest and largest effective motions at the maximum scale factor for (a) 5-story and (b) 13-story building

As in the base shear results presentation, the first story drift ratio results with respect to the spectral acceleration values are also provided in Figures 5.10 and 5.11 for the 5-story and 13-story buildings, respectively. The variations in the drift ratio is influenced by the sequence in which columns transition into the nonlinear phase leading to non-uniform behavior across different analyses. Consequently, while certain analyses in the complete nonlinear case (Case-3) exhibited results similar to those obtained in Case-2, others demonstrated more pronounced nonlinear behavior for the 5-story building.

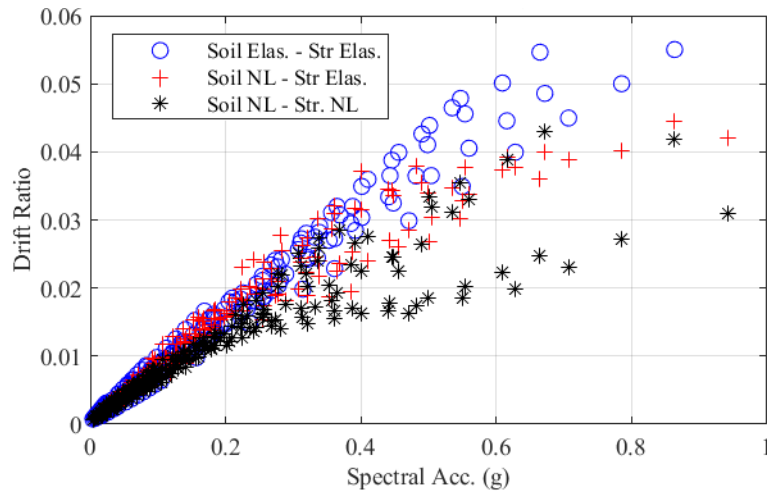


Figure 5.10. First story drift ratio with respect to S_a for 5-story building

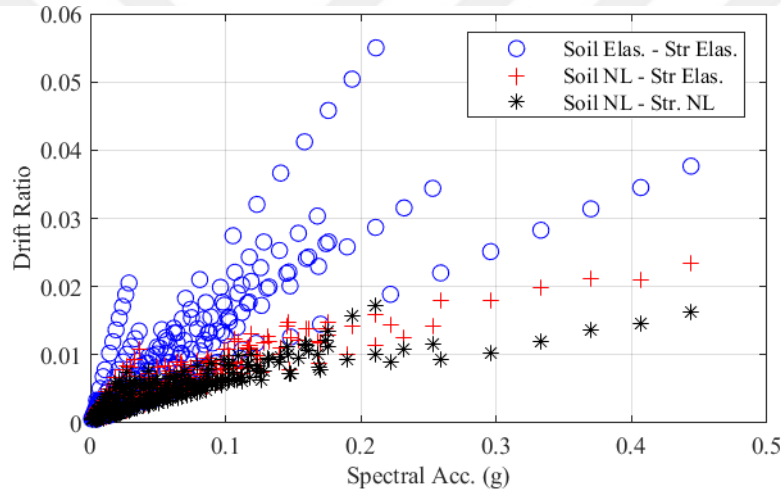


Figure 5.11. First story drift ratio with respect to S_a for 13-story building

5.1.3 Pile Moments

Pile moments are assessed next as the inspection, maintenance, or repair procedures are significantly more challenging and expensive compared to the superstructure components. Consequently, a thorough assessment of the pile demands is essential.

Pile moment distributions from the time history analysis were obtained along the pile length, and the absolute maximum values were recorded as the demand parameter.

The results are presented for end and mid-piles separately. To account for the bidirectional nature of seismic loading, the average of the maximum moments for the end piles (Pile 1 and Pile 4) and the middle piles (Pile 2 and Pile 3) are presented as Pile 1 and Pile 2, respectively. In other words, the average of the moments for Pile 1 is estimated as the envelope of the outer piles for a scale factor as indicated in Equation 5.6.

$$\begin{aligned} \text{Average Moment}_{at\ depth=h,\ scale\ fact.=k} & \quad (5.6) \\ & = \text{mean} [\max(\text{abs Pile 1})\ \max(\text{abs Pile 4})] \end{aligned}$$

In this formulation, absolute maximum of the outer piles are determined for each time history analysis, then the mean of the maximum moments from 24 motions for Pile 1 and 4 is calculated. In other words, for a scale factor and a specific depth, the mean of 48 absolute maximum moment is obtained. Similar approach is used for the 2nd pile considering the piles 2 and 3. Figure 5.12 and 5.13 illustrate the average pile moments for Pile 1 (envelope of Pile 1 and Pile 4) and Pile 2 (envelope of Pile 2 and Pile 3) under 24 ground motion records, plotted for each scale factor, corresponding to both the 5-story and 13-story buildings.

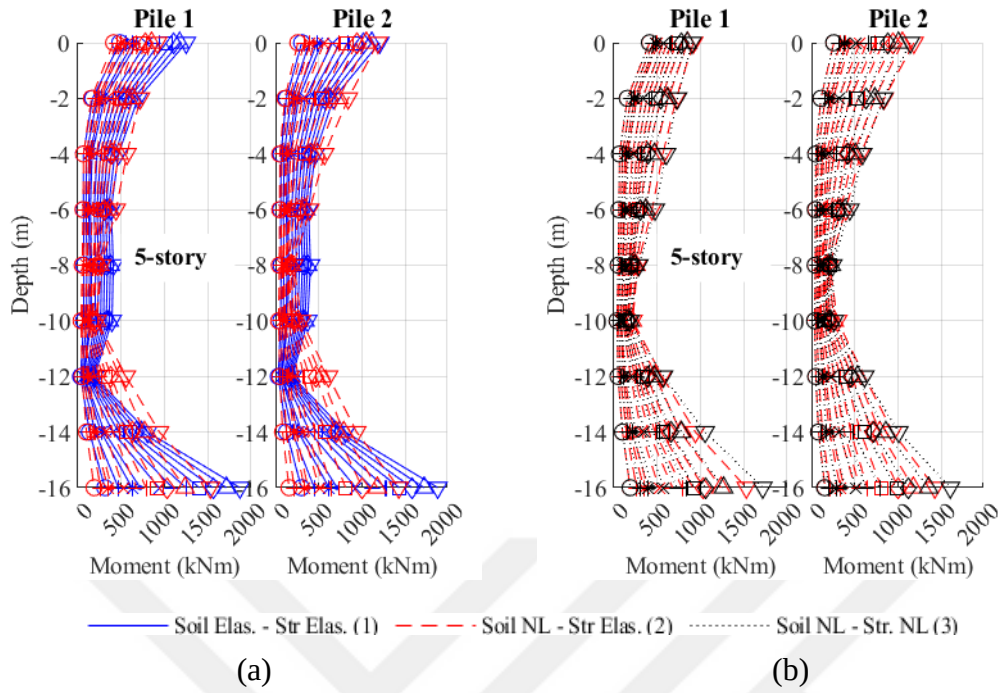


Figure 5.12. Average pile moments: (a) Case 1-2 and (b) Case 2-3, 5-story frame

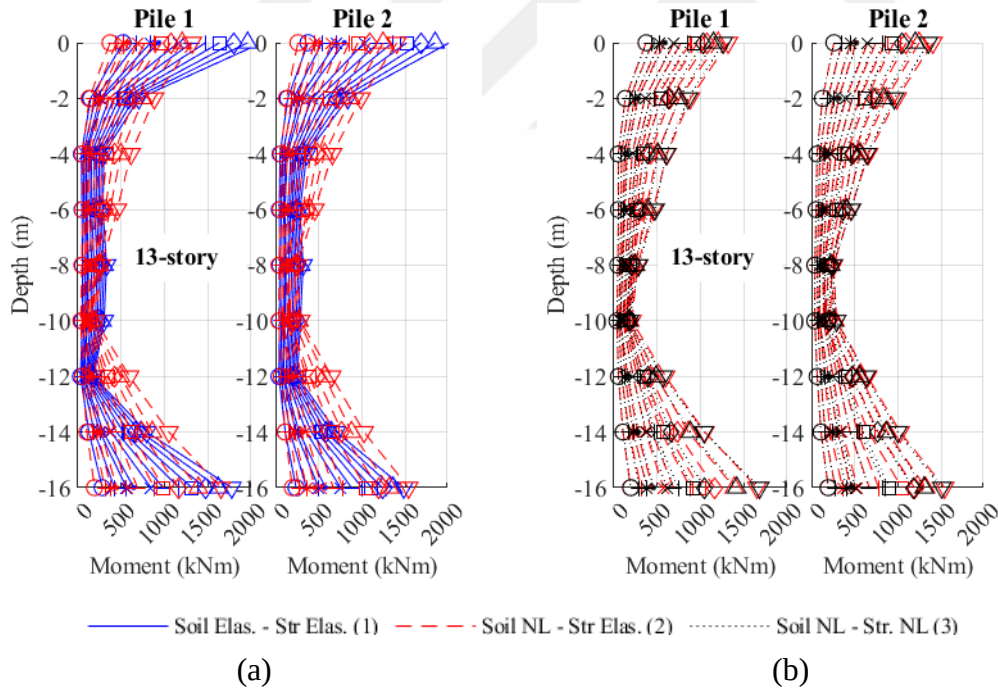


Figure 5.13. Average pile moments: (a) Case 1-2 and (b) Case 2-3, 13-story frame

For both 5-story and 13-story building models similar inferences are reached. The maximum moments occurs at the pile top and bottom regions due to the rotational

restriction assumption at the top and bottom. From the average moment results, soil nonlinearity seems to cause much difference compared to the elastic case and shows a damping effect. In other words, soil nonlinearity decreases the moments exerted on the piles especially at the top and bottom locations of the pile where the maximum moments arise. However, in the middle sections of the pile, close to the top and bottom regions, pile moments are higher in the nonlinear soil model case. On the other hand structural nonlinearity does not seem to significantly affect the demands in pile moments. Effect of the structure seems in the amount of top moments of the piles. In the 5-story building model top moments are smaller compared to the 13-story building model.

To comprehensively present the results of each analysis, the moment values at the pile-to-cap connection region can be examined. Figures 5.14 and 5.15 illustrate the pile top moments obtained from each dynamic analysis for the 5-story building. These figures indicate that soil nonlinearity generally reduces pile top moments in the outer piles (Pile 1). However, for the inner piles (Pile 2), soil nonlinearity may lead to an increase in pile top moments. For some motions, this increase follows a parabolic trend, especially for the inner piles, implying that in case of higher spectral acceleration data greater moments may be obtained. Regarding the structural nonlinearity, pile head moments appear to be minimally affected, as their moment values remain relatively consistent (Figure 5.14b), but it is more pronounced for the outer piles.

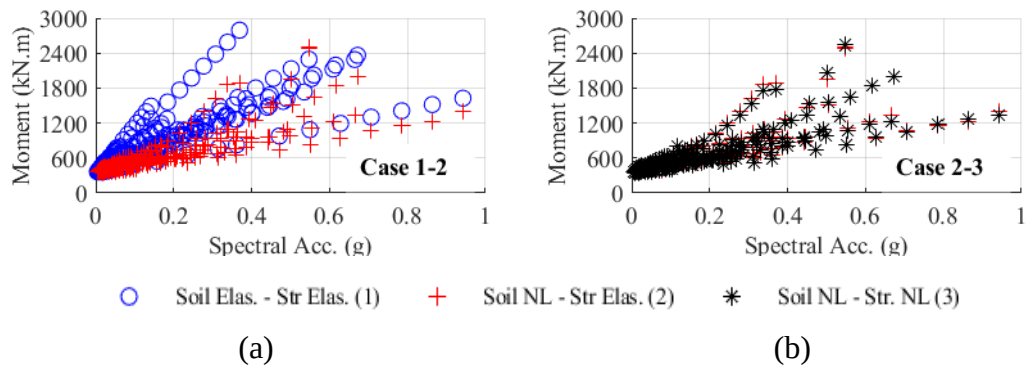


Figure 5.14. Pile-1 top moments with respect to S_a for 5-story building

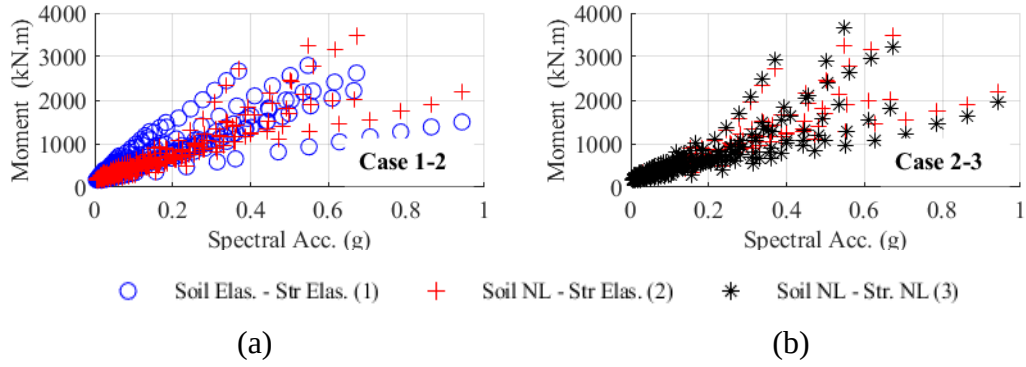


Figure 5.15. Pile-2 top moments with respect to S_a for 5-story building

The top moments of the piles for 13-story building model are presented in Figures 5.16 and 5.17. Soil nonlinearity decreases the pile head moments in general overview for both inner and outer piles. When the Case-2 and Case-3 results are compared, the pile head moments are very close to each other at the small spectral acceleration values. However, when the spectral acceleration increases, nonlinearity in the structure seems to decrease the pile head moments for some motions.

