การจำลองผลกระทบของปฏิสัมพันธ์ระหว่างดินและโครงสร้างที่มีต่อการขยายผลตอบสนองแผ่นดินไหวของอาคารบนชั้นดินกรุงเทพโดยใช้ ETABS

นายศรีเดช พรม

วิทยานิพนธ์นี้เป็นส่วนหนึ่งของการศึกษาตามหลักสูตรปริญญาวิศวกรรมศาสตรมหาบัณฑิต
สาขาวิชาวิศวกรรมโยธา
มหาวิทยาลัยเทคโนโลยีสุรนารี
ปีการศึกษา 2557

SIMULATED EFFECTS OF SOIL-STRUCTURE INTERACTION ON SEISMIC AMPLIFICATION OF BUILDING ON BANGKOK SUBSOIL USING ETABS

Sideth Prum

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

Suranaree University of Technology

Academic Year 2014

SIMULATED EFFECTS OF SOIL-STRUCTURE INTERACTION ON SEISMIC AMPLIFICATION OF BUILDING ON BANGKOK SUBSOIL USING ETABS

Suranaree University of Technology has approved this thesis submitted in partial fulfillment of the requirements for a Master's Degree.

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ศรีเดช พรม : การจำลองผลกระทบของปฏิสัมพันธ์ระหว่างดินและโครงสร้างที่มีต่อการขยายผลตอบสนองแผ่นดินไหวของอาคารบนชั้นดินกรุงเทพโดยใช้ ETABS (SIMULATED EFFECTS OF SOIL-STRUCTURE INTERACTION ON SEISMIC AMPLIFICATION OF BUILDING ON BANGKOK SUBSOIL USING ETABS) อาจารย์ที่ปรึกษา : ผู้ช่วยศาสตราจารย์ ดร.มงคล จิรวัชรเดช, 111 หน้า

ในการปฏิบัติโดยทั่วไป อาคารจะถูกออกแบบโดยสมมุติฐานของฐานรากยึดแน่นความเป็นจริง ดินที่รองรับจะเกิดการเคลื่อนที่ไปพร้อมกับฐานรากของอาคาร ทำให้ผลตอบสนองโครงสร้างมีการเปลี่ยนแปลงต่างไปจากสมมุติฐานฐานยึดแน่นที่มักใช้กัน ในการศึกษานี้ได้ทำการพิจารณาอาคารคอนกรีตเสริมเหล็กบนฐานรากเสาเข็มจากผลกระทบของปฏิสัมพันธ์ดิน-โครงสร้าง (SSI) ที่มีต่อผลตอบสนองของอาคารภายใต้แรงแผ่นดินไหว โดยใช้สปริงที่มีสติฟเนสเทียบเท่าคุณสมบัติของดินโดยรอบสติฟเนสของสปริงถูกคำนวณ และปรับเทียบค่าโดยผลการทดสอบเสาเข็มรับแรงด้านข้าง แบบจำลองของอาคาร และเสาเข็มถูกสร้างขึ้น และวิเคราะห์โดยใช้ซอฟท์แวร์ ETABS การวิเคราะห์สเปคตรัมผลตอบสนองถูกนำมาใช้ในการจำลองแรงแผ่นดินไหวเพื่อเปรียบเทียบผล จากสองสภาวะจุดรองรับที่ต่างกันได้แก่ คาบของโหมดการสั่นไหว, การโยกตัว และการโยกตัว สัมพัทธ์ของอาคาร, แรงเฉือนชั้น และโมเมนต์พลิกคว่ำ นอกจากผลตอบสนอง แผ่นดินไหว ของโครงสร้างที่ใช้สปริงปรับเทียบค่า ยังทำการศึกษาผลตอบสนองจากค่าสติฟเนสสปริงที่หลากหลาย จากการศึกษาพบว่าการคิดรวมผลของดินในการวิเคราะห์ส่งผลกระทบโดยรวมต่อผลตอบสนองโครงสร้าง และคาบการสั่นไหวของโครงสร้างเพิ่มขึ้นสองเท่าเมื่อเมื่อคิดผลของ SSI ในแบบจำลองการเพิ่มขึ้นของคานการสั่นไหวทำให้ความเร่งสเปคตรัม ใน กราฟสเปคตรัมเพิ่มขึ้น จากการศึกษาคาบของโครงสร้างหลายหลายกรณีของสปริงสติฟเนสพบว่า คาบโครงสร้างมีค่า ลดลงเมื่อสปริงสติฟเนสเพิ่มขึ้นสำหรับผลการโยกตัวและการโยกตัวสัมพัทธ์ของชั้นก็มีผลเพิ่มขึ้น เช่นกัน เมื่อ พิจารณา SSI โครงสร้างแสดงผลการโยกตัวที่มากกว่าทั้งในทิศทาง E-W และ N-S อย่างไรในการศึกษานี้อัตราส่วนการโยกตัวสัมพัทธ์ยังคงอยู่ในขีดจำกัดที่กำหนดในมาตรฐานออกแบบ (ASCE) ทั้งการโยกตัวและการโยกตัวสัมพัทธ์แสดงแนวโน้มลดลงในขณะที่สปริงสติฟเนสเพิ่มขึ้น แรงเฉือนชั้นและโมเมนต์ก็มีการเปลี่ยนแปลงอย่างมากเมื่อใช้ SSI โดยมีค่าเพิ่มขึ้นในทั้งสองทิศทาง นอกจากนั้นแรงเฉือนยังมีการเปลี่ยนแปลงที่สติฟเนสต่ำ และคงที่ที่สติฟเนสสูง ความคงที่ของแรงเฉือนอาจได้จากจุดรองรับ ที่มีความแข็งเกร็งเพิ่มขึ้นที่เข้าใกล้เงื่อนไขของจุดรองรับแบบยึดแน่น ผลการวิจัยพบว่า SSI มีผลอย่างมากต่อผลตอบสนองแผ่นดินไหวของอาคาร

สาขาวิชาวิศวกรรมโยธา ลายมือชื่อนักศึกษา________________________

ปีการศึกษา 2557 ลายมือชื่ออาจารย์ที่ปรึกษา__________________

SIDETH PRUM : SIMULATED EFFECTS OF SOIL-STRUCTURE INTERACTION ON SEISMIC AMPLIFICATION OF BUILDING ON BANGKOK SUBSOIL USING ETABS. THESIS ADVISOR : ASST. PROF. MONGKOL JIRAVACHARADET, Ph.D., 111 PP.

SIMULATED EFFECTS/ SOIL-STRUCTURE INTERACTION/ SEISMIC AMPLIFICATION/ BAGNKOK SUBSOIL/ ETABS

In practical work, buildings are generally designed with the assumption of having fixed support. In reality, the supporting soil creates some movement of the foundation. This alters the response of the structures due to inappropriate assumption of building supports. The present study considered a reinforced concrete building resting on pile foundation. Influence of soil-structure interaction (SSI) on response of the building subjected to seismic excitation was investigated by using the equivalent spring stiffness to represent the surrounding soil. The stiffness of the springs were calculated from the literature and calibrated by using the lateral pile load test. The model of the building with its piles was analyzed by using a conventional design software, ETABS. Response spectrum analysis was adopted to simulate the earthquake excitation. Modal periods, story displacements, story drifts, story shear, and overturning moment were observed and compared between 2 different support conditions. In addition to the specific seismic response of the structure with calibrated spring stiffness, various seismic responses were also investigated with variable stiffness of the springs. The study shows that the incorporation of soil in the analysis affects the overall response of the structure. The structural period increases two times when SSI was implemented in the model. The increase in structural period causes the spectral acceleration plotted in response spectrum to increase. The periods of the structure with various case studies of the spring stiffness were also observed. The results reveal that the structural period decreases when the spring stiffness increases. Regarding to story displacement and story drift, significant increasing results are noticed. With the consideration of SSI, the structure exhibits higher displacements and drift ratio in both E-W and N-S directions. However in this study, drift ratios are still in the limit of drift ratio specified in ASCE. Both displacement and drift express the same decreasing trend while the spring stiffness increases. Also, story shear and moment are dramatically altered due to the implementation of SSI. Story shear and moment increase in all considered directions. Moreover shear force exhibits higher fluctuation at low stiffness and tends to be constant at higher stiffness. The constant of the shear force may be obtained when the higher rigidity of the support is satisfied and the value tends to be that obtained in case of fixed support. The research outcome provides a considerable effect of SSI in seismic response of the buildings.

School of Civil Engineering Student's Signature___________________

Academic Year 2014 Advisor's Signature___________________

ACKNOWLEGEMENTS

After being admitted to study at Suranaree University of Technology in 2012, I started my higher education which led to a big change in my life. I came to the university to study Master's degree in the field of Structural Engineering. It was my great honor to work under the supervision of Asst. Prof. Dr. Mongkol Jiravacharadet. I would like to express my deepest gratitude and admiration for Asst. Prof. Dr. Mongkol Jiravacharadet for his invaluable advice, recommendation, guidance and encouragement throughout my study at Suranaree University of Technology.

Without the examining committee, this thesis would not be satisfied for the requirements of this Master's degree. I am thankful to Assoc. Prof. Dr. Chatchai Jothityangkoon for serving as chairman of this Master's degree thesis examining committee. I am also grateful to Asst. Prof. Dr. Pornpot Tanseng, assistant professor in school of civil engineering, for serving as thesis examiner and providing necessary data and advice for my research. Their comments and feedback are very invaluable to improve the quality of this thesis.

I would like to thank Prof. Dr. Suksun Horpibulsuk, Asst. Prof. Dr. Pornpot Tanseng, and Assoc. Prof. Dr. Sittichai Seangatith, professors of School of Civil Engineering, for their meaningful and clear lectures which are really useful in conducting this research as well as in practice.

I would like to thank all the staff and the faculty members in School of Civil Engineering, Suranaree University of Technology, for their supports in academic, administrative, and technical works during my study in Thailand.

I am indebted to my family. My parents and relatives are the greatest source of encouragement and support throughout my life. My parents are my idols and teachers who taught me from the first word until I have got the chance to get my higher education.

Last but not least, a word of thank is devoted to the ASEA UNINET Thailand On-place scholarship program for granting financial support for my study in Thailand. Financial support provided by Suranaree University of Technology is really appreciated.

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CHAPTER I

INTRODUCTION

1.1 Background

A problem endemic in design environment is a poor communication between structural and geotechnical specialists. This is a consequence of ever-increasing fragmentation of the engineering profession into sub-specializations. The structural engineer has a sophisticated understanding of construction materials and complicated design of structural elements, whereas geotechnical engineer is expert in the properties of soil and design of foundations on which structures are founded. The absence of closed involvement between the two results in confusion and/or inefficiency in structure/foundation design, especially when these two main parts of the construction are placed to perform together in severe conditions. The problem turns into more serious if the structure itself locates in an earthquake hazardous area. Earthquake has been a devastating phenomenon happening naturally for hundreds of millions of years (Datta, 2010). Even though the earth suffered from earthquake very long time ago, it was until around nineteenth century that people could develop instruments for measuring earthquake data. With this seismological data, earthquake engineers are able to make a rational design of structure to withstand earthquake. However it has also left the uncertain nature of future earthquakes for which such structures are to be designed. And the cost of damage remains a big problem for people to solve. There have been many cases reporting on seismic damages of structures. One of the most powerful earthquakes in history happened in Chile in 1960. This enormous seismic caused millions dollars of damage and claimed hundreds of people's lives. Therefore the seismic design of structures needs to be carried out rigorously to prevent such an unexpected catastrophe, particularly for seismic hazardous zones.