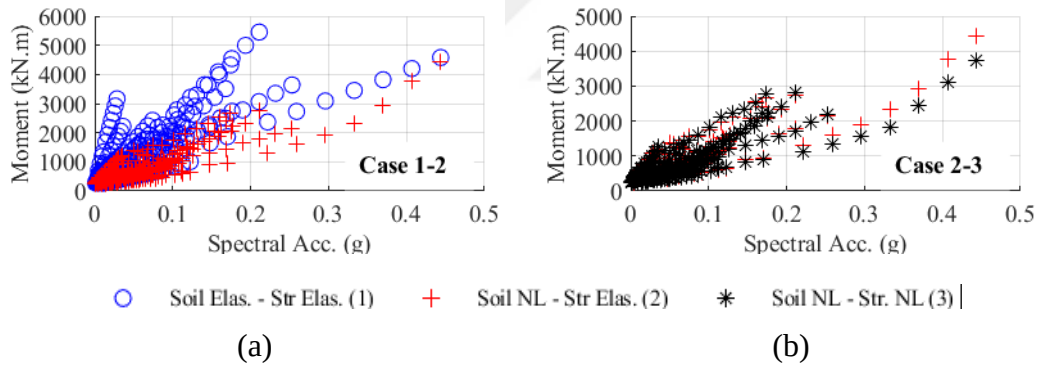


Figure 5.16. Pile-1 top moments with respect to S_a for 13-story building

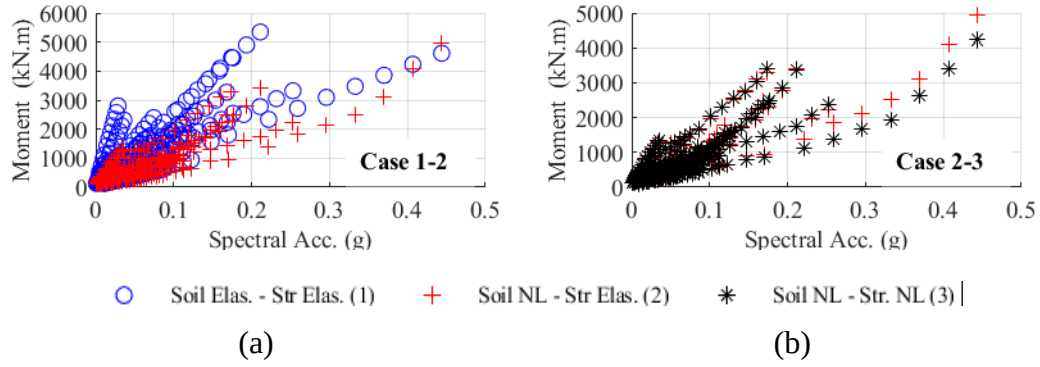


Figure 5.17. Pile-2 top moments with respect to S_a for 13-story building

In the following parts, the influence of soil and structural nonlinearity on structural demands is examined by considering different modeling approaches. The ratio of structural demand estimations between Group-1 and Group-2 serves as an indicator of the soil's reduction or amplification effect, while the ratio between Group-2 and Group-3 reflects the impact of structural nonlinearity on the demand parameters while nonlinear soil-pile interaction is present.

5.2 Effect of Soil Nonlinearity

To assess the impact of soil nonlinearity on the demands experienced by the structure and piles, the analysis results from Group 1 are compared to those from Group 2. This comparison quantifies the effect of soil nonlinearity on the structural response. Specifically, the R factor represents the ratio for force demands, while the μ factor corresponds to the ratio for displacement demands.

$$R_{soil\ NL} \text{ or } \mu_{soil\ NL} = \frac{\text{Group 1 results}}{\text{Group 2 results}} \quad (5.7)$$

5.2.1 Base Shear

The ratio between the Group-1 and Group-2 analysis results is plotted with respect to 264 spectral acceleration (S_a) scenarios, as illustrated in Figure 5.18. In the figure each color represents a specific motion for different scale factors. In these analysis

groups, the soil is modeled with elastic and nonlinear properties, while the superstructure is characterized by elastic properties. To quantitatively assess the effect of soil nonlinearity, the ratio of base shear results from Group-1 and Group-2 analyses is provided in Tables A.3 and A.4 for the 5- and 13-story building models in Appendix.

As both 5-story and 13-story buildings are modeled with identical features, similar effects of soil nonlinearity would be expected, regardless of the building model. However, the influence of applied ground motion characteristics on demands in the complete elastic model (Group-1) becomes apparent, particularly for the 13-story building. As shown in Figure 5.18, the R factors for the 5-story building exhibit a consistent trend within a specific range of around 1 to 2, even for high spectral acceleration (S_a). In contrast, the R factors for the 13-story building display greater dispersion. The dynamic characteristics of the superstructure, such as mode shapes and natural periods, play a significant role in determining the extent to which soil nonlinearity influences structural demands.

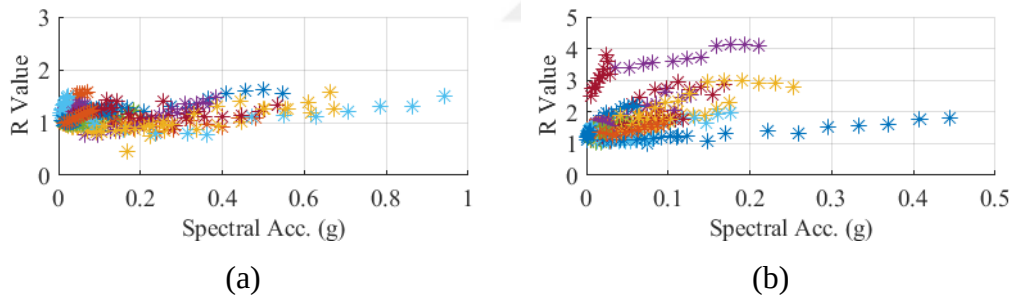


Figure 5.18. Effect of soil nonlinearity on base shear (a) 5-story building (b) 13-story building

For the 5-story building model, from the results of the 264 analyses of each analysis group, the arithmetic mean of the $R_{soil,NL}$ factor has been calculated to be 1.12. The variability of the data is assessed by calculating the coefficient of variation (COV), defined as the ratio of the standard deviation to the mean. The resulting COV is approximately 17.0%. In contrast, the 13-story building model exhibits a higher mean $R_{soil,NL}$ factor of 1.76, accompanied by a larger COV of 37.8%. The disparity

in $R_{\text{soil,NL}}$ factors between the two building models suggests that the influence of soil nonlinearity on structural response seems not to be uniform across different building heights, with greater variability and effect on taller structure.

5.2.2 Drift Ratio

Drift ratios are compared similarly, for elastic structural system and elastic soil (Group 1) and nonlinear soil (Group 2). The results are presented for the 1st story drift ratio in the Figure 5.19. They are similar to the base shear comparisons.

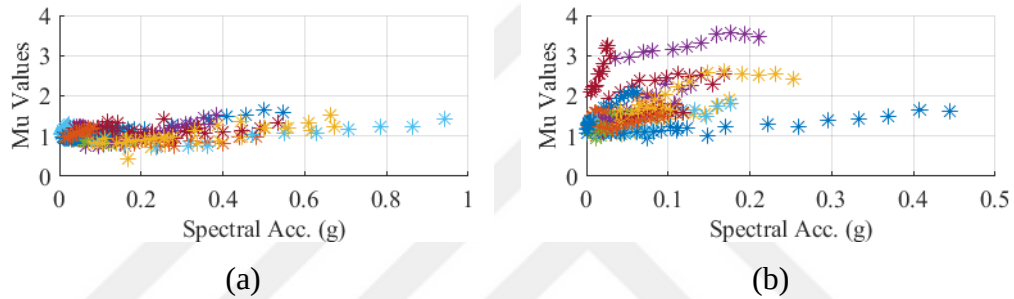


Figure 5.19. Effect of soil nonlinearity on 1st story drift ratio (a) 5-story building
(b) 13-story building

To put it numerically, $\mu_{\text{soil,NL}}$ estimations for 5- and 13-story building models across all scale levels are presented in the Table A.5 and Table A.6 in Appendix. Similar pattern to the ratio for base shear findings are also obtained for the drift ratio. For the 5-story building model, the average $\mu_{\text{soil,NL}}$ factor is determined to be 1.05, with a coefficient of variation (COV) of 16.4%. Conversely, the 13-story building model shows a greater mean $\mu_{\text{soil,NL}}$ factor of 1.61, along with a higher COV of 34.3%.

5.2.3 Pile Moment

For the estimation of the $R_{\text{soil,NL}}$ factor for pile moments, the average moments along the pile length are utilized. The average pile moments at a scale of 24 motions are computed at each depth as described in section 5.1.3. Then, the average moments of Group 1 analysis are divided by those obtained from Group 2 analysis. The ratio

between the Group 1-2 analyses is plotted along the pile depth in the Figure 5.20 for each scale factor. The influence of soil nonlinearity is most pronounced at the pile base and the mid-height near the pile base. However, since the maximum moments occur at both the top and bottom sections of the pile, it is reasonable to consider the R values obtained at those locations. With this purpose, the ratio of the top moments obtained from Group 1 and Group 2 analyses are plotted with respect to the spectral acceleration values.

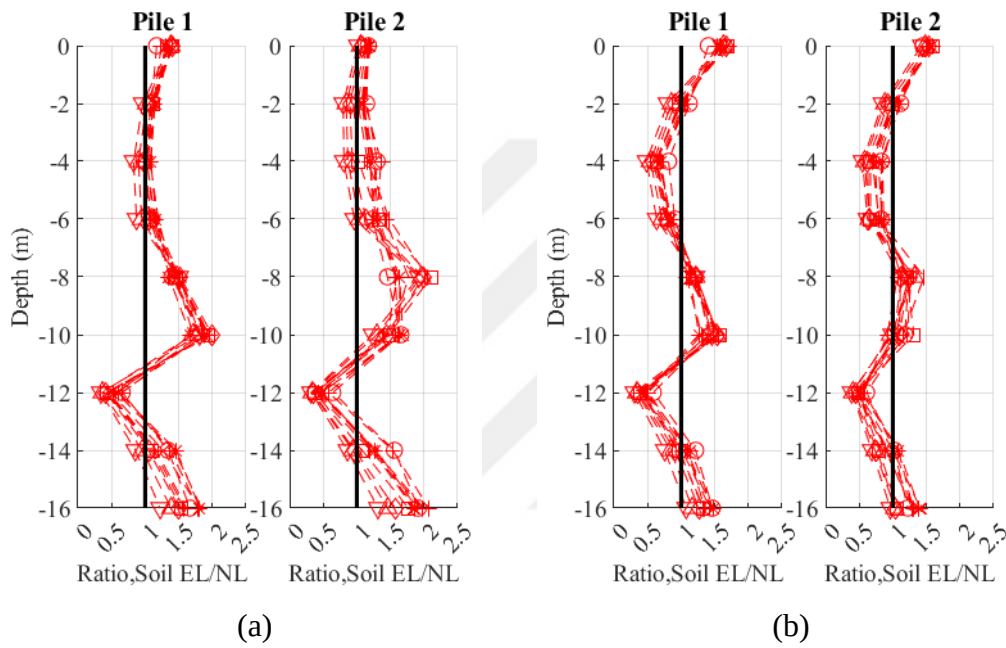


Figure 5.20. Pile moment ratio for soil nonlinearity comparison for (a) 5-story building (b) 13-story building

In the Figure 5.21 and 5.22, the R values are provided for both Pile 1 (envelope of outer piles) and Pile 2 (envelope of inner piles) for 5- and 13-story building models. According to the results, soil nonlinearity leads to significant change in the pile top moments. As the general trend, soil nonlinearity decreases the moment at the pile-cap beam connection. Since the change in the moments due to soil nonlinearity does not show a common trend with respect to S_a values for both building models, mean and variability of the complete results are estimated. In Table 5.5, the mean and variability estimates for the $R_{\text{soil,NL}}$ values corresponding to pile top moments are presented. According to the arithmetic average of overall results, soil nonlinearity

reduces pile top moments by a factor of approximately 1.2 and 1.5 for the 5-story and 13-story building models, respectively, with an acceptable level of variability relative to the overall average values. However, when examining the variation of R values with increasing spectral acceleration (S_a) an initial increase is observed, followed by a decrease for certain ground motions. This suggests that, depending on the spectral acceleration, pile top moments may either increase or decrease, highlighting the need to account for this variability in the analysis.

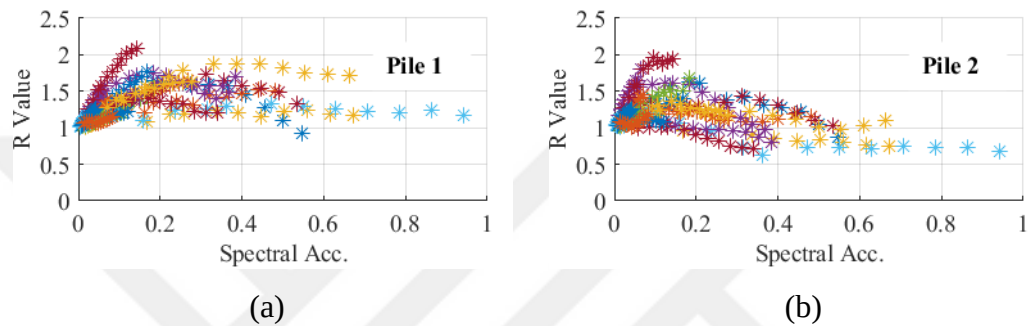


Figure 5.21. Effect of soil nonlinearity on pile top moment for 5-story building

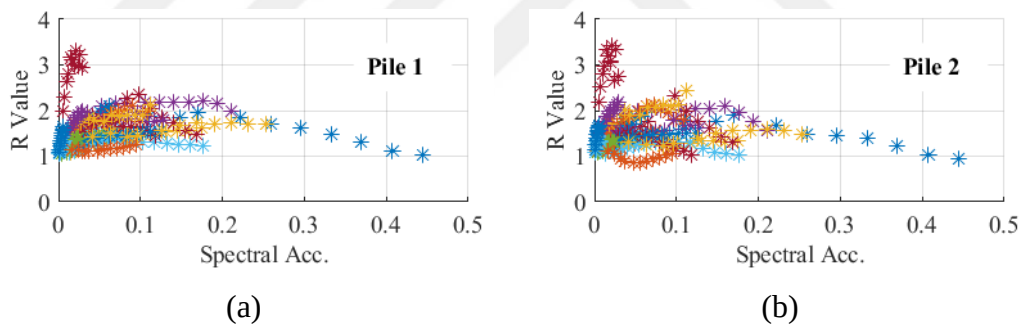


Figure 5.22. Effect soil nonlinearity on pile top moment for 13-story building

Table 5.5 Mean and variability estimates for pile top moments due to soil nonlinearity

5 story	Mean	COV	13 story	Mean	COV
Pile 1	1.33	0.17	Pile 1	1.59	0.25
Pile 2	1.18	0.19	Pile 2	1.54	0.27

5.3 Effect of Structure Nonlinearity

The effect of structural nonlinearity is evaluated by comparing the analysis results of Group 2 and Group 3 models. In both groups, the soil is modeled with nonlinear properties, while the structure is modeled with elastic properties in Group 2 and nonlinear properties in Group 3. The indicators $R_{str,NL}$ and $\mu_{str,NL}$ are the measures chosen for assessing the impact of structural nonlinearity.

$$R_{str,NL} \text{ or } \mu_{str,NL} = \frac{\text{Group 2 results}}{\text{Group 3 results}} \quad (5.8)$$

5.3.1 Base Shear

The ratio of the absolute maximum base shear values obtained from the Group 2 and Group 3 analyses is calculated and presented as a function of spectral acceleration (S_a) in Figure 5.23. The trend in the figures suggest that the structural nonlinearity effect on demand parameters exhibits greater consistency across building heights than the soil nonlinearity effect.