Bangkok, the capital city of Thailand, which is located far from active faults of low seismic activity rate (120 to 300 km) or more seismic active sources (400 to 1000 km) is still suffered from earthquake excitation. This is due to the underlying soft clay that amplifies earthquake ground motions up to 4 times (Warnitchai et al., 2000). The 1985 Michoacan earthquake with amplitude of 8.1 is another well-known case of far fault earthquake excitation causing considerable destruction and death tolls. The earthquake causing the destruction in Mexico City, 350 km from the epicentral location, was due to significant amplification of earthquake ground motions by underlying soft soil deposits in the downtown area of the city (Seed, 1987). These problems have led many researches of seismic response of structures especially with participation of soil performance. The participation of soil performance which is called soil-structure interaction (SSI) has become an active research topic for both structural and geotechnical engineers in the last few decades. A widely-accepted perception of soil-structure interaction in most design codes is its beneficial role in the design of structures. Design acceleration spectra resulting from actual recordings of many elastic response spectra consists normally three branches: increasing, constant and decreasing acceleration branches. Whereas the constant acceleration branch of a soft deposit soil can take up to 1 sec period (Gazetas, 2006). This long natural period may lead to smaller acceleration, bending moment, and base shear of majority of building structures and their foundation due to its position in the decreasing acceleration branch of conventional response spectra (Fardis, 2005). It is also noted similarly in ASCE7-05 (2006) that the base shear of the structure is reduced for an amount in case of soil-structure interaction. However the beneficial role of soil in SSI has become an unclear thing. It has been shown in many documents and case histories that over-simplification of the beneficial role of SSI may lead to a non-conservative design of structures, hence causes destruction of structures during earthquake. The collapses of long elevated highway section of Hanshin Expressway's Route 3 in Kobe (Mylonakis et al., 2006) and buildings in the recent Adana-Ceyhan earthquake (Celebi, 1998) have been caused by detrimental role of soil.

Therefore this paper aims to study the detrimental role of soil participating in seismic response of structures. A case study of a building with its corresponding soil profile was used to observe its elastic response while it is subjected to earthquake excitation. This study is useful for understanding the performance of a structure with its underlying soil properties when earthquake occurs.

1.2 Research objective

1.2.1 To analyze of a reinforced concrete structure subjected to earthquake excitation.

1.2.2 To compare the seismic response of building structure with different support conditions: fixed and spring-support bases and to compare both results with design requirements specified in design provisions.

1.3 Hypothesis of research

A reinforced concrete core-wall building with 10 stories was analyzed by using a conventional structural analysis and design software, ETABS (Computers & Structure Inc, Berkeley, USA, 1995). The building located in Bangkok area was constructed on pile foundation. The soil surrounding the pile foundation was simulated to be equivalent springs with appropriate stiffness and be applied to the corresponding piles.

1.4 Scope of research

1.4.1 Analysis of 10-story building by using three-dimensional analysis program with different support conditions: fixed base which is commonly adopted in building design and spring-support base (elastic base) in which surrounding soil is taken into account.

1.4.2 Considered building is a reinforced concrete building with post tension flat-slab. In the ease to understand clearly the behavior as well as interaction properties, structural model is modified to be a symmetry model.

1.4.3 The underlying soil is soft clay in Bangkok area.

1.4.4 The equivalent soil springs are considered in linearly elastic range.

1.4.5 Response spectrum in Bangkok area is used for simulating earthquake excitation on building.

1.5 Research procedure

1.5.1 Study the previous research on related problems and considered building.

1.5.2 Determine significant properties of soil to be applied in the analysis procedure.

1.5.3 Evaluate the equivalent soil spring properties.

1.5.4 Analyze response spectrum resulted from earthquake excitation in Bangkok.

1.5.5 Create the models and apply input data.

1.5.6 Analyze the model and check the results.

1.5.7 Conclusion and discussion on obtained results.

1.6 Advantage of research

1.6.1 Understand the behavior of reinforced concrete building structure subjected to earthquake excitation.

1.6.2 Understand the participation of soil in seismic response of building and influence of soil stiffness on seismic response of structures

1.6.3 Be able to predict and decide whether or not soil structure interaction should be taken into account in building design.

CHAPTER II

THEORITICAL BACKGROUND AND LITERATURE REVIEW

2.1 Introduction

The state-of-the-art in Soil-Structure-Interaction (SSI) has been developed gradually over the last several decades. The participation of soil in the analysis of structures has started since the machinery basements were analyzed for the dynamic interaction with soil foundation (the most impressive of them probably being turbines). It was obviously a quasi-static approach such as the well-known static solution for rigid stamps, beams and plates on elastic foundation. However, the term “SSI” was not yet introduced at the time (Tyapin, 2012). To understand more clearly on SSI, the development of new powerful tools is needed to analyze more realistic models. Then homogeneous half-space with surface rigid stamp was used as SSI models. The improvement of the model was applied to move from the homogeneous half space to the horizontally-layered medium in soil modeling (Enrique Luco, 1976; Kausel et al., 1975). Through the continuous process of researches in SSI, its application has been introduced in some seismic provisions such as NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures and FEMA 440, Improvement of Inelastic Seismic Analysis Procedures. However the latter provides a practical application of SSI since it incorporates the effects of soil-structure interaction in nonlinear static pushover-type analyses. The procedures were finally adopted into

ASCE/SEI 41-06, Seismic Rehabilitation of Existing Buildings (ASCE, 2007). This has shown the importance of soil participation in the analysis of structures subjected to seismic loading.

2.2 Overview on the analysis of SSI

The analysis of SSI can be done with the finite element method or any other discretization scheme such as finite differences or the boundary element method (Villaverde, 2009). The finite element method is a powerful technique for modeling soil-structure systems since it considers the three dimensional problem, irregular geometries, the vertical and horizontal variation of soil stiffness, foundation embedment, foundation flexibility, and nonlinear behavior of the soil and structural elements. Even though finite element method can be used for various conditions of soil-structure interaction, it is not free of problem. First, artificial boundary which is theoretically unbounded is required. Highly computational computer with large amount of storage is needed as the modeling of soil and structure with finite elements consists of an extraordinarily large number of degrees of freedom. Different methods have been used to solve soil-structure interaction problems due to the boundary condition and the desire to reduce the complexity of the problem. Broadly, the methods of the analysis are categorized as direct and substructure approaches. In a direct analysis, the soil and structure are modeled and analyzed as a complete system. In a substructure approach, soil and structure are divided into distinct parts that are combined to formulate a complete solution.

2.2.1 Direct analysis

As mentioned above, the soil and structure are included in one unit and analyzed as a complete system. The soil is represented as a continuum together with structural elements and foundation. The method is illustrated in Figure 2.1.

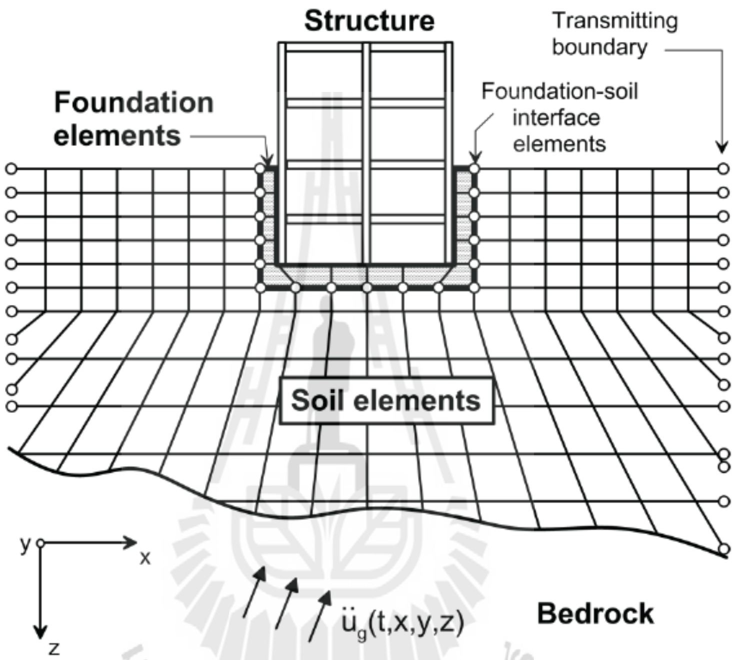


Figure 2.1 Schematic illustration of a direct analysis of soil-structure interaction using continuum modeling by finite element (NIST, 2012)

The performance of such analysis is normally done by using equivalent linear representation of soil properties in finite element, finite difference or boundary element formulations (Wolf, 1985). Figure 2.2 shows an example of a direct method using linear soil and structural elements in the program OpenSees (Mazzoni et al., 2006).

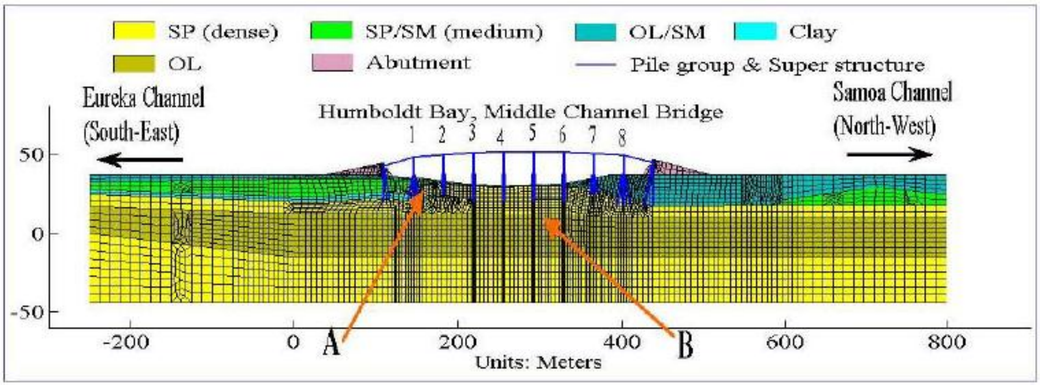


Figure 2.2 Example of direct model of soil-foundation-structure system using openSees (Stewart & Tileylioglu, 2007).

However, adaptation of this method in the analysis requires high computational effort particularly when the geometry is complex or contains significant nonlinearities in the soil or structural materials. Hence it is rarely used in practice (NIST, 2012).

2.2.2 Substructure approach

Soil is considered to be unbounded, while structure is a bounded system. It seems reasonable to consider the two sub systems with different properties separately. Hence substructure methods have been proposed in analyzing SSI. In the substructure approach, a proper consideration of SSI effects is required as followed: (i) an evaluation of free-field soil motions and corresponding soil material properties; (ii) an evaluation of transfer functions to convert free-field motions to foundation input motions; (iii) incorporation of springs and dashpots to represent the stiffness and damping at the soil foundation interface; and (iv) a response analysis of the combined structure-spring/dashpots system with the foundation input motion applied (NIST, 2012). The step of the substructure approach is show in Figure 2.3.

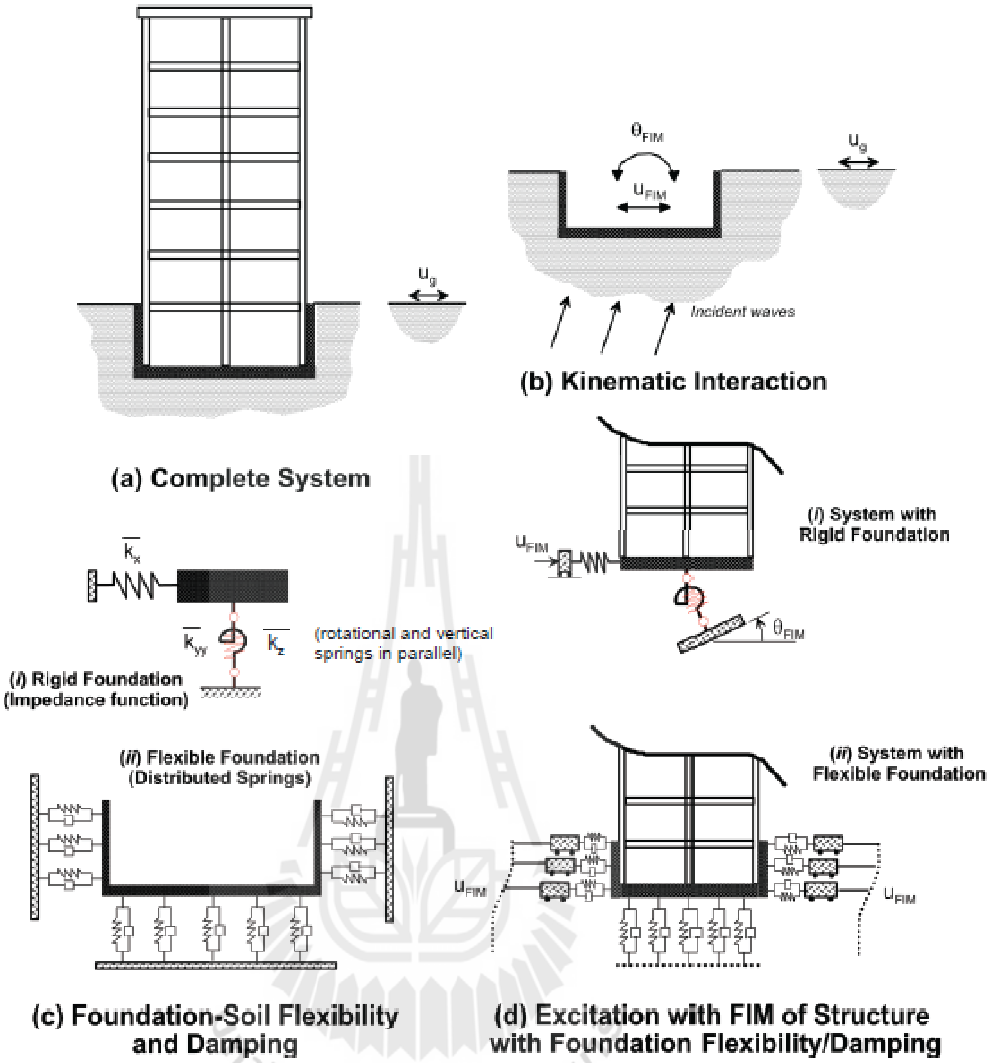


Figure 2.3 Schematic illustration of a substructure approach to analysis of soil-structure interaction using either: (i) rigid foundation: (a) Complete System; (b) Kinematic Interaction; or (ii) flexible foundation assumptions: (c) Foundation-Soil Flexibility and Damping; (d) Excitation with FIM of Structure with Foundation Flexibility/Damping (Stewart & Tileylioglu, 2007).