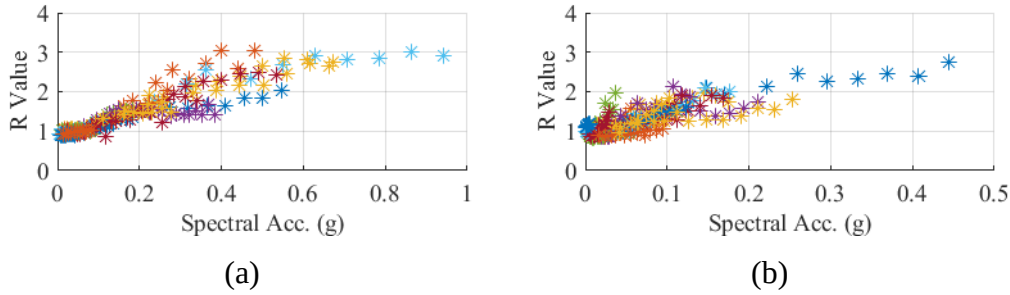


Figure 5.23. Effect of structure nonlinearity on base shear (a) 5-story building (b) 13-story building

The $R_{str,NL}$ estimates are provided in the Table A.7 and A.8 for 5- and 13-story buildings in Appendix. The data analysis is performed for specific S_a ranges to evaluate the mean and variations of the R estimations within the intervals of 0–0.1g, 0.1–0.2g, 0.5g, and 0.5–1g. Tables 5.6 and 5.7 provide the mean, standard deviation, and coefficient of variation of the R values for these S_a ranges. As the spectral

acceleration increases, the mean values of R estimates also increase, while maintaining reasonable coefficients of variation. However, a more definitive conclusion could be drawn if a larger dataset are available within each spectral acceleration range. The results suggest a potential correlation between the effect of structural nonlinearity and the spectral acceleration corresponding to the first mode of the structure.

Table 5.6 Mean, Std. Dev. and COV values of R estimates for 5 story building

Sa Range	Mean	Std. Dev	COV	# of data
0-0.1	1.00	0.07	0.07	139
0.1-0.2	1.37	0.26	0.19	49
0.2-0.5	1.90	0.44	0.23	60
0.5-1	2.61	0.33	0.13	16

Table 5.7 Mean, Std. Dev. and COV values of R estimates for 13 story building

Sa Range	Mean	Std. Dev	COV	# of data
0-0.1	1.11	0.21	0.02	212
0.1-0.2	1.67	0.24	0.14	41
0.2-0.5	2.12	0.38	0.18	11

After the separate evaluation of the results of the building models, considering the similar variability in each S_a range, both data sets are combined to develop a single equation for predicting the structural nonlinearity effect in presence of pile-soil nonlinearity with respect to spectral acceleration. Figure 5.24 illustrates the combined data sets along with a representative equation derived from this analysis.

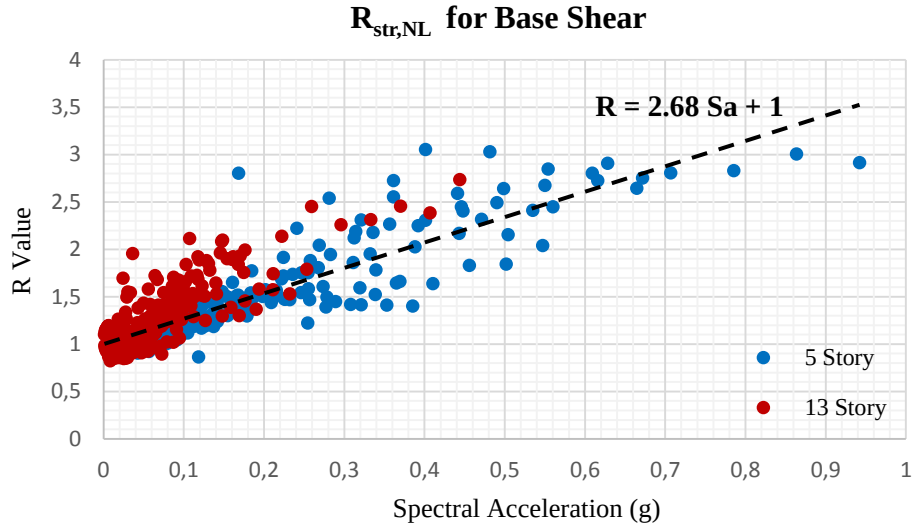


Figure 5.24. Effect of structure nonlinearity on base shear: relation with spectral accelerations

Further investigation may be warranted to validate the applicability of this combined model across a broader range of building configurations and seismic scenarios. However, R estimations for specific spectral acceleration ranges seems more feasible than combining whole data.

5.3.2 Drift Ratio

A similar analysis is conducted for the drift ratio data at the first story. The drift ratio results from the Group-2 analysis are divided by those from the Group-3 analysis. As the general trend, the nonlinearity in the structure along with the soil nonlinearity tends to decrease the first story drift ratio values in both structures as indicated in section 5.1.2. In Figure 5.25, the $\mu_{str,NL}$ factors are plotted against the corresponding S_a values. The numerical results for μ estimations are summarized in Appendix.

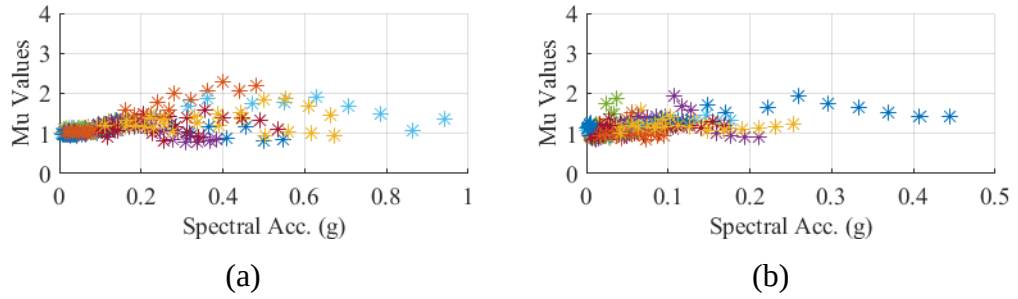


Figure 5.25. Effect of structure nonlinearity on drift ratio (a) 5-story building (b) 13-story building

A comparable data analysis is performed for designated spectral acceleration ranges to evaluate the mean and variability of μ estimates within the intervals of 0–0.1g, 0.1–0.2g, 0.2–0.5g, and 0.5–1g. Tables 5.8 and 5.9 display the mean, standard deviation, and coefficient of variation of μ values associated with various S_a ranges. The findings demonstrate that an increase in spectral acceleration correlates with a rise in the mean values of μ estimations, while preserving acceptable coefficients of variation. However, the variability in μ is greater compared to the corresponding base shear values, as drift values are influenced by the sequential yielding of the columns. For this reason, identifying a trend with respect to drift ratio is less straightforward compared to the base shear. A more conclusive determination may be attained if a more extensive dataset are accessible within each spectral acceleration band.

Table 5.8 Mean, Std. Dev and COV values of μ estimates for 5 story building

Sa Range	Mean	Std. Dev	COV	# of data
0-0.1	1.05	0.05	0.05	139
0.1-0.2	1.24	0.19	0.16	49
0.2-0.5	1.33	0.36	0.27	60
0.5-1	1.31	0.37	0.28	16

Table 5.9 Mean, Std. Dev and COV values of μ estimates for 13 story building

Sa Range	Mean	Std. Dev	COV	# of data
0-0.1	1.10	0.15	0.02	212
0.1-0.2	1.27	0.20	0.15	41
0.2-0.5	1.44	0.29	0.20	11

Given the observed consistency in variations across specific S_a ranges for both structural models, the data sets are combined to explore the potential for a unified predictive equation relating first-story drift ratio to spectral acceleration. Figure 5.26 presents a visual representation of the combined data sets along with the derived equation, providing insight into the relationship between structural nonlinearity along with the soil nonlinearity and spectral acceleration.

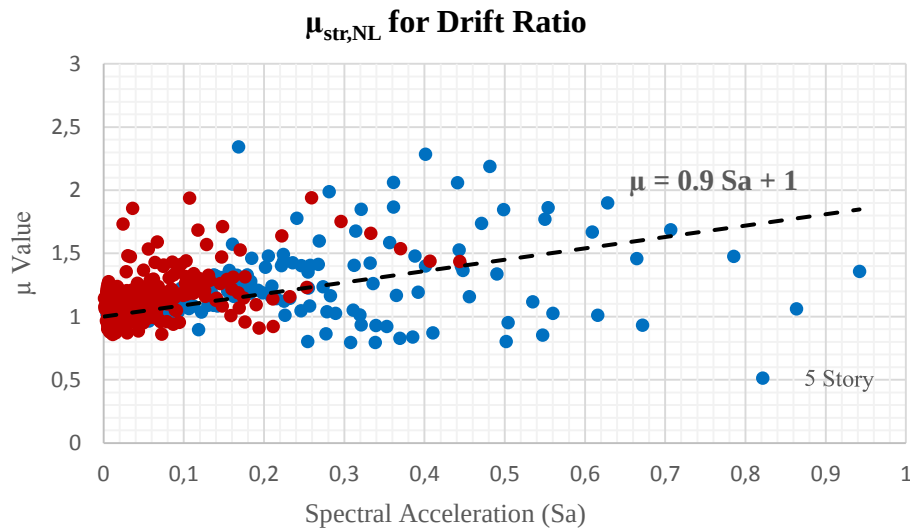


Figure 5.26. Effect of structure nonlinearity on 1st story drift ratio: relation with spectral accelerations

5.3.3 Pile Moment

The average moments throughout the pile length are employed to estimate the $R_{str,NL}$ factor for pile moments for each scale factor. The ratios of the average moments from Group 2 to those from Group 3 are estimated and plotted in Figure 5.27. As the maximum moments are present at both the upper and lower sections of the pile, it is prudent to take into account the R values derived from both sites. At the pile top joints, the average moment ratio approximates unity across all scale variables, signifying that structure nonlinearity exerts negligible influence on the pile top moments when the piles are represented with elastic characteristics.

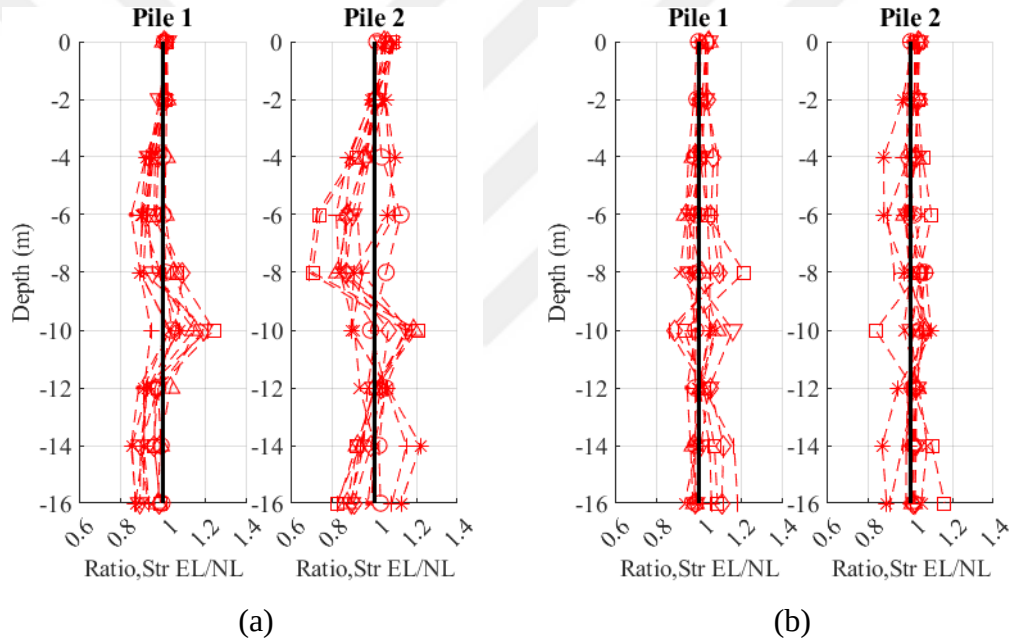


Figure 5.27. Pile moment ratio for structure nonlinearity comparison for (a) 5-story building (b) 13-story building

To evaluate the change in top moment values due to structural nonlinearity, the $R_{str,NL}$ values are plotted with respect to spectral acceleration in Figures 5.28 and 5.29 for 5- and 13-story buildings. These figures indicate that structural nonlinearity does not cause significant changes in pile top moments. To quantify this effect, the mean and variability of the $R_{str,NL}$ values are calculated and presented in Table 5.10. The mean values and low coefficient of variation shows that the nonlinear superstructure

produces results similar to those of the elastic superstructure. However, piles in the inner regions for the 5-story building are more affected by superstructure nonlinearity than those in the outer regions. The structure nonlinearity increased the dispersion on the pile top moments with respect to the spectral acceleration of the applied motion for the inner piles of 5-story building. For certain S_a values, moment values tend to decrease.