2.3 Modelling of SSI

Various approaches have been used for modeling the base of the buildings to account for soil-structure interaction. Those approaches can be relatively simple or complicated and time-consuming. The problem is whether a complicated and time-consuming model can produce significantly more accurate results. The modeling depends also on choosing the method of analysis (direct or substructure approach) and whether it is an embedded structure or structure resting the ground surface.

2.4 Response of pile in SSI

To account for SSI, various methods have been used to observe the behaviors of a structure according to the model used in the analysis. When a model consisting of both superstructure and pile foundation is used, the performance of the piles obviously has influence on the response of the superstructure. Several methods have been published for predicting the response of single piles under lateral loading (Broms, 1966; Desai, 1974; Hetényi, 1946).

Dash et al. (2008) used p-y curve to model lateral response of pile foundations in liquefied soils. Beam on Nonlinear Winkler Foundation (BNWF) was used to analyze versatile soil-pile interaction. In the BNWF model, the soil is represented by a set of independent springs lumped at discrete location along the pile. The study also discussed the effect of the load-displacement curve in soil-pile interaction. Figure 2.4 illustrates the influence of pile movement on its p-y curve. As shown in Figure 2.4a, when the differential soil-pile movement is small, the resistance on pile depends on the initial stiffness of the soil and the value of deflection. When the differential soil-pile movement is large, the resistance offered by soil over pile is governed by the ultimate strength of the pile.

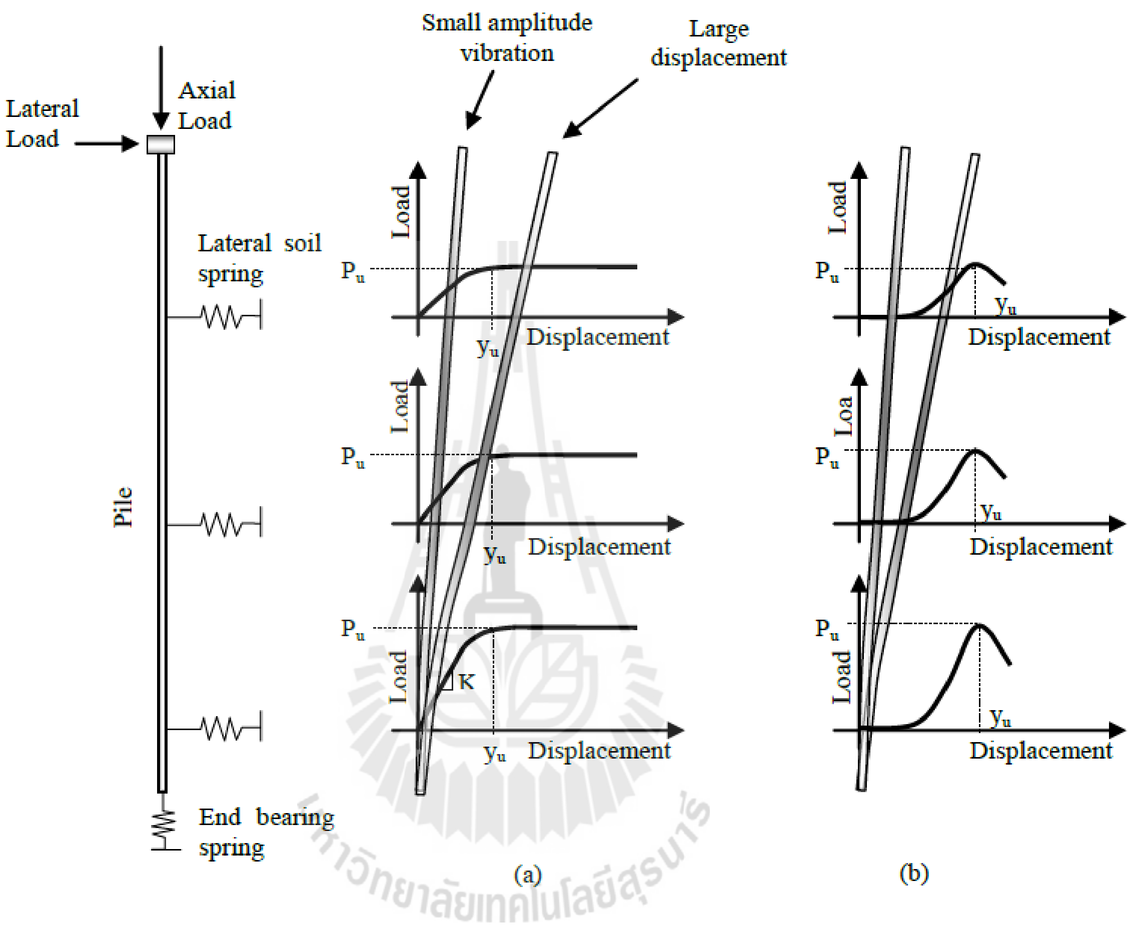


Figure 2.4 Soil-pile interaction, (a) for small amplitude soil-pile movement, (b) large amplitude soil-pile movement. (Dash et al., 2008)

However the response of the pile will exactly change if the shape of the p-y curve is chosen as in Figure 2.4b. The advantage of using the later model is the higher strength and stiffness at large differential pile-soil movement, which may prevent a complete collapse of a structure.

In the evaluation of seismic capacity of post-tensioned concrete slab-column frame buildings, Boonyapinyo et al. (2006) employed Winkler component model (Figure 2.5) represented by series of independent or uncoupled lateral and axial springs in order to study the behavior of foundations.

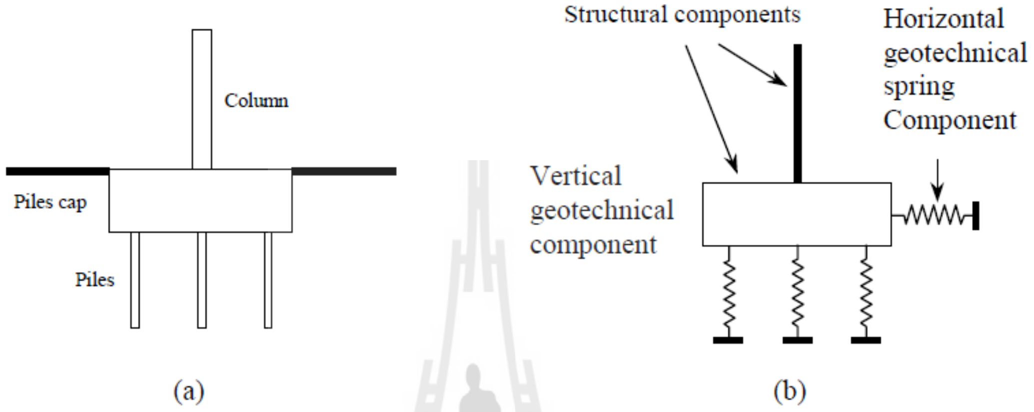


Figure 2.5 Winkler component model, (a) deep foundation; (b) model for analysis. (Boonyapinyo et al., 2006)

On the other hand, it is complicated to analyze a pile under lateral loading since the movements of soil and pile are dependent. In this study, the subgrade reaction model originally proposed by Winkler in 1867 is used to determine the lateral force-deformation relations. The model of the subgrade reaction is illustrated in Figure 2.6a.

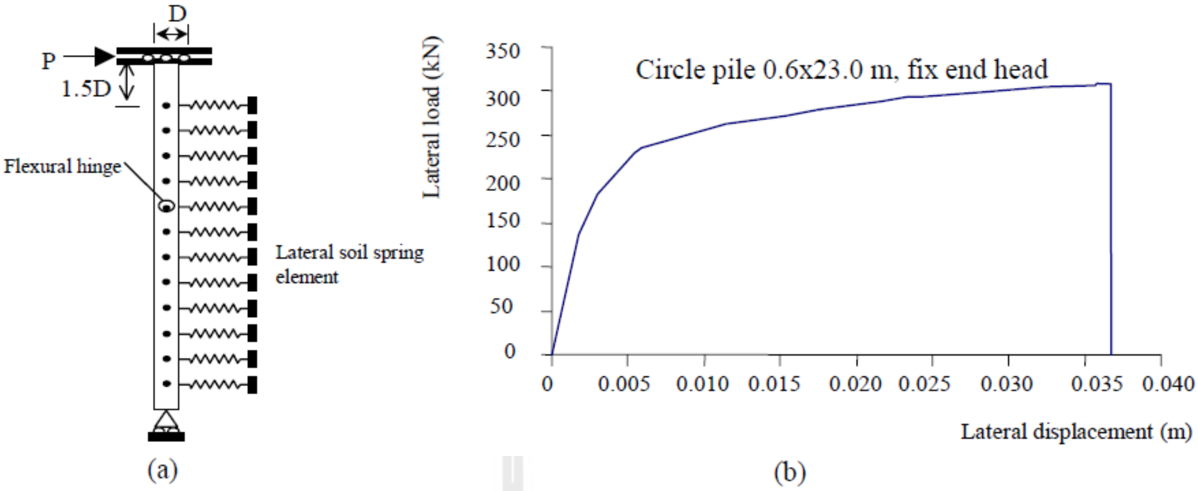


Figure 2.6 (a) Model of pile with fixed head; (b) lateral load-displacement relationship of the pile. (Boonyapinyo et al., 2006)

The flexural hinge having moment-rotation relation is applied at the toe of the pile to represent the flexural behavior of reinforced concrete pile under lateral load. The predicted lateral load-displacement of pile is in good correlation with the test results obtained from static lateral load test of three sites in Bangkok (Figure 2.6b).

2.5 Implementation of SSI in seismic design codes

Soil-structure interaction has been implemented in many seismic code provisions such as ATC-40 (Comartin et al., 2000), ASCE. (1998), FEMA (2009), PEER (2010). Even though soil-structure interaction is included in many seismic provisions, its usage in practical work is still an optional procedure. It is believed that accounting for SSI can only reduce base shear demands. Whereas ignoring SSI is not only easier, but also more conservative. However including SSI in the analysis provides a better insight into structural performance and to improve accuracy in the analytical simulation of important structural response quantities.

2.5.1 Force-based Procedures

The effective building period or period lengthening is calculated using an equation (19.2-3) in ASCE 7-10.

$$\tilde{T} = T\sqrt{1+\frac{\bar{k}}{K_y}\left(1+\frac{K_y\bar{h}^2}{K_\theta}\right)} \tag{2.1}$$

where $\tilde{T}$: the effective period of the building

T : the fundamental period of the structure

$\bar{k}$: the stiffness of the structure with fixed base, defined by the following equation

$$\bar{k} = 4\pi\left(\frac{\bar{W}}{gT^2}\right) \tag{2.2}$$

where $\bar{h}$: the effective height of the structure ($0.7h_n$)

K_y : the lateral stiffness of the foundation

K_θ : the rocking stiffness of the foundation

g : the acceleration of gravity

$\bar{W}$: the effective seismic weight of the structure

However, ASCE/SEI 7-10 does not specify how to evaluate K_y and K_θ . Their values are recommended elsewhere in the commentary to the NEHRP Recommended Provisions (FEMA, 2009). Both ASCE/SEI 7-10 and NEHRP Recommended Provisions provide adjusted values for average shear modulus (*G*) and average shear wave velocity v_s (at large strain levels from shear modulus at small strain, G_o, to account for large strain effects. The values of both *G* and v_s in Table

19.2-1 of ASCE/SEI 7-10 is summarized in Table 1 in which S_{D1} is the design spectral response acceleration parameter at a period of 1.0 s.

Table 2.1 Values of G/G_o and v_s/v_{so}

	Spectral Response Acceleration, S_{D1}			
	≤ 0.10	≤0.15	≤0.20	≥0.30
Value of G/G_o	0.81	0.64	0.49	0.42
Value of v_s/v_{so}	0.90	0.80	0.70	0.65

In special case of near-ground surface or embedded structure supported on mat foundations that the side wall contact with the soil is not considered to be effective during the design ground motion, the effective period of the structure is determined by Equation 2.3.

$$\tilde{T} = T\sqrt{1+\frac{25\alpha r_a \bar{h}}{v_s T^2}\left(1+\frac{1.12 r_a \bar{h}^2}{\alpha_\theta r_m^{\ 3}}\right)} \tag{2.3}$$

where α : the relative weight density of the structure and the soil defined as in Equation (2.4).

$$\alpha = \frac{\bar{W}}{\gamma A_o \bar{h}} \tag{2.4}$$

γ : the average unit weight of the soils

r_a and r_m : characteristic foundation lengths shown in Equations (2.5) and (2.6).

$$r_a = \sqrt{\frac{A_o}{\pi}} \tag{2.5}$$

and
$$r_m = \sqrt[4]{\frac{4I_o}{\pi}} \tag{2.6}$$

where A_o : the area of the load-carrying foundation

I_o : the static moment of inertia of the load-carrying foundation about a horizontal centroidal axis normal to the direction in which the structure is analyzed

α_θ : dynamic foundation stiffness modifier for rocking as shown in Table 2.2.

Table 2.2 Value of α_θ

$r_m / v_s T$	α_θ
<0.05	1.00
0.15	0.85
0.35	0.70
0.50	0.60

Chapter 19 of ASCE/SEI 7-10 specifies the application of soil-structure interaction into the equivalent lateral force procedure in which shear force is expressed in Equation (2.7).

$$V = C_s \bar{W} \tag{2.7}$$

Where V : the seismic base shear considering SSI.

C_s : the seismic coefficient taken as the design response spectral ordinate.

$\bar{W}$: the effective seismic weight of the structure (taken as 70% of the total weight).

Kinematic interaction effect is neglected in ASCE/SEI 7-10 and NEHRP Recommended Provisions but account for inertial interaction effects related to period lengthening and damping ratio. The reduction of base shear (ΔV) shall be computed as follows and shall not exceed 0.3V (ASCE7-05, 2006).

$$\Delta V = \left[C_s - \tilde{C}_s \left(\frac{0.05}{\tilde{\beta}} \right)^{0.4} \right] \bar{W} \leq 0.3\mathrm{V} \tag{2.8}$$

where $\tilde{C}_s$: the value of C_s calculated from the design spectrum at an elongated period, $\tilde{T}$

$\tilde{\beta}$: the fraction of critical damping for the structure foundation system determined as follows.

$$\tilde{\beta} = \beta_o + \frac{0.05}{(\tilde{T}/T)^3} \tag{2.9}$$

where β_o : the foundation damping factor as shown in Figure 2.7.

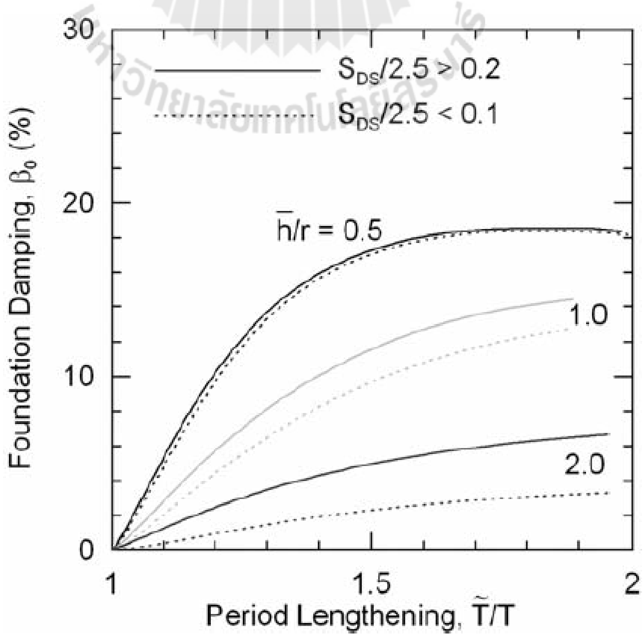


Figure 2.7 Foundation damping factor (ASCE 7-10)

For values of $\frac{S_{DS}}{2.5}$ between 0.10 and 0.20 the values of β_0 shall be calculated by linear interpolation between the solid lines and the dashed lines of Figure 2.7.

The quantity r in Figure 2.7 is a characteristic foundation length that is determined as follows:

$$\text{For } \frac{\bar{h}}{L_o} \le 0.5, \quad r = r_a \tag{2.10}$$

$$\text{For } \frac{\bar{h}}{L_o} > 1, \quad r = r_m \tag{2.11}$$

where L_0 : the overall length of the side of the foundation in the direction being analyzed.

As noticed from the above calculation, base shear of the structure is reduced when SSI is taken into account. In practice, beneficial effects of period lengthening and foundation damping are negligible for tall, flexible building (NIST, 2012).

2.5.2 Response History Procedures

Many seismic provisions have included the methods for accounting soil-structure interaction in force-based procedure. However some provisions are still silent on the application of SSI effects in response history analyses. ASCE/SEI 7-10 does not offer specific guidance on how to select and utilize springs in response history even though they allow the use of equivalent soil springs in principal. Guidelines for Performance-Based Seismic Design of Tall Buildings (PEER, 2010) recommends a response history analysis procedure for SSI analysis. Figure 2.8 shows simplified model to streamline response history analysis. Two idealization of SSI were considered in PEER Guidelines, depending on the shaking intensity: service level earthquake or maximum considered earthquake which are shown in Figure 2.8b and Figure 2.8c, respectively. As noticed in Figure 2.8b, response history analysis for service level earthquake uses simple model with fixed base support, while that for maximum considered earthquake is performed by considering soil-foundation interaction represented by springs and dashpots.

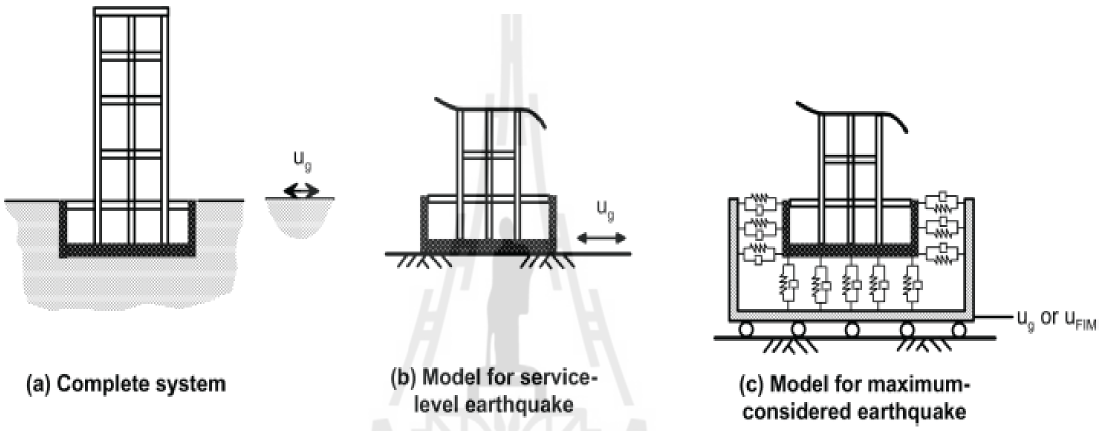


Figure 2.8 Schematic illustration of a tall building with subterranean levels: (a) complete system; (b) simplified for service-level earthquake intensity and (c) simplified foundation model for maximum considered earthquake intensity. (PEER, 2010)

2.6 Review of soil and structure responses in SSI

Seismic input and soil conditions are both important in determining the performance of the soil-structure system. When flexibility of the soil underneath a structure is taken into account in the analysis, it does not only change the behavior of whole system but also increases the seismic response of the structure especially in case that the underlying soil is soft deposit. Ashford et al. (2000) studied the potential amplification of earthquake ground motions in soft Bangkok soil. The study was conducted by using the equivalent linear method. Soil property namely shear wave velocity estimated from existing correlation with field and laboratory data was used as input in the analysis of seismic site response. The estimated shear wave velocity is shown in Figure 2.9. The value was also confirmed by a certain number of in situ tests by the downhole method. Five input strong motion records from far-field sites and the effect of the assumed depth to rock-like material were studied and used in the analysis.

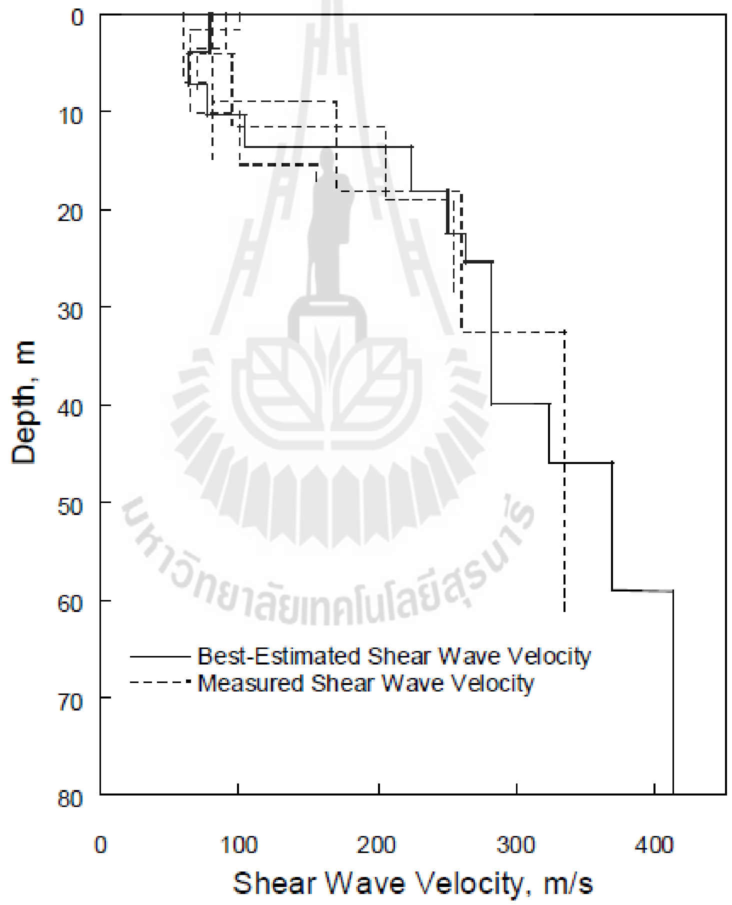


Figure 2.9 Comparison between measured shear wave velocity profiles to the best-estimated profile for Bangkok based on empirical correlations. (Ashford et al., 2000)

The results of the study reveal that the soils underlying Bangkok has the ability to amplify earthquake ground motion, both in peak ground acceleration and spectral acceleration. Figure 2.10 illustrates normalized acceleration response spectra for Bangkok site. It was stated similarly on the amplification at soft clay sites in downtown Mexico City and in the San Francisco Bay Area.