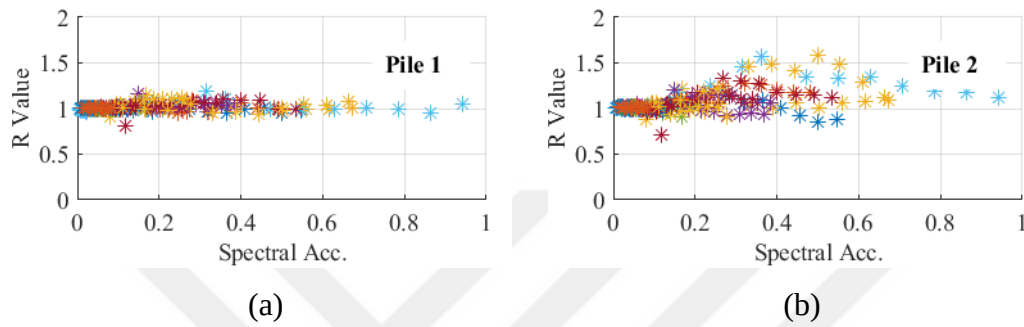


Figure 5.28. Effect of structure nonlinearity on pile top moment for 5-story building

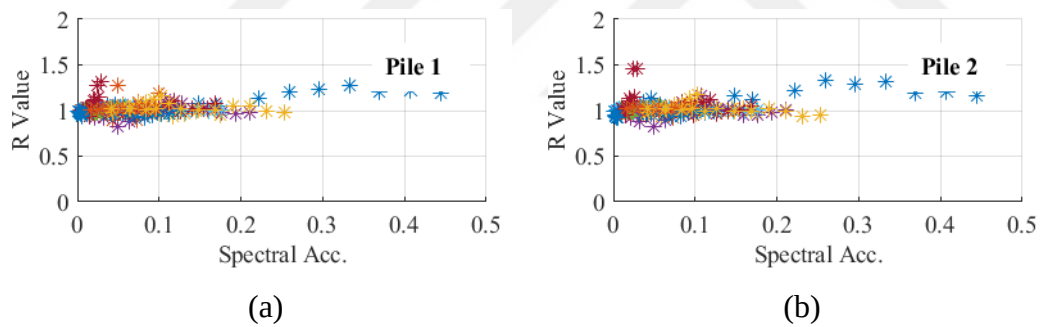


Figure 5.29. Effect of structure nonlinearity on pile top moment for 13-story building

Table 5.10 Mean and variability estimates for pile top moments due to structure nonlinearity

5 story	Mean	COV	13 story	Mean	COV
Pile 1	1.01	0.04	Pile 1	1.01	0.05
Pile 2	1.05	0.11	Pile 2	1.02	0.07

5.4 Effect of Cap Beam Rigidity

The rigidity of the cap beam in pile foundation structures influences the structural demands during dynamic analysis, particularly under seismic loading since the pile cap contributes to the lateral and rotational stiffness of the foundation (Norris, 1992). In order to evaluate the influence of cap beam stiffness on key demand parameters, the Group-2 analysis set is repeated with varying cap beam stiffness values. Specifically, the modulus of elasticity is adjusted to represent different stiffness conditions: $0.4E$ for a cracked cap beam, E for an un-cracked cap beam, and $10E$ for a rigid cap beam. The effect of different cap beam stiffness on system demands is evaluated below.

The drift ratio at the first story is illustrated in Figure 5.30 for both the 5-story and 13-story models. To further assess the impact of cap beam stiffness, the drift ratio obtained from the cracked cap beam model is compared to that from the rigid cap beam model, with the results presented in Figure 5.31. The results indicate that within the elastic range, corresponding to lower spectral acceleration (S_a) values, the cap beam stiffness effect is substantial on the drift ratio estimate, with ratios varying between 1.25 and 0.8 with even higher ratios at very low spectral accelerations. For higher accelerations, the effect disappears with ratio converging toward 1.0. For the 13 story building, cap beam stiffness leads to greater variability for lower spectral acceleration values.

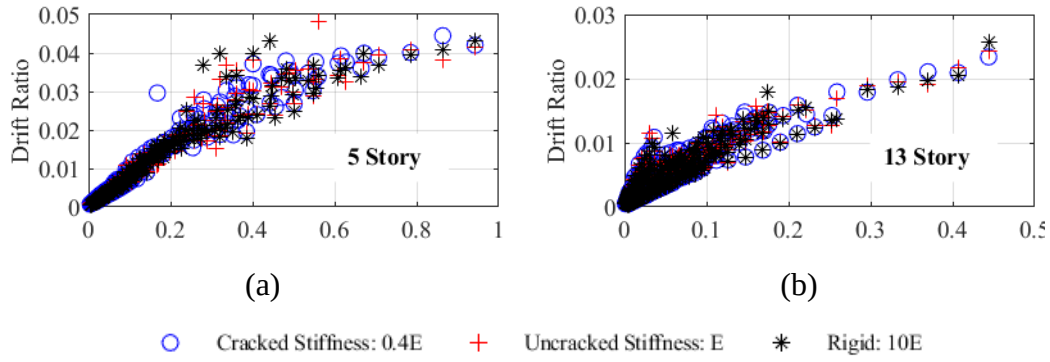


Figure 5.30. Drift ratio comparison for cap beam stiffness differences for (a) 5-story building (b) 13-story building

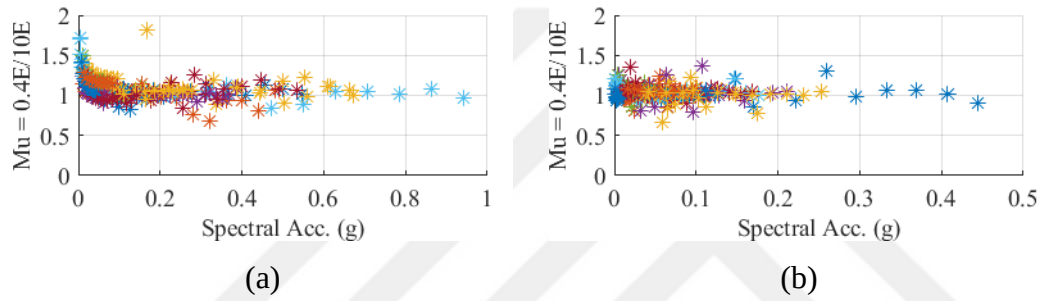


Figure 5.31. Effect of cap beam stiffness on first story drift ratio for (a) 5-story (b) 13-story buildings

In the Figure 5.32, the average absolute moments for each scale factor are plotted for both 5- and 13-story buildings to compare the effect of cap beam stiffness on the pile moments. The average of the absolute moments does not show a considerable variability along the pile length.

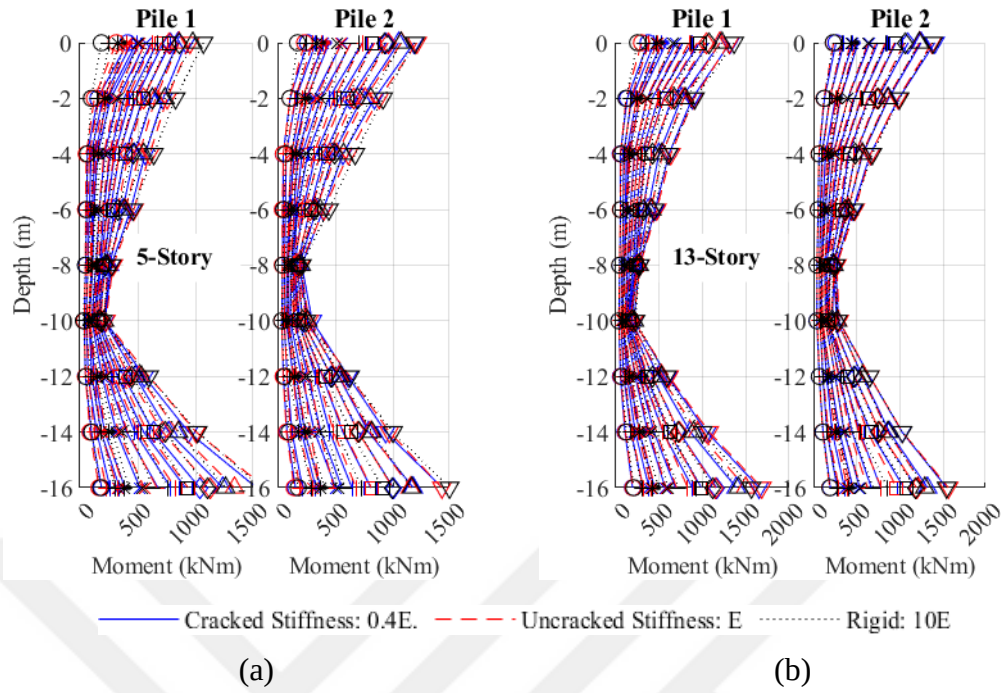


Figure 5.32. Average pile moment comparison for cap beam stiffness for (a) 5-story building (b) 13-story building

The absolute moments at the top of the pile are also plotted in the Figure 5.33 and 5.34. When the pile-head moments are individually shown for each analysis, it is noted that, for lower spectral acceleration values, the moment magnitudes are reduced in scenarios where a rigid cap beam is assumed. This trend is particularly evident in the five-story building model and for the outer piles. Such behavior can be attributed to the influence of the rotation of the foundation which is more obvious in the 5-story building. However after 0.2g spectral acceleration, the cap beam rigidity tends to increase the top moment for the outer pile of the 5-story building.

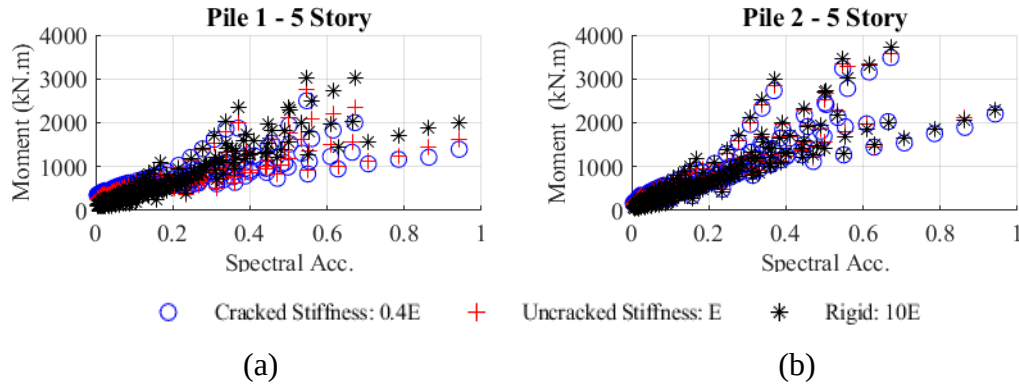


Figure 5.33. Top moments with respect to the spectral acceleration values for 5-story building – Cap beam stiffness comparison

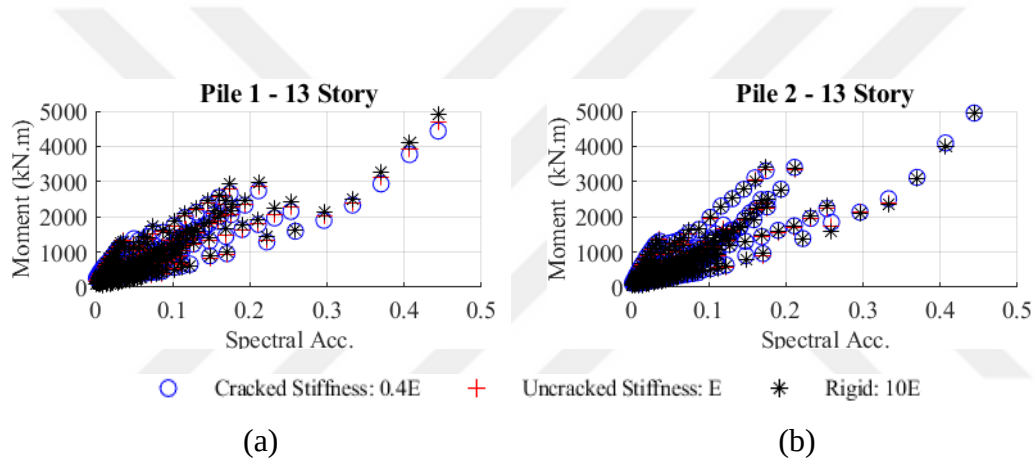


Figure 5.34. Top moments with respect to the spectral acceleration values for 13-story building – Cap beam stiffness comparison

In this study, the pile bases are modeled with vertical restraint, which restricts the influence of cap beam stiffness to a limited range of spectral acceleration (S_a) values and results in only a minor effect on moment magnitudes, except for the outer piles in the 5-story building. However, in the absence of vertical restraint at the pile base, cap beam stiffness would likely exert a more significant influence on both the drift ratio and pile-head moments. This effect is expected to be particularly pronounced in the 5-story building, where the removal of vertical restraints would amplify rocking behavior, thereby making the impact of cap beam rigidity more noticeable.

The results of the analysis are discussed in the next section.

5.5 Discussion of Results

The dynamic analyses of the soil, pile foundation and frame system show the influence of soil and structural nonlinearity on the response predictions highlighting the soil-structure interaction (SSI) effects on seismic response. Inertial interaction is evident through the observed reduction in base shear and drift ratios as soil nonlinearity is introduced. The flexibility and damping effects of the nonlinear soil medium dissipate seismic energy, reducing force transmission to the superstructure. As a result, base shear and drift ratio demands decrease with the inclusion of soil nonlinearity due to damping effects and flexibility of the system.

The inertial forces are transferred to the supporting soil through the foundation. This transfer can cause the soil to reach its nonlinear phase earlier, especially if the forces are substantial. When soil enters its nonlinear phase, it deforms more significantly. This deformation affects the structural response by altering the base shear characteristics and potentially changing the mode shapes of the structure. Those effects are observed more significantly in the 13-story building model because the taller structure generates larger inertial forces. The inertial forces are transferred to the supporting soil, causing it to reach the nonlinear phase earlier. As a result, the soil deforms more significantly, influencing the base shear response more prominently in the 13-story model than in the 5-story model. Similarly, a significant change in the mode shapes of the 13-story building model is observed with the inclusion of soil nonlinearity.

Kinematic interaction in pile foundations refers to the forces induced in piles as a result of soil movement during seismic events, as well as the alteration of incoming seismic motion due to the presence of the foundation and superstructure. These kinematic effects are particularly pronounced at the interface between soil layers with differing stiffness properties, where significant bending moments can develop (Grassi and Massimino, 2009).

Although this study considers a single-layer soil profile, the differences in pile moments observed between elastic (Group 1) and nonlinear soil (Group 2) analyses can be attributed to kinematic effects. Notably, the nonlinearity of the superstructure (Group 3) had minimal impact on pile moments, whereas soil nonlinearity played a more substantial role. When examining the distribution of these changes along the pile length, the most substantial variations were observed in the middle and near the pile tip, indicating that this region is particularly sensitive to kinematic effects.

Continuum modeling of soil, in conjunction with the analyzed structural system, is not commonly used in the literature. Most previous studies have employed finite element models of soil in conjunction with single piles, pile groups, or single-degree-of-freedom systems (Grassi and Massimino, 2009; Yu et al., 2022; Yigit et al., 2022). In some studies, rather than a direct analysis approach, a sub-structuring method has been utilized, incorporating the Beam on Winkler Foundation (BWF) method to analyze building structures with pile foundations. In one of the rare studies performing direct analysis of a structure on a continuum model, Jiménez and Dias (2022) examine the dynamic soil–structure interaction (SSI) effects on mid-rise buildings supported by pile foundations and rigid inclusion systems with a similar approach. Their research focuses on how different pile toe conditions (floating, placed, and anchored) influence the seismic response of structures, specifically three-, five-, and seven-story buildings. Their findings indicate that floating piles exhibit the highest rocking response due to the lack of a direct connection to a hard soil layer, allowing greater rotational movement. Rocking response was not observed in this work as well with piles encased in rigid stratum.

Due to differences in methodology and research focus, only a limited comparison can be made, as the present study is not directly comparable to these works.