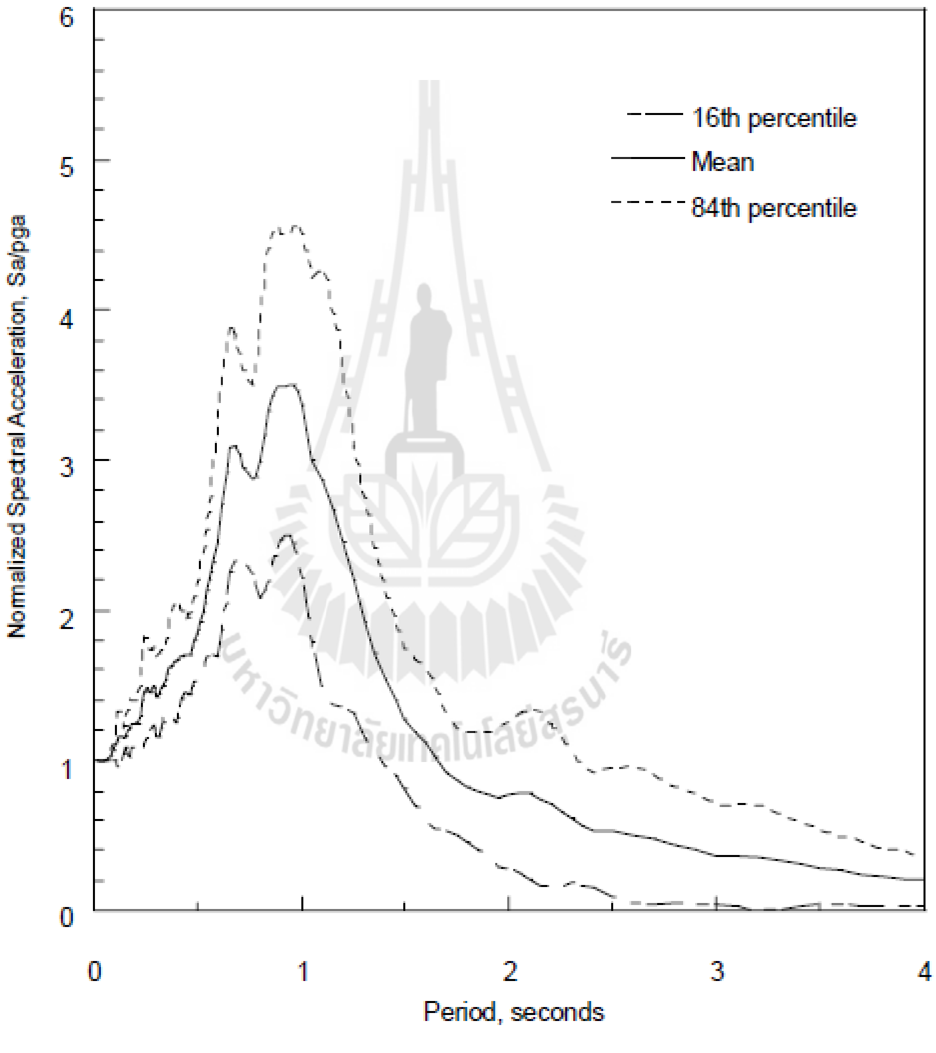


Figure 2.10 Best-estimate normalized acceleration response spectra for Bangkok. (Ashford et al., 2000)

Similar to previous research, Warnitchai et al. (2000) investigated seismic hazard in Bangkok due to long-distance earthquakes. The study was to assess a seismic hazard of Bangkok by predicting peak ground accelerations for various levels of probability of excedance in a 50-year period and the corresponding elastic response spectra. To conduct the research, soil properties for generalized soil profile were developed. Those properties includes shear wave velocity (or low-strain dynamic shear modulus) and mass density, shown in Figure 2.11, and relationships for variation of dynamic shear modulus and damping ratio as a function of strain, adopted from Vucetic and Dobry (1991) for clay and Seed et al. (1986) for sand.

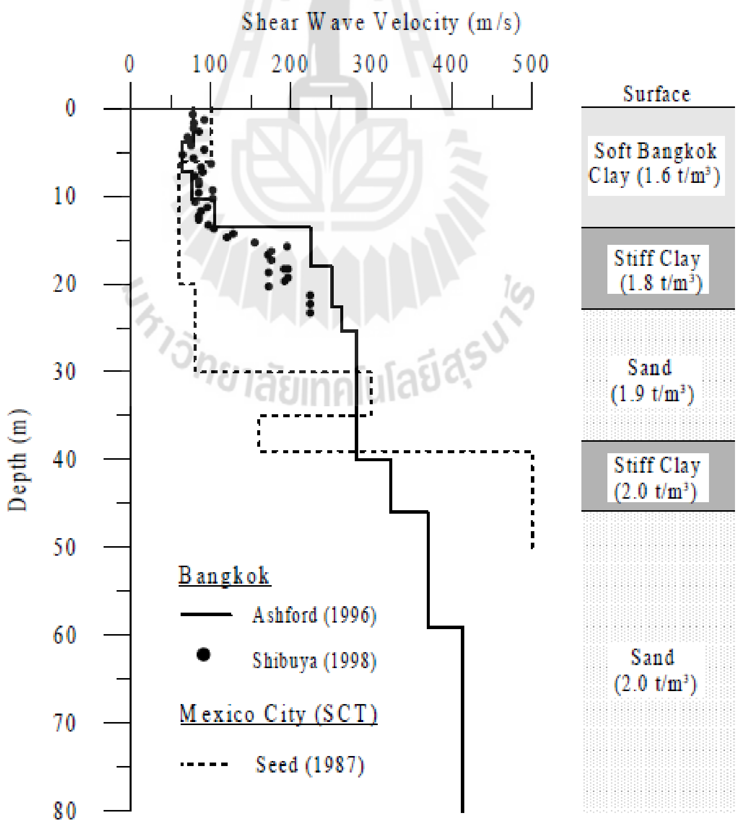


Figure 2. 11 Generalized Bangkok soil and shear wave velocity profiles.

(Warnitchai et al., 2000)

Seven different accelerograms selected from actual acceleration records at rock sites with magnitude from 7 to 8 earthquakes at source-to-site distances from 80 to 350 km were used to represent rock outcrop motions in Bangkok area. Based on the study result shown in Figure 2.12, the relationship between amplification factor and peak rock acceleration clearly indicated that soft Bangkok soil has the ability to amplify earthquake ground motion from 3 to 6 times for extremely low intensity input motions and 3 to 4 times for relatively stronger input motions. The amplified ground motion can be noticed by narrowband random motions with a relatively long predominant period for about 1 second as shown in Figure 2.13. The mean and 84th percentile spectra for ground motions characterized capable ground motion in Bangkok area are comparable to spectral acceleration in Mexico City during the 1985 earthquake.

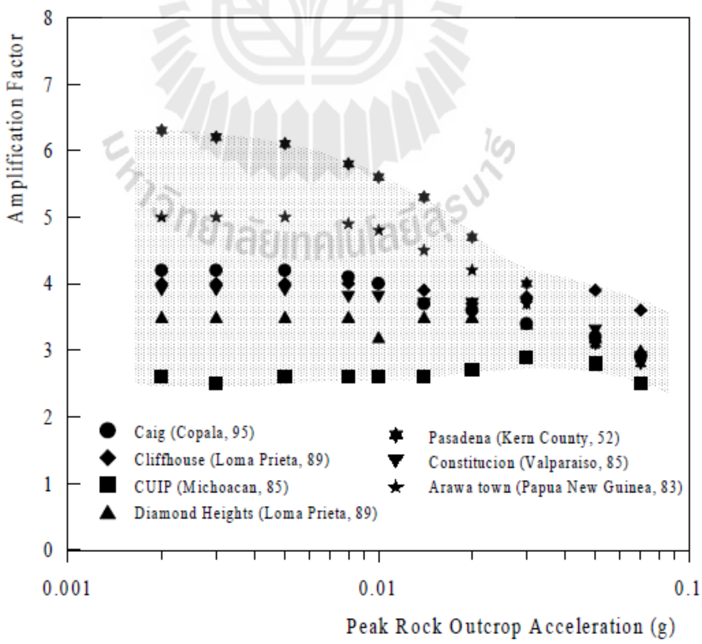


Figure 2.12 Relationship between computed amplification factor and peak rock outcrop acceleration. (Warnitchai et al., 2000)

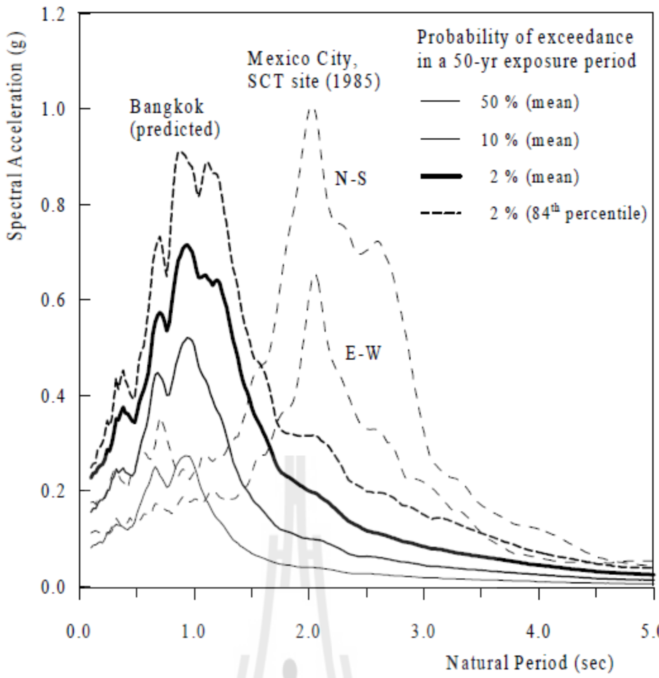


Figure 2.13 Comparison between the elastic response spectra of predicted ground motions and the spectra of the damaging ground motions in Mexico City. (Warnitchai et al., 2000)

According to these results, severe damage or complete collapse of structures with periods ranging from about 0.5 sec to 1.5 sec as well as to short-period structures would occur during earthquake event if sufficient lateral strength is provided.

Chandrasakha (2013) studied the effect of SSI on the response spectra for earthquake resistant design in Bangkok by using conventional finite element analysis program, STAAD Pro. The study was conducted on a case-study of a 10-storey reinforced concrete building resting on pile foundation in Bangkok area. The equivalent soil-spring stiffness was determined by using the modulus of subgrade reaction and was applied along the pile. Figure 2.14 shows the model of equivalent soil spring. The model was used to perform pushover analysis in STAAD Pro. A series of horizontal loads were applied at the head of the pile and displacements of pile head were recorded correspondingly. Figure 2.15 shows the load-displacement curve of the pile.

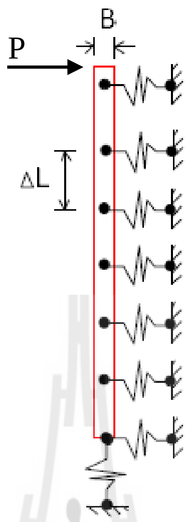


Figure 2.14 Model of equivalent soil spring along the pile. (Chandrasakha, 2013)

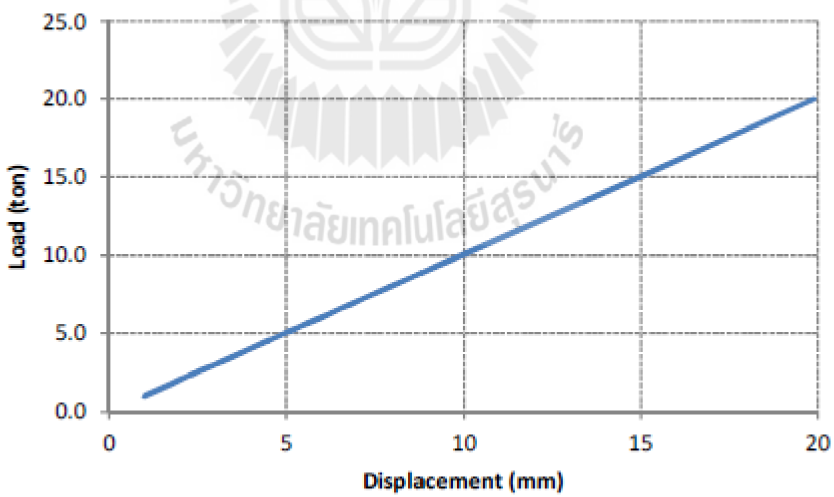
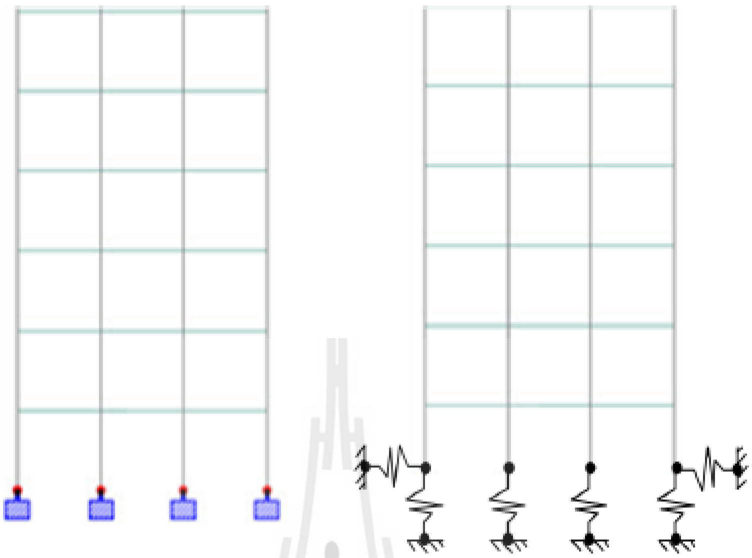


Figure 2.15 Load-displacement curve of pile head. (Chandrasakha, 2013)

Horizontal stiffness of the system (pile-spring) was computed and applied at the base of the structure to account for flexibility of the pile foundation. The vertical stiffness of the pile was calculated by using the properties of the pile. The models for the considered structure are illustrated in Figure 2.16a and 2.16b.