CHAPTER 6

CONCLUSION

This study investigated the dynamic response of pile-supported building structures under seismic loading, focusing on the effects of soil-structure interaction (SSI), soil and structural nonlinearities, and cap beam stiffness. A comprehensive numerical modeling approach was employed to analyze the behavior of soil-pile-superstructure systems, incorporating elastic and nonlinear material properties for both soil and structure. The soil-pile-structure system has been modeled in three distinct ways: elastic soil with elastic structure, nonlinear soil with elastic structure, and nonlinear soil with nonlinear structure. Two building configurations — a 5-story and a 13-story structure—were analyzed under 24 ground motions scaled to 11 different intensity levels, resulting in a total of 2,640 dynamic analyses with the different modeling cases. The seismic loads have been calculated considering the approach described in the study of Lokke and Chopra (2018) and have been applied around the three-dimensional finite element model of the soil through transmitting boundaries.

Key engineering demand parameters (EDPs), including base shear, drift ratio, and pile moments, were evaluated to assess the influence of soil nonlinearity, structural nonlinearity with the existence of the soil nonlinearity and the cap beam stiffness. The ratio of the demand parameters between the modeling cases was found to quantify the soil and structure nonlinearity influences. $R_{\text{soil,NL}}$ and $R_{\text{str,NL}}$ parameters are used to define the ratio of forces and moments between the corresponding modeling cases, for soil nonlinearity and structure nonlinearity, respectively. For the drift ratio parameter, $\mu_{\text{soil,NL}}$ and $\mu_{\text{str,NL}}$ are defined as the ratio of related analysis cases, for soil and structural nonlinearity, correspondingly. These parameters are summarized as follows:

$R_{\text{soil,NL}}$	Ratio of Group 1 to Group 2 analysis for base shear and pile moments
$R_{\text{str,NL}}$	Ratio of Group 2 to Group 3 analysis for base shear and pile moments
$\mu_{\text{soil,NL}}$	Ratio of Group 1 to Group 2 analysis for drift ratio
$\mu_{\text{str,NL}}$	Ratio of Group 2 to Group 3 analysis for drift ratio

6.1 Concluding Remarks

The correlation of soil and structural nonlinearity parameters with the spectral acceleration at the fundamental mode of the structures is investigated to facilitate practical inferences regarding to the system behavior. However, the low acceleration amplitudes of the selected records, small scale factors, and high fundamental periods of the buildings have led to the concentration of S_a values at lower levels. Therefore, the extracted conclusions are based on the limited data set. Concluding remarks are explained as follows:

- The inclusion of nonlinear soil behavior significantly influences the seismic response of the system. Soil nonlinearity generally reduces base shear and drift ratios due to its damping effect. For the 5-story building, the reduction in drift ratios is more pronounced in the upper stories, while for the 13-story building, the effect is more uniform along the height.
- The estimates for soil nonlinearity effect ($R_{\text{soil,NL}}$, $\mu_{\text{soil,NL}}$) for base shear and first story drift ratio does not show an obvious correlation with the spectral acceleration, for the 13-story frame model. However in five-story frame a more uniform effect of the soil nonlinearity is observed. The mean and variability of $R_{\text{soil,NL}}$ and $\mu_{\text{soil,NL}}$ are summarized as shown in the Table 6.1. Similar mean and variability estimates are obtained for base shear and drift ratio within each building type (5-story and 13-story), but the results differ between the two.

Table 6.1 Mean, Std. Dev and COV values for soil nonlinearity effects

		Soil Nonlinearity		
		Mean	Std. Dev.	COV
5 story	Base Shear	1.12	0.19	0.17
	Drift Ratio	1.05	0.17	0.16
	Pile 1	1.33	0.23	0.17
	Pile 2	1.18	0.23	0.19
13 story	Base Shear	1.76	0.66	0.38
	Drift Ratio	1.61	0.55	0.34
	Pile 1	1.59	0.39	0.25
	Pile 2	1.54	0.42	0.27

- Soil nonlinearity significantly affects the pile top moments. In general, analysis with soil nonlinearity led to reduced prediction for pile moments compared to the case with elastic soil layer.
- The structural nonlinearity impacts on pile top moments is minimal compared to that of soil nonlinearity. The findings suggest that structural nonlinearity primarily influences superstructure demands rather than foundation-level responses.
- The rigidity of the cap beam influenced the drift ratios, leading to the variability in the predicted drift for the first story of considered buildings.

6.2 Future Studies

The dynamic analysis of pile foundation structures is a broad field of study. In this work, 2D frame structure models representing mid-rise reinforced concrete buildings, supported on 3D continuum with rigorous modeling of piles and soil nonlinearity, were used to assess the effects of various factors on the building-pile-soil interaction. Utilizing the modeling and the analysis procedure, several areas warrant further exploration:

- For the existing analyzed models, the data set for higher spectral acceleration values can be expanded by adding new ground motions and increasing the scale factors of the motions in order to assess the level of shaking on SSI.
- A single clay layer seating on hard soil (rock) was used the underlying soil layer for the considered buildings. Real site conditions are typically more challenging requiring work in multi-layer settings.
- The interaction of different component of motions with multiple piles setup under the buildings has not been investigated. Comparison of 2D models with three dimensional building structure/soil medium models is required to discern differences and assess the practicality of 2D models.