(a) (b)

Figure 2.16 The model of considered structure with (a) fixed base; (b) flexible base (Chandrasakha, 2013)

Based on the study results, it was revealed that the performance of the structure changed dramatically. Figure 2.17 illustrates the increase of floor displacement of the building while its inter-story drifts are altered. It is noticed as well that the forces in the structure such as base shears and overturning moments are also changed due to the flexibility of the foundation.

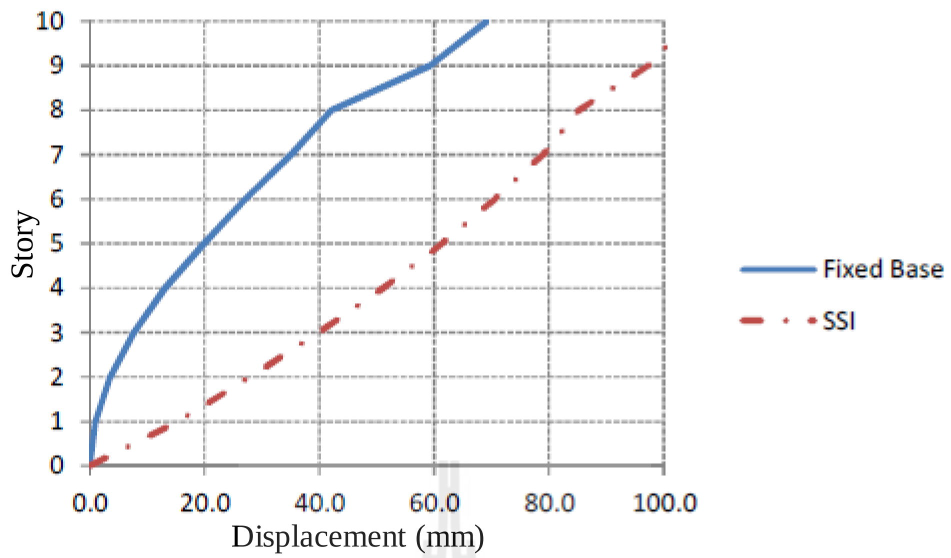


Figure 2.17 Floor displacements of the structure. (Chandrasakha, 2013)

However in this study, the load-displacement curve of the pile head is presented as linear curve which is not a well-represented curve for behavior of the soil. The present considers this effect by calibrating the considered model of the pile with test data in order to obtain a reasonable performance of the used model.

Nakpant (2007) conducted a research on the seismic capacity of a building with consideration of soil-structure interaction effects. A 14-storey building locating in Bangkok area was used in the study. Vertical and horizontal springs were modeled at the base of the building. In this study, nonlinear dynamic analysis was employed. According to the study results, it was observed that the seismic capacity of the building with flexible support was less than that of the building with fixed base. In addition, the seismic damage of the structure with flexible support was greater than that of fixed base. Nevertheless, point springs were employed at the base of the structure which stood on pile foundation. This assumption may reduce the performance of the pile in the analysis. The aim of the present study is to model the pile foundation together with its super-structure model. Hence, whole performance of the structure can be observed.

Similarly, Wan et al. (2000) studied the effect of SSI for continuous bridges. The equivalent soil springs for both pile foundation and footing were calculated from empirical equations and implemented in the model. The methodology in analyzing the footing and pile foundation was to modify them as linear spring to calculate the continuous bridge response. It was recommended that the simulation in linear soil spring may not be accurate enough to predict the structural response.

CHAPTER III

METHOD AND METHODOLOGY

3.1 Structural elements

3.1.1 Super-structure elements

To simplify the analysis and understand clearly the effect of soil structure interaction, a case-study of a symmetric structural model was used. The structure is a reinforced concrete core-wall building. The dimensions of the structure are 36 m length by 24 m width with 27.5 m height. Floor plan supporting live load of 3 kN and supper imposed dead load of 1 kN of the building is shown in Figure 3.1.

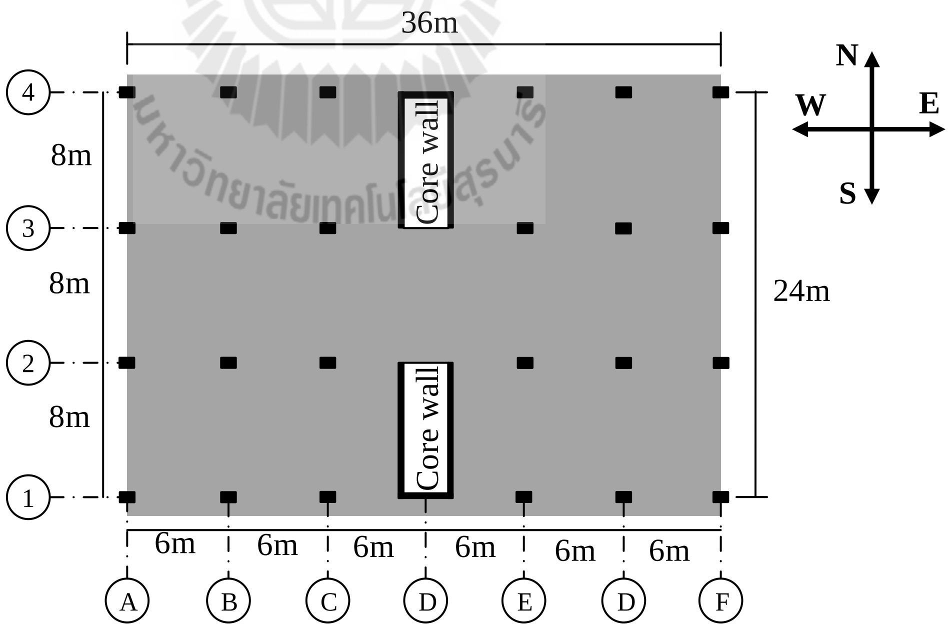


Figure 3.1 Floor plan of the case-study building

There are totally 10 floors with height of 3 m for each floor. At each floor level, it consists of a flat slab with thickness of 20 cm. 10-story building was chosen in this study because the low rise building and medium rise buildings possess short structural periods which are critical for the considered response spectrum in which the maximum spectral acceleration is observed at the period of about 2 seconds. The flat slab is supported by columns with dimensions of 50 cm by 60 cm and two core-wall structures with the same thickness of 25 cm. The core wall were employed as laterally resisting elements. In this study, the compressive strength of concrete was chosen to be 30 MPa and yielding strength of rebar was 400 MPa. Young modulus of concrete and steel were 25,743 MPa and 200,000 MPa, respectively.

3.1.2 Sub-structure elements

The sub-structure model consists of footings and piles while the links between the first floor slab and the footings are stump columns with height of 0.5 m and have the same dimensions as the columns of the upper stories. Each footing measuring 1.5 m x 1.5 m x 1 m located under each column, while the footings with dimensions of 4.2 m x 9.2 m x 1 m were used to support each core-wall structure. Figure 3.2 illustrates floor plan at footing level.

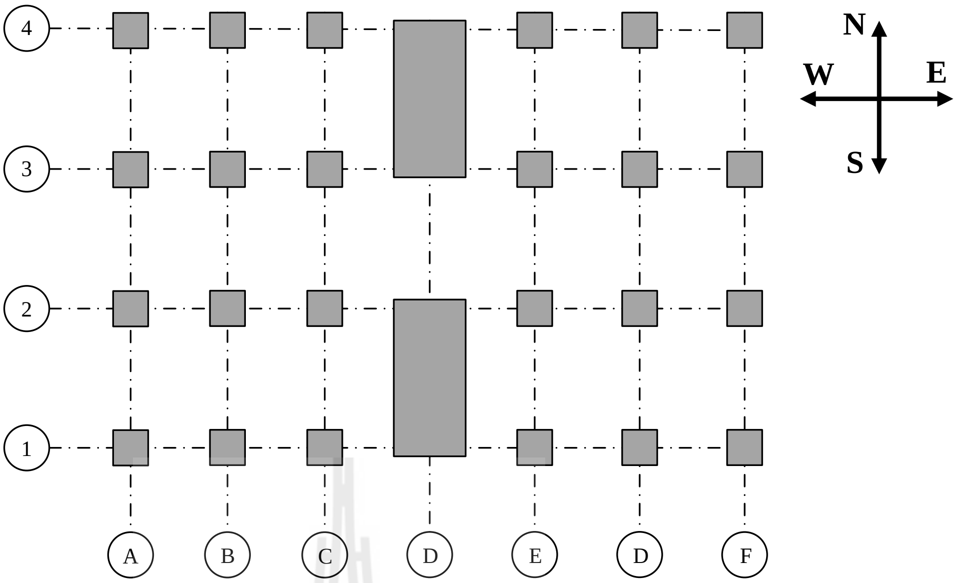


Figure 3.2 Floor plan at footing level

Under each column as well as each footing, a pile with diameter of 1 m and length of 55 m was modeled to support the load transferred from super-structure. On the other hand, 6 piles with the same diameters and length were used to support the load transferred from each core-wall. Figure 3.3 shows the 3 D model of piles with footings.

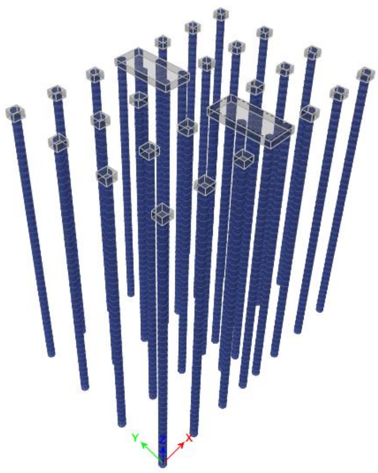


Figure 3.3 3D pile-footing model

3.2 Response spectrum acceleration

Response spectrum is a practical approach used in the design of structures and development of lateral force requirements in building codes via implementing the knowledge of structural dynamics. It is the plot of the peak value of a response quantity such as spectral acceleration, circular frequency, and cyclic frequency as a function of natural vibration period of the structure. Since it is the plot of the response quantity of a specific site, it should be chosen corresponding to the location of the site. In this study, the response spectrum acceleration was obtained from Thai seismic code which is used specifically for seismic design of structures in Bangkok area. The design response spectrum for the analysis is shown in Figure 3.4. As seen in Figure 3.4, the response spectrum consists of 6 different branches. The first 3 branches are in the increasing parts of the spectrum, whereas the last 3 branches are in the decreasing parts and the maximum period considered is up to 6 seconds while the maximum spectral acceleration occurs at the period of about 2 seconds. In addition this response spectrum shows no constant acceleration branch as in some seismic codes. This case complies with the study of the seismic response of soft soil such as that indicated by Ashford et al. (2000) shown in Figure 2.10.

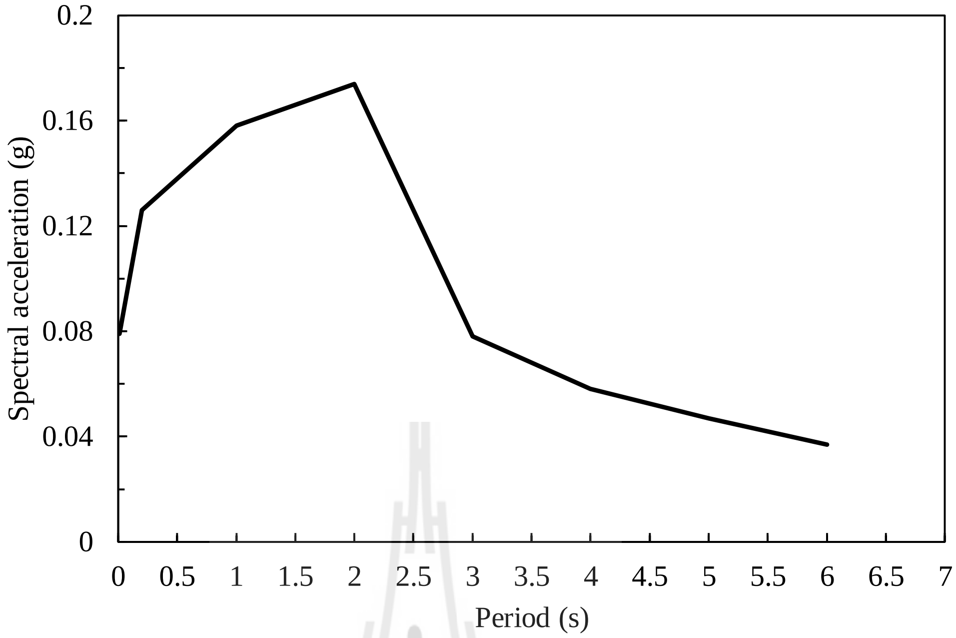


Figure 3.4 Bangkok Response Spectrum

3.3 Soil properties of Bangkok

Bangkok is situated on a large flat plain with the length of about 250 km from north to south and the average width of about 200 km. The underlying soil of Bangkok area at the uppermost layer is soft silty marine clay, generally referred to as soft Bangkok clay. Figures 3.5 and 3.6 shows the properties of the considered soil profile under Bangkok area (Submaneewong, 2009). As noticed in the Figure, very soft to medium clay layer, approximately 15 m thick, lies under a 1 m thick of weathered crust. This shows a deep deposit of soft soil which may be detrimental for structures during earthquake.

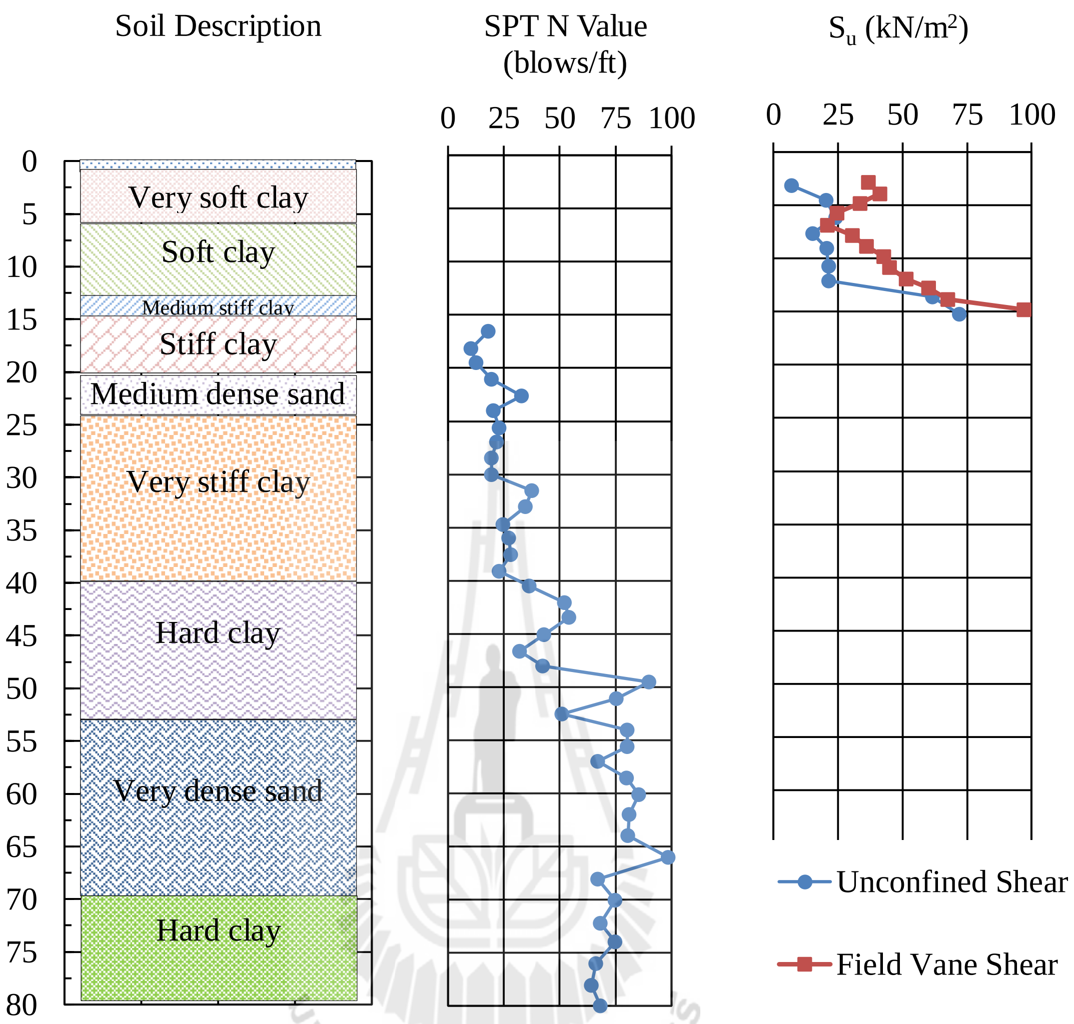


Figure 3.5 Considered soil profile in Bangkok area-SPT N value and shear

(Submaneewong, 2009)

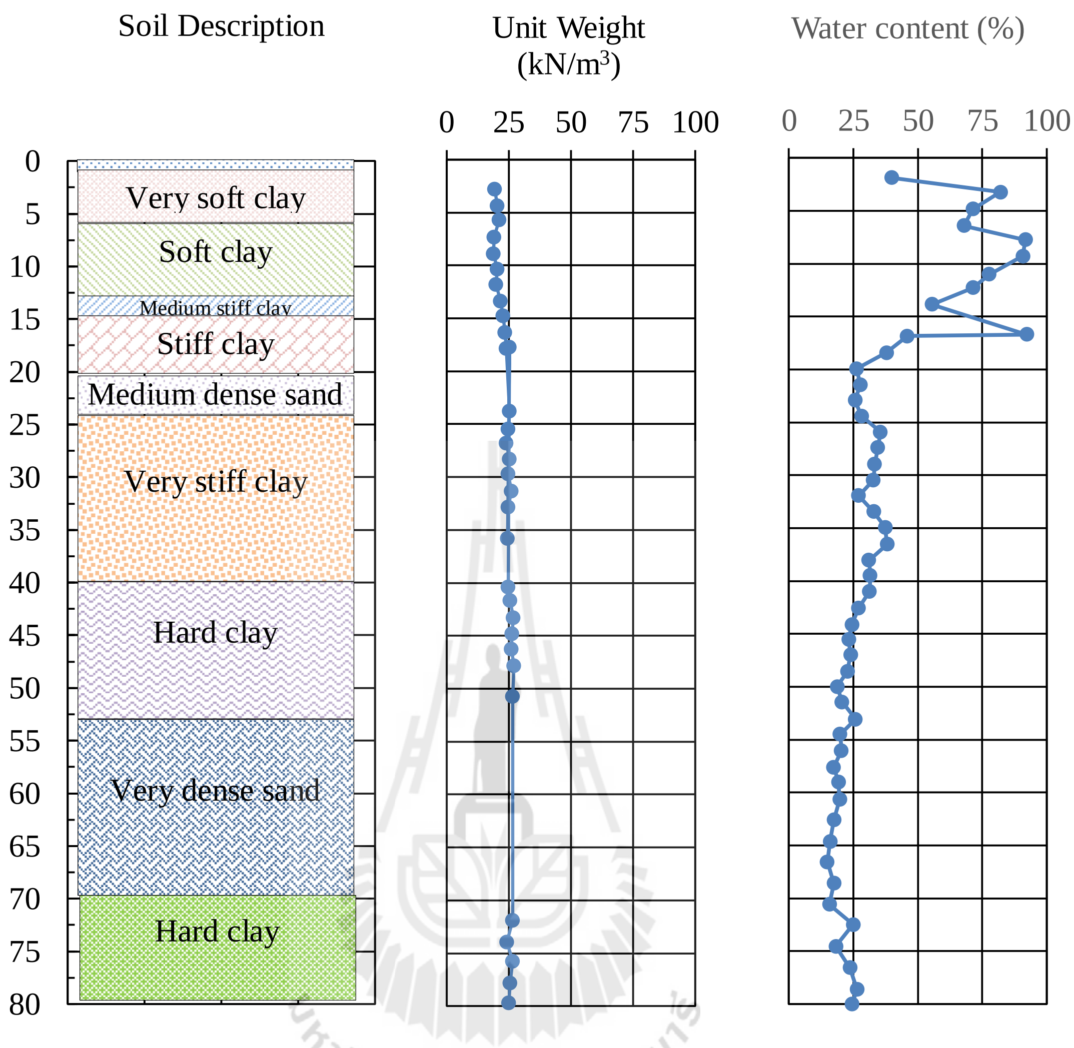


Figure 3.6 Considered soil profile in Bangkok area- Unit weight and Water content.

(Submaneewong, 2009)

Table 3.1 Summary of soil properties for the analysis. (Submaneewong, 2009)

Layer	Soil type	Depth (m)	γ (kN/m^3)	S_u (kN/m^2)	E (kN/m^2)	K_o
1	Crust	0-1	18.00	35	20000	0.70
2	Very soft clay	1-6	16.00	18	9700	0.60
3	Soft clay	6-13	16.50	22	12000	0.60
4	Medium stiff clay	13-15	18.00	35	19000	0.65
5	Stiff clay	15-20.5	17.50	100	110000	0.70
6	Medium dense sand	20.5-23.5	18.00	-	50000	0.80
7	Very stiff clay	23.5-40	18.00	110	120000	0.80
8	Hard clay	40-53	19.00	200	190000	0.80
9	Very dense sand	53-70	19.50	-	120000	0.80

3.4 Evaluation of spring stiffness

3.4.1 Equivalent soil springs

Subgrade reaction model, which was originally proposed by Winkler in 1867, characterizes the soil as a series of unconnected linearly-elastic springs. There exists some disadvantages on using the subgrade reaction to analyze laterally loaded pile. Lack of continuity is one of the drawbacks, while another disadvantage is that the spring modulus of the model is dependent on the size of the foundation (Poulos & Davis, 1980). Despite the disadvantages, subgrade reaction approach is preferred to be used for its simplicity of application. This method can also enable factors such as nonlinearity, variation of soil stiffness with depth, and layering of the soil profile to be taken into account. In Winkler model, the pressure p and the defection y at a point are assumed to be related through a modulus of subgrade reaction, which for horizontal loading, is denoted as k_h. Thus,

$$p = k_h y \tag{3.1}$$

where k_h : denoted modulus of subgrade reaction with the units of (force/length^3)

y : deflection of the pile

p : pressure exerted on pile

Figure 3.7 demonstrates the physical meaning of Equation 3.1.