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APPENDICES

Selected Ground Motions

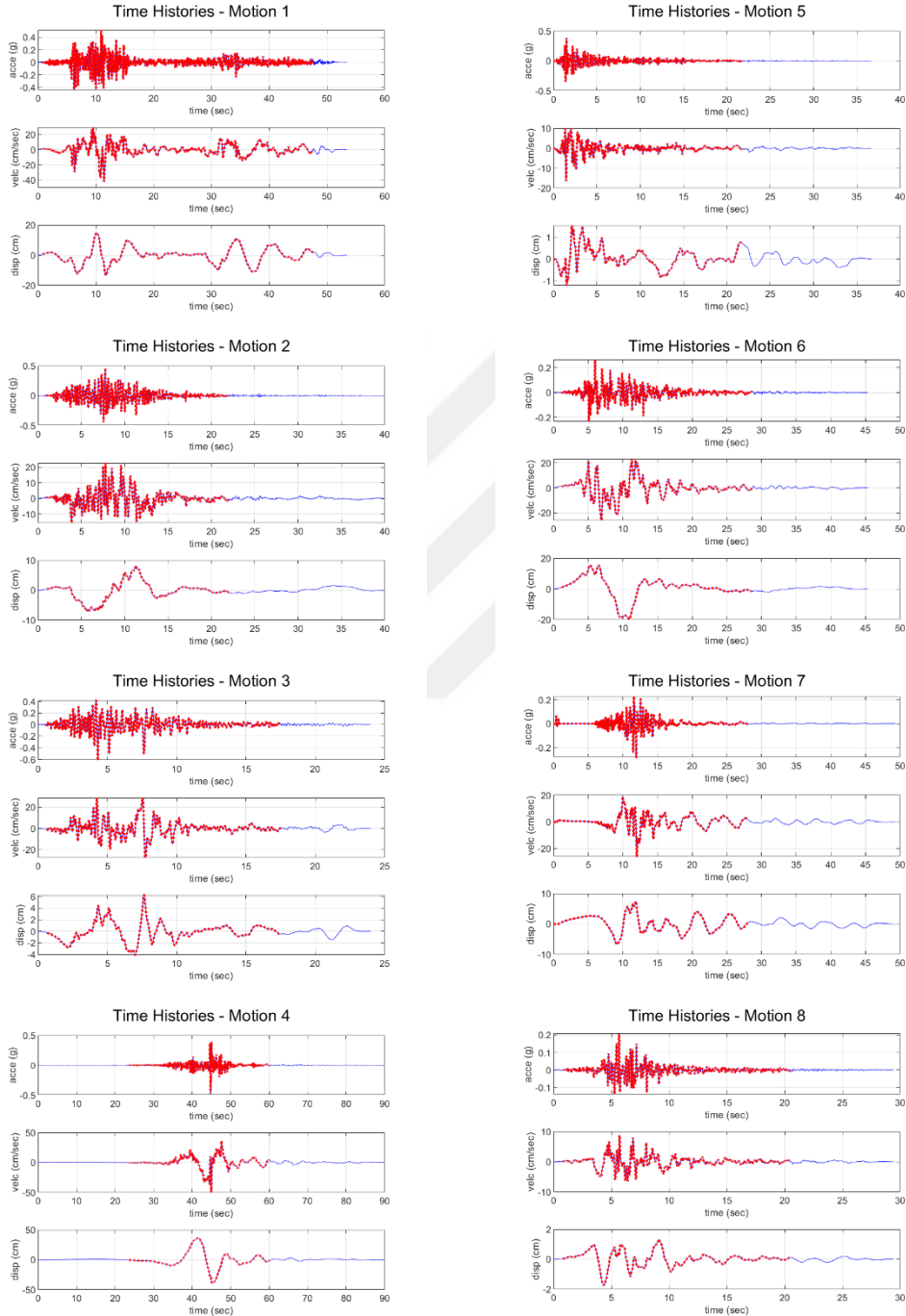


Figure A.1. Ground motion time histories for motions 1-8

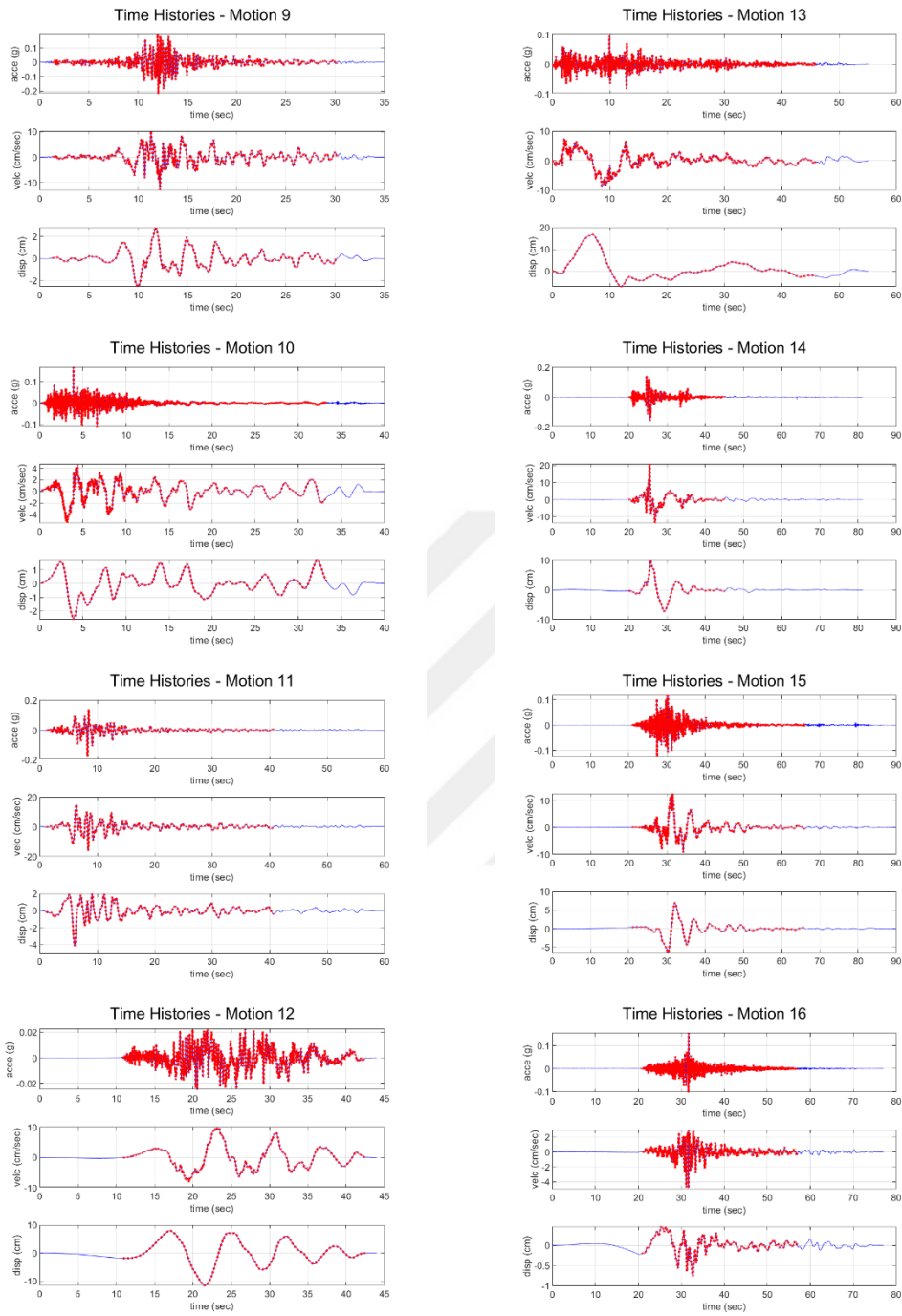


Figure A.2. Ground motion time histories for motions 9-16

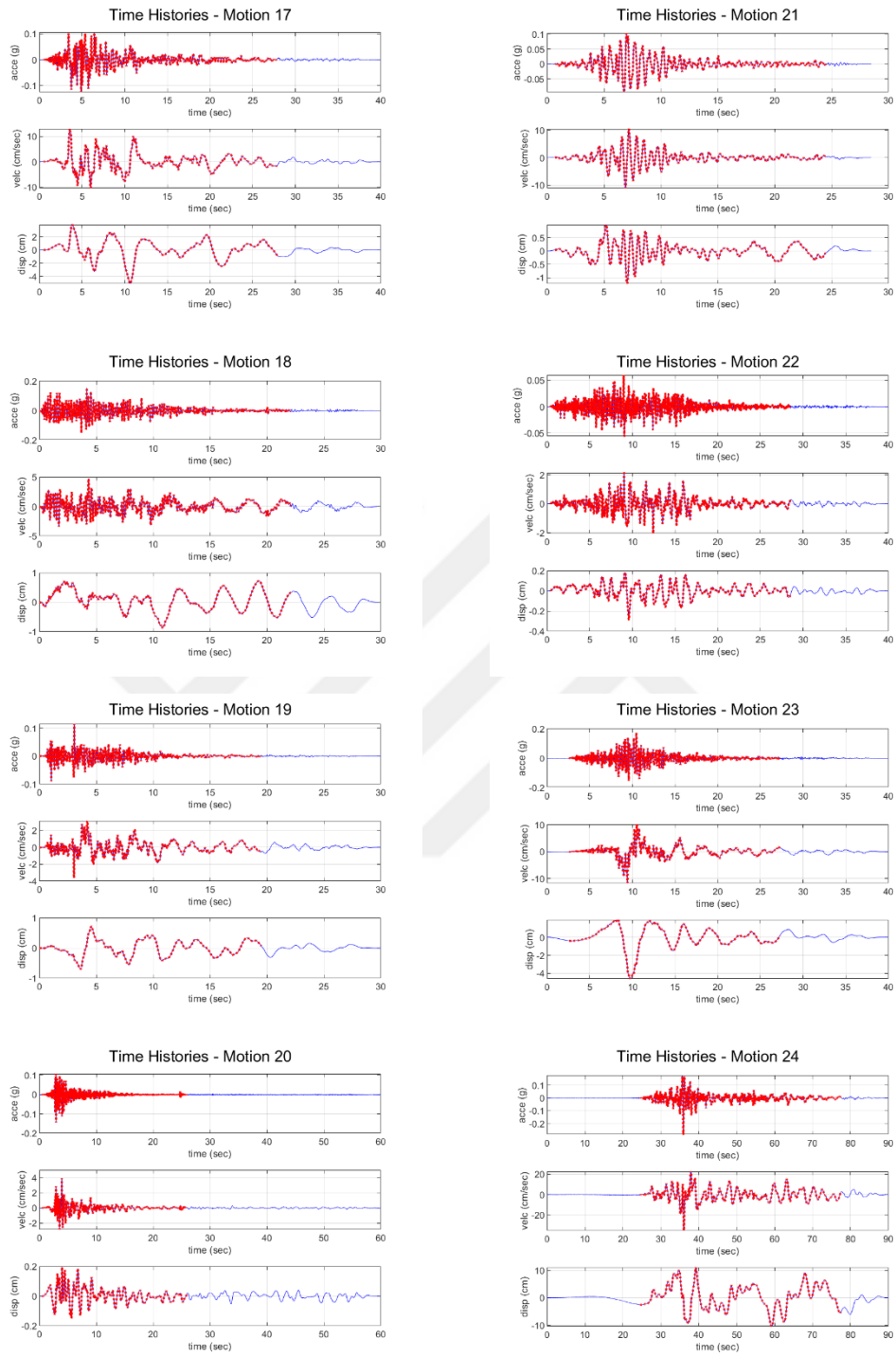


Figure A.3. Ground motion time histories for motions 17-24

Average Drift Ratio for 5- and 13- Story Models

Table A.1 Average drift ratio at each story – 5-story building

Case 1 : Elastic Soil - Elastic Structure - 5 Story Building											
	1	2	3	4	5	6	7	8	9	10	11
1	0.004	0.006	0.007	0.009	0.011	0.013	0.015	0.017	0.018	0.020	0.022
2	0.004	0.006	0.007	0.008	0.011	0.013	0.015	0.017	0.018	0.020	0.022
3	0.004	0.006	0.008	0.010	0.012	0.014	0.017	0.019	0.021	0.023	0.025
4	0.004	0.006	0.007	0.009	0.011	0.013	0.015	0.017	0.019	0.021	0.022
5	0.003	0.004	0.006	0.006	0.008	0.010	0.011	0.012	0.014	0.015	0.017
Case 2 : Nonlinear Soil - Elastic Structure - 5 Story Building											
	1	2	3	4	5	6	7	8	9	10	11
1	0.004	0.007	0.008	0.009	0.011	0.013	0.014	0.015	0.017	0.017	0.018
2	0.004	0.006	0.008	0.009	0.011	0.012	0.013	0.014	0.015	0.016	0.017
3	0.003	0.006	0.007	0.008	0.009	0.010	0.011	0.012	0.013	0.014	0.014
4	0.003	0.005	0.005	0.006	0.007	0.008	0.009	0.009	0.010	0.011	0.011
5	0.002	0.003	0.003	0.004	0.005	0.005	0.006	0.006	0.007	0.007	0.007
Case 3 : Nonlinear Soil - Nonlinear Structure - 5 Story Building											
	1	2	3	4	5	6	7	8	9	10	11
1	0.004	0.005	0.006	0.007	0.009	0.010	0.011	0.012	0.013	0.015	0.016
2	0.004	0.005	0.007	0.007	0.009	0.011	0.011	0.013	0.014	0.015	0.016
3	0.003	0.005	0.006	0.006	0.008	0.009	0.010	0.011	0.012	0.013	0.013
4	0.003	0.004	0.004	0.005	0.006	0.006	0.007	0.008	0.009	0.009	0.009
5	0.002	0.002	0.003	0.003	0.004	0.004	0.005	0.005	0.005	0.005	0.006

Table A.2 Average drift ratio at each story – 13-story building

Case 1 : Elastic Soil - Elastic Structure - 13 Story Building											
	1	2	3	4	5	6	7	8	9	10	11
1	0.003	0.004	0.006	0.007	0.009	0.010	0.011	0.013	0.014	0.016	0.017
2	0.003	0.004	0.005	0.006	0.008	0.009	0.011	0.012	0.013	0.014	0.016
3	0.002	0.004	0.005	0.006	0.007	0.008	0.010	0.011	0.012	0.013	0.014
4	0.003	0.004	0.005	0.006	0.008	0.009	0.010	0.012	0.013	0.014	0.015
5	0.004	0.006	0.008	0.009	0.011	0.013	0.015	0.017	0.019	0.021	0.023
6	0.004	0.006	0.009	0.010	0.013	0.015	0.017	0.019	0.021	0.024	0.026
7	0.004	0.006	0.008	0.009	0.011	0.013	0.015	0.017	0.019	0.021	0.023
8	0.003	0.004	0.006	0.007	0.009	0.010	0.012	0.013	0.015	0.016	0.018
9	0.003	0.004	0.006	0.007	0.008	0.010	0.011	0.013	0.014	0.016	0.017
10	0.004	0.007	0.009	0.010	0.013	0.015	0.017	0.020	0.022	0.024	0.026
11	0.005	0.008	0.011	0.013	0.016	0.019	0.022	0.025	0.027	0.030	0.033
12	0.005	0.008	0.010	0.012	0.016	0.018	0.021	0.024	0.026	0.029	0.031
13	0.004	0.005	0.007	0.008	0.011	0.013	0.014	0.016	0.018	0.020	0.022
Case 2 : Nonlinear Soil - Elastic Structure - 13 Story Building											
	1	2	3	4	5	6	7	8	9	10	11
1	0.002	0.003	0.004	0.004	0.005	0.006	0.006	0.007	0.008	0.008	0.009
2	0.002	0.003	0.004	0.004	0.006	0.006	0.007	0.007	0.008	0.009	0.009
3	0.002	0.003	0.004	0.004	0.005	0.006	0.007	0.007	0.008	0.008	0.009
4	0.002	0.003	0.004	0.004	0.005	0.005	0.006	0.007	0.008	0.008	0.009
5	0.002	0.003	0.004	0.004	0.005	0.006	0.007	0.008	0.009	0.009	0.010
6	0.002	0.003	0.004	0.004	0.006	0.007	0.007	0.008	0.009	0.010	0.011
7	0.002	0.003	0.004	0.004	0.006	0.007	0.008	0.009	0.010	0.010	0.011
8	0.002	0.003	0.004	0.005	0.006	0.007	0.008	0.009	0.010	0.010	0.011
9	0.003	0.004	0.006	0.006	0.008	0.009	0.010	0.011	0.012	0.012	0.013
10	0.003	0.005	0.006	0.007	0.009	0.009	0.010	0.011	0.012	0.013	0.013
11	0.003	0.005	0.006	0.006	0.008	0.008	0.009	0.010	0.011	0.011	0.012
12	0.003	0.004	0.005	0.005	0.006	0.007	0.008	0.008	0.009	0.009	0.009
13	0.002	0.003	0.003	0.003	0.004	0.004	0.005	0.005	0.006	0.006	0.006

Case 3 : Nonlinear Soil - Nonlinear Structure - 13 Story Building											
	1	2	3	4	5	6	7	8	9	10	11
1	0.002	0.003	0.003	0.004	0.004	0.005	0.005	0.006	0.006	0.007	0.007
2	0.002	0.003	0.004	0.004	0.005	0.006	0.006	0.007	0.008	0.008	0.009
3	0.002	0.003	0.004	0.004	0.005	0.006	0.006	0.007	0.008	0.009	0.010
4	0.002	0.003	0.003	0.004	0.005	0.005	0.006	0.007	0.008	0.009	0.010
5	0.002	0.003	0.004	0.004	0.005	0.006	0.007	0.007	0.008	0.009	0.010
6	0.002	0.003	0.004	0.004	0.005	0.006	0.007	0.008	0.008	0.009	0.010
7	0.002	0.003	0.004	0.004	0.005	0.006	0.007	0.008	0.008	0.009	0.009
8	0.002	0.003	0.004	0.005	0.006	0.006	0.007	0.008	0.008	0.009	0.009
9	0.003	0.004	0.005	0.006	0.007	0.007	0.008	0.008	0.008	0.009	0.009
10	0.003	0.004	0.005	0.006	0.007	0.007	0.008	0.008	0.008	0.008	0.009
11	0.003	0.004	0.005	0.005	0.006	0.006	0.007	0.007	0.007	0.007	0.007
12	0.003	0.003	0.004	0.004	0.005	0.005	0.005	0.005	0.006	0.006	0.006
13	0.002	0.002	0.003	0.003	0.003	0.003	0.004	0.004	0.004	0.004	0.004

First Story Drift Ratio Time Histories of 13-Story Building for the Original Motions

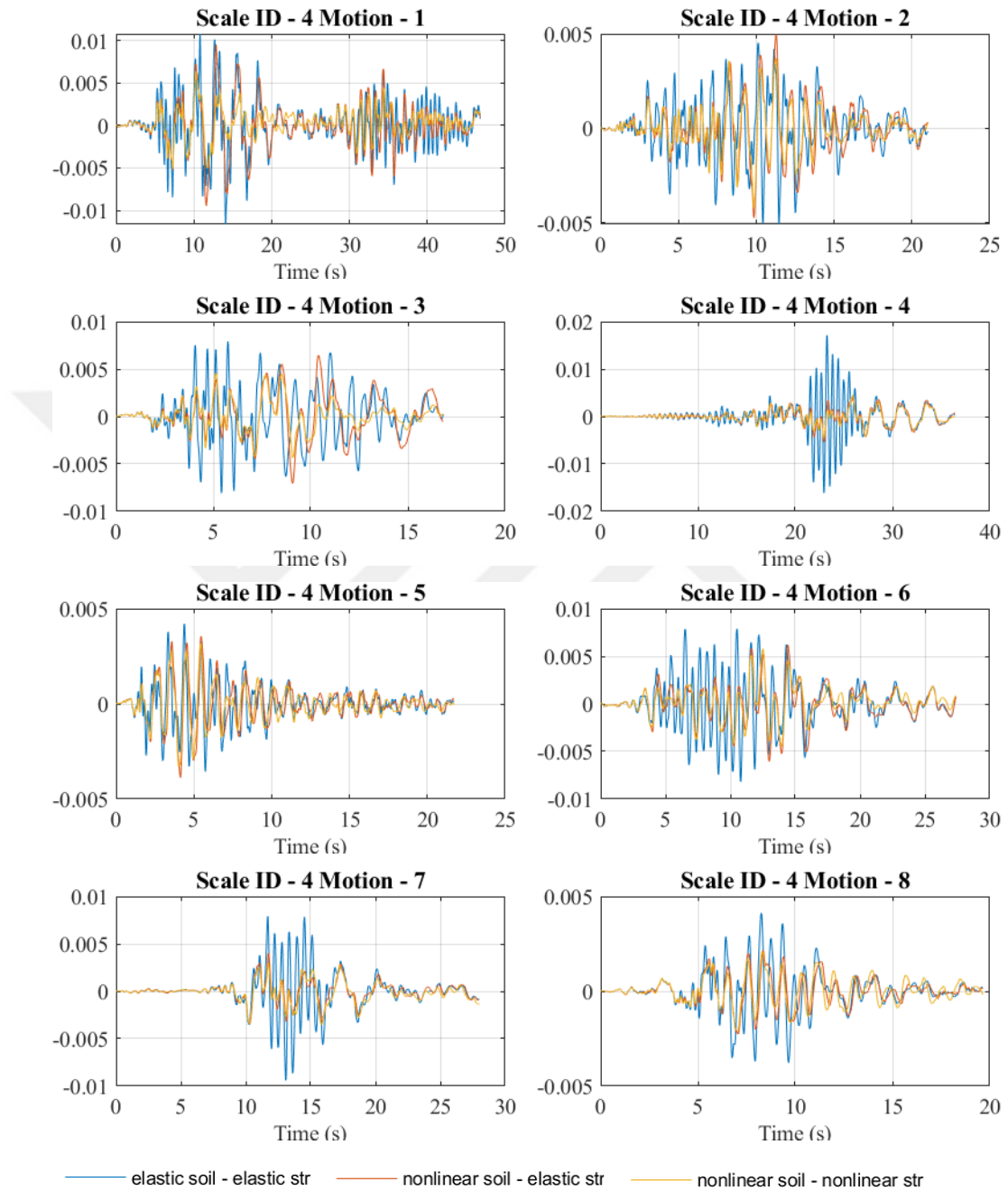


Figure A.4. Drift ratio time histories for the 1st story, for scale set-4, motions 1-8

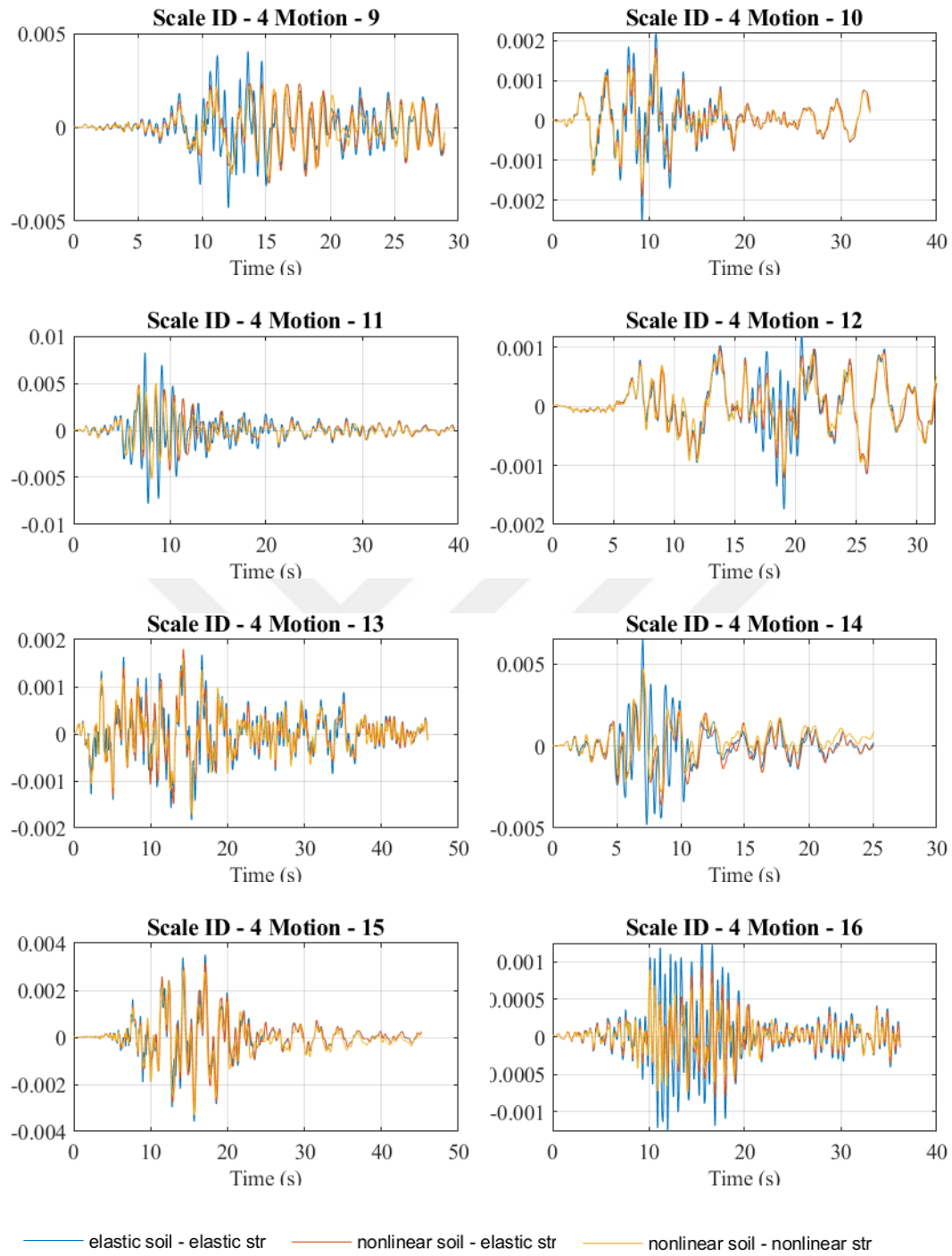


Figure A.5. Drift ratio time histories for the 1st story, for scale set-4, motions 9-16

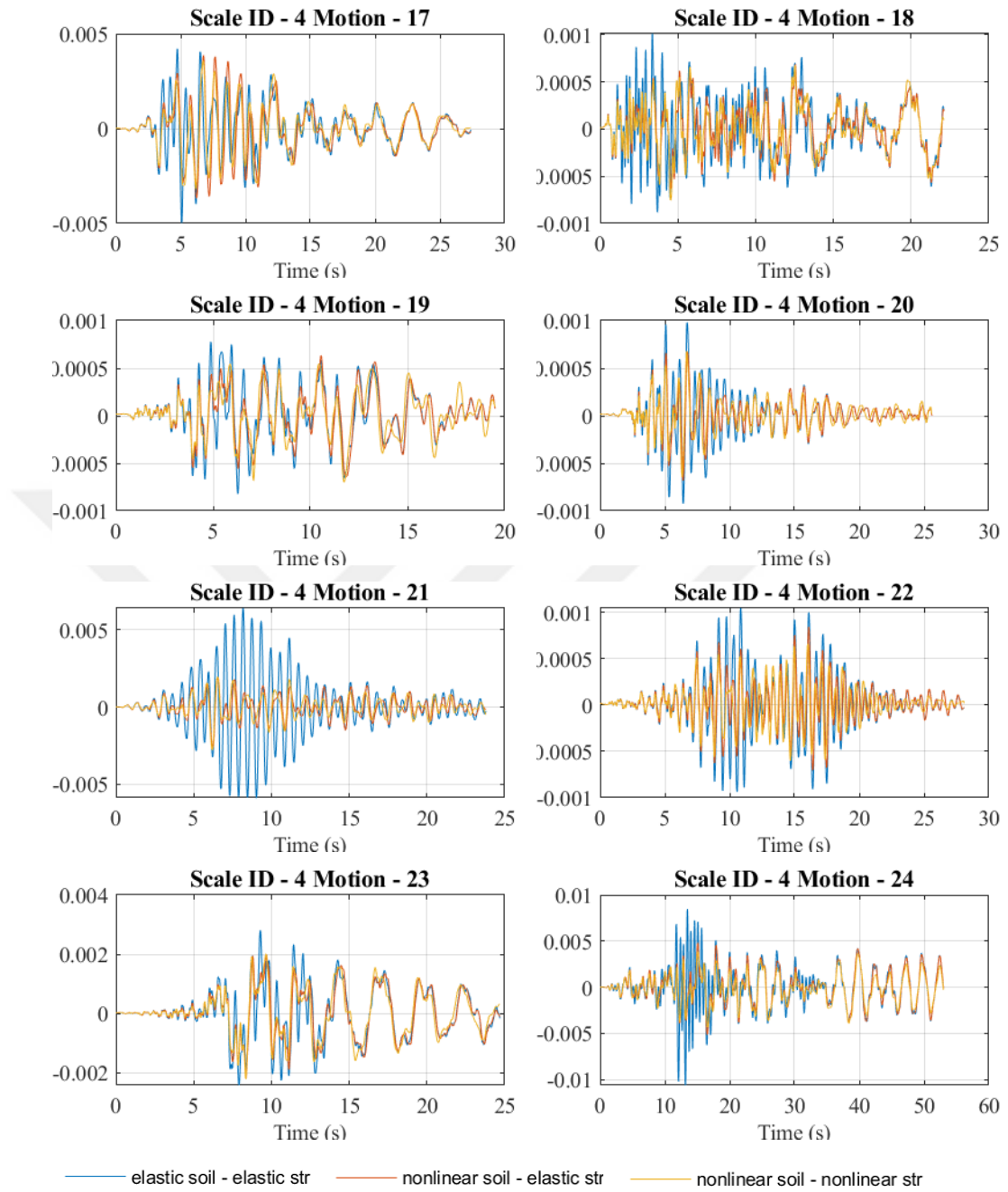


Figure A.6. Drift ratio time histories for the 1st story, for scale set-4, motions 17-24

R and μ Estimations

Table A.3 $R_{\text{soil,NL}}$ estimations for base shear parameter – 5 story building model

Motion	Scale ID										
	1	2	3	4	5	6	7	8	9	10	11
1	1.23	1.20	1.21	1.21	1.30	1.29	1.46	1.55	1.61	1.62	1.55
2	1.20	1.07	0.88	0.89	0.83	0.86	1.03	0.97	0.92	1.09	1.08
3	0.95	0.44	0.76	0.84	0.86	0.95	1.02	1.26	1.26	1.18	1.27
4	1.20	1.17	1.08	1.02	0.97	1.02	1.13	1.26	1.34	1.35	1.35
5	1.04	1.11	1.10	1.13	1.08	1.05	1.06	1.10	1.12	1.21	1.23
6	0.85	0.80	0.79	0.77	1.11	1.12	1.11	1.22	1.31	1.30	1.50
7	1.00	1.00	1.10	1.02	1.13	1.17	1.10	1.12	1.14	1.21	1.34
8	1.27	1.29	1.28	1.27	1.23	1.26	1.24	1.32	1.20	1.27	1.28
9	1.02	1.10	1.11	1.08	1.07	0.99	1.02	1.03	1.01	1.03	0.95
10	0.96	0.94	0.92	0.92	0.92	0.92	0.92	0.91	0.91	0.90	0.89
11	0.77	0.78	0.84	0.84	0.93	1.00	1.10	1.19	1.26	1.39	1.48
12	0.98	0.98	0.99	1.00	0.99	0.99	0.98	0.97	0.96	0.96	0.95
13	1.01	0.99	0.98	0.97	0.96	0.96	0.96	0.97	0.97	0.97	0.97
14	1.18	1.24	1.24	1.21	1.16	1.07	1.08	1.41	0.94	1.08	1.20
15	1.11	1.20	1.24	1.25	1.26	1.26	1.25	1.26	1.21	1.17	1.16
16	1.14	1.23	1.31	1.36	1.46	1.50	1.54	1.55	1.55	1.56	1.57
17	0.90	0.85	0.81	0.81	0.83	0.85	0.87	0.92	0.94	0.93	0.97
18	1.18	1.22	1.24	1.26	1.29	1.30	1.31	1.32	1.32	1.33	1.33
19	1.01	1.00	0.98	0.97	0.97	0.98	0.98	0.99	0.99	1.00	0.99
20	1.13	1.19	1.24	1.27	1.32	1.36	1.39	1.41	1.44	1.47	1.51
21	1.05	1.14	1.15	1.15	1.18	1.20	1.22	1.26	1.43	1.27	1.39
22	1.04	1.03	1.02	1.02	1.00	1.00	1.01	1.02	1.02	1.03	1.03
23	1.00	1.00	1.02	1.04	1.08	1.11	1.14	1.16	1.19	1.21	1.23
24	0.94	0.91	0.94	1.01	1.15	1.29	1.39	1.25	1.25	1.39	1.57

Table A.4 $R_{\text{soil,NL}}$ estimations for base shear parameter – 13 story building model

Motion	Scale ID										
	1	2	3	4	5	6	7	8	9	10	11
1	0.98	1.19	1.09	1.31	1.41	1.33	1.52	1.55	1.61	1.78	1.79
2	1.08	1.09	1.29	1.10	1.50	1.51	1.39	1.52	2.00	1.78	2.18
3	1.32	1.25	1.38	1.38	1.66	1.70	1.91	1.90	1.87	2.09	2.30
4	3.40	3.37	3.50	3.55	3.58	3.68	3.72	4.08	4.14	4.13	4.07
5	1.18	1.12	1.02	1.10	1.05	1.42	1.19	1.26	1.44	1.41	1.31
6	1.22	1.21	1.33	1.43	1.56	1.70	1.80	1.82	1.65	1.93	1.99
7	1.90	2.09	2.27	2.44	2.70	2.75	2.96	2.72	2.78	2.55	2.85
8	1.59	1.71	1.87	1.96	1.92	1.94	2.11	2.18	2.12	2.20	2.16
9	1.61	1.71	1.62	1.55	1.45	1.71	1.83	1.79	2.01	1.76	1.95
10	1.18	1.26	1.34	1.37	1.44	1.47	1.48	1.48	1.48	1.49	1.49
11	1.29	1.37	1.37	1.86	1.68	2.22	2.28	2.63	2.04	2.45	2.56
12	1.31	1.42	1.49	1.52	1.58	1.60	1.62	1.63	1.64	1.65	1.67
13	1.09	1.09	1.07	1.07	1.08	1.08	1.09	1.10	1.11	1.12	1.14
14	1.23	1.36	1.49	1.59	1.65	1.80	1.67	1.90	1.79	1.78	1.79
15	1.10	1.11	1.12	1.12	1.13	1.15	1.16	1.18	1.24	1.23	1.22
16	1.33	1.43	1.50	1.54	1.59	1.60	1.61	1.61	1.60	1.58	1.57
17	1.53	1.44	1.41	1.40	1.61	1.83	1.78	1.78	1.79	1.76	2.35
18	1.48	1.50	1.57	1.64	1.64	1.63	1.63	1.59	1.60	1.58	1.57
19	1.24	1.37	1.46	1.45	1.44	1.42	1.41	1.40	1.38	1.36	1.35
20	1.23	1.33	1.40	1.40	1.42	1.43	1.44	1.45	1.46	1.43	1.50
21	2.51	2.74	2.78	2.78	3.08	3.05	3.17	3.35	3.82	3.59	3.21
22	1.10	1.18	1.25	1.28	1.33	1.35	1.38	1.38	1.39	1.40	1.40
23	1.38	1.33	1.28	1.27	1.35	1.39	1.45	1.51	1.53	1.61	1.60
24	1.89	1.84	2.12	2.28	2.58	2.96	2.98	3.00	2.90	2.92	2.79

Table A.5 $\mu_{\text{soil,NL}}$ estimations for drift ratio parameter – 5 story building model

Motion	Scale ID										
	1	2	3	4	5	6	7	8	9	10	11
1	1.21	1.15	1.17	1.17	1.28	1.26	1.42	1.50	1.53	1.64	1.58
2	0.98	0.93	0.77	0.82	0.75	0.76	0.89	0.85	0.82	0.97	0.96
3	0.90	0.42	0.71	0.79	0.81	0.90	0.97	1.20	1.20	1.14	1.22
4	1.21	1.16	1.07	1.01	0.93	0.98	1.08	1.21	1.29	1.30	1.30
5	0.92	1.00	0.89	0.87	0.82	0.79	0.82	0.85	0.86	0.93	0.95
6	0.77	0.73	0.76	0.74	1.05	1.07	1.06	1.16	1.25	1.24	1.43
7	1.01	1.01	1.10	1.00	1.10	1.14	1.08	1.11	1.13	1.20	1.34
8	1.19	1.17	1.15	1.14	1.10	1.14	1.15	1.19	1.12	1.18	1.18
9	0.98	0.99	0.98	0.96	0.95	0.87	0.90	0.91	0.89	0.90	0.86
10	0.96	0.94	0.92	0.92	0.92	0.91	0.91	0.90	0.89	0.88	0.87
11	0.74	0.75	0.82	0.84	0.95	1.02	1.13	1.22	1.30	1.44	1.51
12	0.98	0.97	0.96	0.96	0.96	0.96	0.96	0.95	0.95	0.95	0.94
13	0.98	0.98	0.97	0.97	0.97	0.97	0.96	0.94	0.93	0.92	0.91
14	1.02	1.12	1.13	1.12	1.11	1.07	1.09	1.41	0.95	1.09	1.21
15	1.02	1.08	1.12	1.13	1.10	1.07	1.05	1.05	1.04	1.01	1.01
16	0.94	0.95	0.96	0.98	1.00	1.02	1.03	1.05	1.06	1.07	1.08
17	0.92	0.85	0.81	0.81	0.83	0.86	0.88	0.92	0.93	0.94	0.97
18	1.10	1.14	1.17	1.19	1.21	1.21	1.22	1.22	1.22	1.22	1.22
19	0.97	0.98	0.98	0.97	0.96	0.96	0.95	0.94	0.93	0.92	0.91
20	1.06	1.09	1.12	1.13	1.17	1.20	1.22	1.24	1.25	1.27	1.30
21	0.99	1.20	1.19	1.19	1.20	1.22	1.21	1.23	1.37	1.22	1.34
22	0.97	0.96	0.95	0.95	0.93	0.93	0.94	0.94	0.95	0.95	0.95
23	0.98	0.99	1.01	1.02	1.06	1.09	1.11	1.14	1.16	1.17	1.19
24	0.95	0.91	0.92	0.99	1.12	1.27	1.35	1.21	1.21	1.34	1.52

Table A.6 $\mu_{\text{soil,NL}}$ estimations for drift ratio parameter – 13 story building model

Motion	Scale ID										
	1	2	3	4	5	6	7	8	9	10	11
1	0.96	1.15	1.01	1.22	1.30	1.23	1.40	1.43	1.49	1.65	1.61
2	1.02	1.01	1.23	1.06	1.41	1.44	1.36	1.48	1.97	1.77	2.02
3	1.12	1.04	1.15	1.14	1.38	1.42	1.59	1.57	1.55	1.74	1.90
4	2.93	2.96	3.08	3.13	3.16	3.23	3.30	3.55	3.57	3.54	3.47
5	1.13	1.11	0.96	1.04	1.00	1.35	1.14	1.21	1.37	1.34	1.24
6	1.13	1.12	1.24	1.35	1.48	1.57	1.65	1.66	1.50	1.76	1.80
7	1.92	2.08	2.23	2.39	2.40	2.45	2.53	2.47	2.51	2.28	2.54
8	1.41	1.55	1.71	1.72	1.72	1.77	1.93	2.00	2.04	2.12	2.08
9	1.57	1.59	1.48	1.42	1.33	1.56	1.66	1.73	1.82	1.66	1.76
10	1.15	1.23	1.29	1.32	1.37	1.41	1.43	1.45	1.45	1.45	1.45
11	1.12	1.21	1.22	1.64	1.49	1.96	2.01	2.30	1.80	2.15	2.25
12	1.24	1.33	1.38	1.41	1.45	1.47	1.48	1.50	1.51	1.52	1.54
13	1.06	1.10	1.14	1.12	1.10	1.10	1.10	1.11	1.11	1.12	1.13
14	1.12	1.24	1.36	1.45	1.50	1.62	1.51	1.69	1.60	1.59	1.59
15	1.07	1.08	1.08	1.09	1.09	1.10	1.10	1.14	1.22	1.20	1.19
16	1.24	1.37	1.45	1.50	1.58	1.59	1.56	1.53	1.50	1.47	1.45
17	1.38	1.29	1.32	1.34	1.55	1.77	1.70	1.69	1.68	1.65	2.23
18	1.27	1.28	1.29	1.33	1.40	1.35	1.34	1.32	1.34	1.32	1.33
19	1.17	1.26	1.25	1.25	1.24	1.23	1.22	1.21	1.20	1.18	1.17
20	1.21	1.26	1.29	1.30	1.33	1.34	1.35	1.36	1.38	1.34	1.41
21	2.10	2.17	2.24	2.27	2.54	2.53	2.61	2.79	3.17	3.25	2.99
22	1.07	1.17	1.25	1.28	1.34	1.35	1.36	1.36	1.36	1.37	1.37
23	1.22	1.19	1.18	1.19	1.28	1.33	1.38	1.45	1.47	1.52	1.49
24	1.66	1.65	1.90	2.03	2.34	2.58	2.60	2.55	2.51	2.53	2.42

Table A.7 $R_{\text{str,NL}}$ estimations for base shear parameter – 5 story building model

Motion	Scale ID										
	1	2	3	4	5	6	7	8	9	10	11
1	1.04	1.18	1.36	1.47	1.61	1.59	1.64	1.64	1.83	1.84	2.04
2	1.04	1.30	1.65	1.77	2.22	2.54	2.31	2.72	3.05	2.59	3.03
3	1.23	2.80	1.92	1.88	2.18	2.25	2.40	2.15	2.45	2.73	2.75
4	0.99	1.12	1.30	1.44	1.53	1.57	1.54	1.39	1.42	1.52	1.66
5	1.02	0.97	1.00	1.04	1.22	1.27	1.41	1.48	1.51	1.51	1.54
6	1.37	1.74	2.19	2.55	2.32	2.67	2.91	2.81	2.83	3.00	2.92
7	1.08	1.31	1.30	1.51	1.81	2.12	2.26	2.31	2.45	2.49	2.41
8	0.91	0.90	0.92	0.97	1.16	1.19	1.23	1.19	1.34	1.30	1.33
9	0.95	0.96	1.06	1.13	1.21	1.49	1.49	1.57	1.72	1.75	2.04
10	1.07	1.07	1.07	1.07	1.07	1.07	1.07	1.09	1.10	1.14	1.16
11	1.11	1.28	1.44	1.55	1.54	1.48	1.47	1.45	1.41	1.41	1.40
12	0.97	0.97	0.96	0.96	0.98	0.98	0.99	1.01	1.05	1.09	1.12
13	0.97	0.98	0.99	0.99	1.01	1.01	1.03	1.05	1.08	1.10	1.11
14	1.01	1.12	1.21	1.24	1.35	1.50	1.47	1.22	1.95	1.86	1.78
15	0.97	0.99	0.99	0.99	1.00	1.03	1.07	1.08	1.14	1.18	1.17
16	1.01	1.01	1.01	1.01	0.99	0.98	0.98	0.99	0.99	0.98	0.98
17	1.01	1.06	1.17	1.25	1.35	1.41	1.48	1.44	1.47	1.58	1.49
18	0.92	0.94	0.94	0.95	0.95	0.96	0.97	0.97	0.98	0.98	0.99
19	1.01	0.99	1.00	1.00	1.01	1.01	1.01	1.01	1.01	1.01	1.02
20	0.93	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
21	1.01	0.92	0.93	0.93	1.01	1.09	1.18	1.19	0.87	1.25	1.24
22	0.88	0.89	0.90	0.90	0.91	0.91	0.92	0.92	0.93	0.93	0.93
23	0.92	0.94	0.96	0.96	0.98	0.98	0.99	1.00	1.00	1.02	1.03
24	1.33	1.51	1.69	1.75	1.95	2.02	2.17	2.64	2.85	2.80	2.65

Table A.8 $R_{str,NL}$ estimations for base shear parameter – 13 story building model

Motion	Scale ID										
	1	2	3	4	5	6	7	8	9	10	11
1	1.41	1.53	2.10	1.93	2.14	2.45	2.26	2.31	2.46	2.38	2.74
2	1.11	1.08	1.04	1.12	1.08	1.00	1.43	1.58	1.18	1.41	1.34
3	0.90	1.21	1.42	1.68	1.71	1.66	1.73	1.85	1.96	1.90	1.75
4	1.05	1.10	1.20	1.26	1.41	1.51	1.53	1.39	1.45	1.58	1.74
5	0.89	1.04	1.16	1.16	1.26	1.05	1.70	1.34	1.55	1.54	1.96
6	0.97	1.14	1.30	1.33	1.47	1.53	1.69	1.78	2.08	1.92	1.99
7	1.27	1.22	1.22	1.22	1.32	1.30	1.29	1.51	1.64	1.90	1.84
8	1.00	0.98	0.97	0.99	1.01	1.14	1.05	1.14	1.27	1.26	1.33
9	1.02	1.00	1.05	1.04	1.22	1.11	0.90	1.26	1.15	1.39	1.40
10	1.16	1.15	1.13	1.13	1.12	1.09	1.08	1.12	1.19	1.24	1.30
11	0.92	1.13	1.38	1.04	1.72	1.54	1.61	1.52	2.11	1.92	1.88
12	1.09	1.05	1.02	1.01	0.99	0.99	0.98	0.98	0.98	0.98	0.98
13	0.92	0.94	0.96	0.97	0.99	1.00	1.03	1.11	1.18	1.21	1.24
14	0.91	0.97	1.04	1.06	1.19	1.19	1.41	1.36	1.59	1.72	1.89
15	1.00	0.98	0.96	0.98	1.09	1.22	1.37	1.47	1.51	1.51	1.61
16	0.92	0.97	1.01	1.00	0.98	0.98	0.99	1.00	1.02	1.05	1.07
17	1.08	1.08	1.13	1.06	1.20	1.23	1.40	1.51	1.67	1.84	1.50
18	0.92	0.91	0.88	0.86	0.88	0.86	0.86	0.86	0.85	0.85	0.86
19	0.94	0.87	0.82	0.84	0.85	0.85	0.86	0.86	0.87	0.88	0.89
20	0.99	0.96	0.94	0.96	0.99	1.00	1.01	1.02	1.02	1.06	1.03
21	0.91	0.87	0.88	0.90	0.93	1.16	1.07	1.07	1.15	1.24	1.50
22	1.10	1.13	1.15	1.15	1.16	1.17	1.17	1.18	1.19	1.19	1.19
23	0.88	0.88	0.90	0.91	0.91	0.93	0.96	0.98	1.02	1.02	1.06
24	0.95	1.21	1.26	1.26	1.25	1.30	1.30	1.37	1.58	1.53	1.79

Table A.9 $\mu_{\text{str,NL}}$ estimations for drift ratio parameter – 5 story building model

Motion	Scale ID										
	1	2	3	4	5	6	7	8	9	10	11
1	1.02	1.09	1.18	1.24	1.23	1.01	1.17	0.87	1.16	0.80	0.85
2	1.08	1.29	1.57	1.46	1.78	1.99	1.85	2.06	2.28	2.06	2.19
3	1.20	2.34	1.49	1.41	1.26	1.19	1.36	0.95	1.03	1.01	0.93
4	1.01	1.10	1.21	1.29	1.23	1.14	1.05	0.86	0.80	0.79	0.83
5	1.08	0.96	1.13	1.15	1.20	1.24	1.29	1.33	1.30	1.30	1.28
6	1.25	1.42	1.68	1.87	1.74	1.77	1.90	1.69	1.48	1.06	1.36
7	1.04	1.20	1.27	1.48	1.41	1.41	1.59	1.40	1.40	1.34	1.12
8	0.94	0.95	0.96	0.99	1.14	1.19	1.23	1.20	1.32	1.20	1.23
9	1.01	1.02	1.03	1.09	1.10	1.37	1.33	1.39	1.44	1.40	1.60
10	1.13	1.14	1.15	1.14	1.13	1.12	1.11	1.11	1.08	1.12	1.12
11	1.13	1.22	1.29	1.31	1.21	1.12	1.08	1.03	0.93	0.92	0.84
12	1.05	1.05	1.05	1.05	1.04	1.04	1.03	1.03	1.05	1.08	1.11
13	1.10	1.12	1.14	1.15	1.13	1.09	1.09	1.09	1.08	1.06	1.04
14	1.05	1.13	1.22	1.20	1.18	1.19	1.01	0.80	1.16	1.05	0.93
15	1.04	1.05	1.07	1.07	1.09	1.09	1.12	1.10	1.09	1.08	1.04
16	1.02	1.00	1.03	1.03	1.02	1.03	1.02	1.03	1.05	1.09	1.13
17	1.08	1.13	1.17	1.17	1.15	1.16	1.20	1.16	1.14	1.23	1.04
18	0.98	0.97	0.98	0.99	1.00	1.00	1.00	1.02	1.04	1.05	1.05
19	1.11	1.07	1.07	1.06	1.06	1.07	1.08	1.09	1.10	1.11	1.11
20	1.02	1.01	1.01	1.01	1.00	1.00	1.00	1.00	0.99	0.99	0.99
21	1.07	1.04	1.04	1.03	1.04	1.05	1.07	1.06	0.90	1.09	1.08
22	0.98	0.98	0.98	0.99	0.99	0.99	0.99	0.99	0.99	0.99	0.99
23	1.03	1.04	1.05	1.05	1.05	1.05	1.05	1.05	1.04	1.05	1.04
24	1.20	1.33	1.40	1.35	1.42	1.48	1.53	1.85	1.86	1.67	1.46

Table A.10 $\mu_{\text{str,NL}}$ estimations for drift ratio parameter – 13 story building model

Motion	Scale ID										
	1	2	3	4	5	6	7	8	9	10	11
1	1.16	1.23	1.71	1.53	1.64	1.94	1.75	1.66	1.54	1.44	1.43
2	1.19	1.13	1.12	1.13	1.06	0.89	1.37	1.54	1.05	1.24	1.40
3	0.87	1.12	1.29	1.59	1.33	1.26	1.30	1.33	1.26	1.22	1.15
4	1.14	1.13	1.18	1.18	1.18	1.20	1.14	1.01	0.96	0.91	0.92
5	0.99	1.01	1.23	1.17	1.16	0.95	1.73	1.12	1.48	1.47	1.86
6	1.05	1.03	1.11	1.07	1.15	1.18	1.27	1.29	1.47	1.31	1.31
7	1.36	1.26	1.20	1.09	1.16	1.18	1.18	1.21	1.13	1.29	1.19
8	1.04	1.04	1.04	1.08	1.08	1.21	1.07	1.21	1.24	1.24	1.30
9	1.10	1.07	1.13	1.11	1.22	1.06	0.86	1.15	1.05	1.19	1.27
10	1.22	1.20	1.19	1.18	1.17	1.16	1.16	1.16	1.19	1.19	1.20
11	0.94	1.18	1.24	0.95	1.43	1.41	1.43	1.41	1.94	1.68	1.57
12	1.14	1.11	1.09	1.08	1.06	1.05	1.05	1.04	1.04	1.04	1.03
13	0.99	0.95	0.93	0.95	0.98	0.99	1.02	1.09	1.16	1.16	1.17
14	0.97	0.97	0.96	0.95	1.01	0.97	1.10	1.05	1.17	1.21	1.30
15	1.07	1.04	1.01	0.99	1.03	1.13	1.31	1.40	1.32	1.34	1.29
16	0.97	1.00	1.07	1.08	1.00	1.00	1.03	1.06	1.09	1.12	1.15
17	1.10	1.13	1.18	1.08	1.19	1.03	1.16	1.22	1.35	1.44	1.20
18	0.90	0.88	0.87	0.86	0.87	0.95	1.03	1.06	1.02	1.02	1.01
19	0.96	0.90	0.92	0.92	0.94	0.94	0.94	0.94	0.94	0.94	0.95
20	1.08	1.06	1.06	1.06	1.07	1.08	1.09	1.10	1.10	1.15	1.12
21	0.93	0.95	0.95	0.97	0.95	1.24	1.00	0.97	1.09	1.04	1.21
22	1.14	1.14	1.15	1.15	1.17	1.20	1.21	1.23	1.25	1.26	1.28
23	0.94	0.96	0.99	0.99	0.97	0.99	0.99	0.96	0.96	0.94	0.95
24	1.01	1.18	1.14	1.17	1.16	1.08	1.07	1.09	1.14	1.16	1.23

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PUBLICATIONS

1. Ozgenoglu M. (2015). Comparison of ASCE/SEI (2010) and Model Pushover based Ground Motion Scaling Procedures for Pre-tensioned Concrete Bridges., (*MSc. Thesis*). Middle East Technical University, Ankara, Türkiye.
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3. Ozgenoglu M. and Arıcı Y. (2019). Dynamic response of pile foundations: experimental research and current practice. *5th International Conference on Earthquake Engineering and Seismology Conference (5ICEES)*, Ankara, Türkiye.