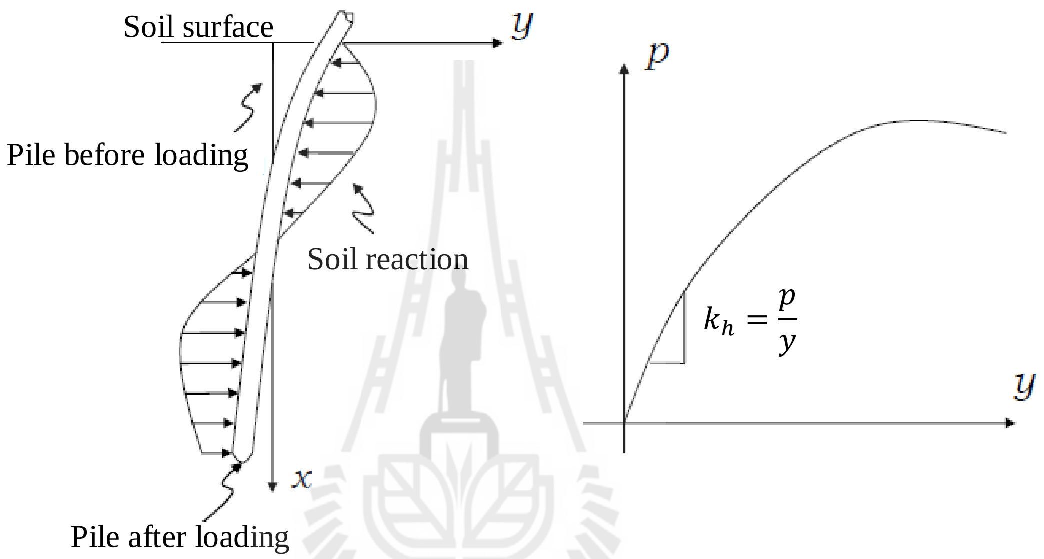


Figure 3.7 Horizontal subgrade reaction of soil. (Poulos & Davis, 1980)

In addition, even though the determination of the modulus of subgrade reaction is not an easy task, a considerable amount of experience has been obtained in using theories to practical problems, and a number of empirical correlations are available for calculating the modulus of subgrade reaction.

Terzaghi (1955) presented the evaluation of modulus of subgrade reaction (k_h) for sand as shown in Equation 3.2.

$$k_h = \frac{n_h x}{B} \tag{3.2}$$

where x : depth below surface (m)

B : width of pile or diameter (m)

n_h : constant of horizontal subgrade reaction determined by Tables 3.2 and 3.3.

Table 3.2 Recommended value for constant of subgrade reaction (Davisson, 1970)

Soil type	n_h (kN/m^3)
Granular	2840-28380
Silt	110-850
Peat	60

Table 3.3 Recommended value for constant of subgrade reaction (Prakash, 1990)

	Loose (kN/m^3)	Moderate (kN/m^3)	Dense (kN/m^3)
Terzaghi (1955)	740-2180	2180-7380	7380-14470
Reese et al. (1974)	5680	17030	35470

Tomlinson and Woodward (2008) investigated the relationship of SPT-N value with relative density of soil by using the graphic given by Terzaghi and Reese as shown in Figure 3.8. Table 3.4 shows the relationship of SPT-N value with various soil types.

Modulus of subgrade reaction for clay was introduced by (Davisson, 1970) as shown in Equation 3.3.

$$k_s = 67\frac{S_u}{B} \tag{3.3}$$

where S_u : undrained shear strength (kN/m^2)

B : width of pile or diameter (m)

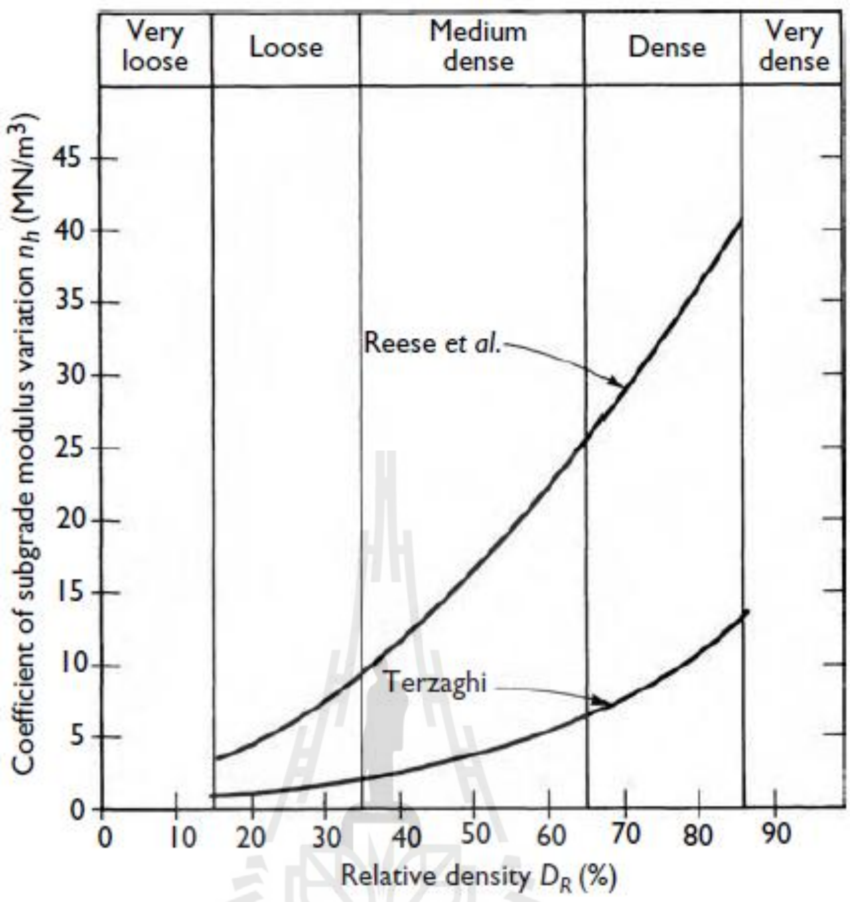


Figure 3.8 Relationship between relative density and coefficient of subgrade reaction. (Tomlinson & Woodward, 2008)

Table 3.4 Relationship between relative density and SPT-N.

SPT-N (blow/ft)	Relative density	Soil property
0-4	0-0.2	Very loose
4-10	0.2-0.4	Loose
10-30	0.4-0.6	Moderate
30-50	0.6-0.8	Hard
>50	0.8-1.0	Very hard

Spring stiffness is determined by using the Equation 3.4 below:

$$K = k_s \times B \times \Delta L \tag{3.4}$$

Where ΔL is the increment along the pile length (m).

By using th e equations and data given above, stiffness of equivalent soil springs was determined and shown in Table 3.5.

Table 3.5 Equivalent soil spring stiffness

Spring No	Soil type	S_u (kN/m^2)	n_h (kN/m^3)	K (kN/m)	Label
0	-	0	0	0	PSpr0
1	Crust	35	-	2345	PSpr1
2	Very soft clay	18	-	1206	PSpr2
3		18	-	1206	
4		18	-	1206	
5		18	-	1206	
6		18	-	1206	
7	Soft clay	22	-	1474	PSpr3
8		22	-	1474	
9		22	-	1474	
10		22	-	1474	
11		22	-	1474	
12		22	-	1474	
13		22	-	1474	
14	Medium stiff clay	35	-	2345	PSpr4
15		35	-	2345	
16	Stiff clay	100	-	6700	PSpr5
17		100	-	6700	
18		100	-	6700	
19		100	-	6700	
20		100	-	6700	
21		100	-	3350	PSpr6
22	Medium dense sand	-	2180	22890	PSpr7
23		-	2180	47960	PSpr8
24		-	2180	50140	PSpr9
25		-	2180	25615	PSpr10
26	Very stiff clay	110	-	3685	PSpr11
27		110	-	7370	PSpr12
28		110	-	7370	
29		110	-	7370	
30		110	-	7370	
31		110	-	7370	

Table 3.5 Equivalent soil spring stiffness (Continued)

32	Very stiff clay	110	-	7370	
33		110	-	7370	
34		110	-	7370	
35		110	-	7370	
36		110	-	7370	
37		110	-	7370	
38		110	-	7370	
39		110	-	7370	
40		110	-	7370	
41		110	-	7370	
42		110	-	7370	
43	Hard clay	200	-	13400	PSpr13
44		200	-	13400	
45		200	-	13400	
46		200	-	13400	
47		200	-	13400	
48		200	-	13400	
49		200	-	13400	
50		200	-	13400	
51		200	-	13400	
52		200	-	13400	
53		200	-	13400	
54		200	-	13400	
55		200	-	13400	
56	Very dense sand	-	7380	398520	PSpr14
57		-	7380	405900	PSpr15

3.4.2 Lateral pile capacity analysis

Earthquake creates force on structure in the form of lateral load which transfers from superstructure to its foundation. While pile foundation was used in this study, the lateral pile capacity was required to be analyzed. Figure 3.9 shows the simplified pile load test in which horizontal force was applied at the pile head. The description and records of the tested pile for static lateral load test is given in Table 3.6

(Submaneewong, 2009). In order to analyze the lateral capacity of the pile, complete cross-section of the pile is required. Figure 3.10 shows cross section the studied reinforced concrete pile.

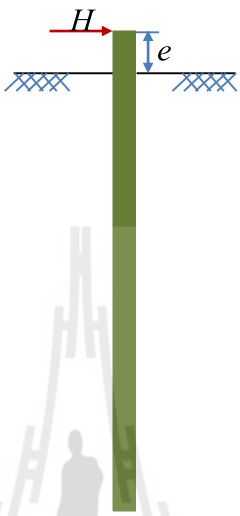


Figure 3.9 Simplified pile load test

Table 3.6 The description and records of the test pile for static lateral load test (Submaneewong, 2009)

No.	Type	ϕ (m)	Tip (m)	f_c' (MPa)	f_y (MPa)	1st Case
T4P13 & T4P14	Bored	1.65	-55.52	27.468	490.5	44DB32

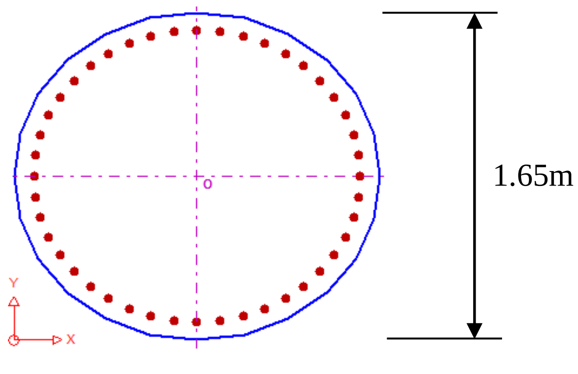


Figure 3.10 Cross section of the pile

Within the given structural parameters of the pile, its yield moment could be calculated and is equal to 9534.333 kNm. Broms (1964) proposed the method to calculate the lateral capacity of the restrained and free-head piles in both cohesive and cohesiveless soils. The capacity of the free-head pile in cohesive soil was applied in this study in which Equation (3.5) shows the ultimate lateral capacity of the pile respected to its yield moment.

$$M_y = H_u \left(\mathrm{e} + 1.5B + \frac{0.5H_u}{9S_u B} \right) \tag{3.5}$$

where M_y : yield moment of the pile

H_u : ultimate lateral capacity of the pile

B : diameter of the pile

e : the distance from the point of applied load to ground surface and e was taken to be 0.65m.

From the calculated value of M_y = 9534.333 kNm, the ultimate lateral capacity of the pile is equal 1571.927kN. This value of H_u = 1571.92kN was divided by a factor of safety SF = 2.5 in order to determine the allowable lateral load in which the pile can resist. From the calculation, H_u = 628.771 kN.

3.4.3 Calibration of spring stiffness

After stiffness of springs at each layer was computed, a series of spring was modeled along a pile. Pushover analysis was performed to observe the lateral response of the pile by using a series of lateral load. The response of the laterally loaded pile was then compared with the test data given in the literature. The method used is called p-y method adopted and modified from Winkler principle by assuming that surrounding ground as a series of non-linear spring and pile is an elastic beam.

However, in this analysis, the springs were limited by their linear elastic property in order to simplify the analysis and to meet the limit to the analysis program used. Once the analysis was done, load-displacement curve of the response of the pile was created and compared with the lateral load test data as shown in Figure 3.11. Since the springs were considered to be linear, the result of the pushover analysis of the pile was in linear form. To get an appropriate response of the soil subjected to lateral loading, the allowable lateral load of the pile was used as the reference point for creating linear response of the pile. The allowable lateral load was plotted on the Load axis and a projection line parallel to the Displacement axis was drawn. The intersection between projection line and the curve of the lateral load test was the reference point from where the allowable load line was drawn. This line was used to represent the soil response with which the analysis model was fitted. However as seen Figure 3.12, the lateral load has big influence on deflection of the pile to the depth of the soft clay and has very little effect to the pile in medium stiff clay. Hence only the stiffness of the springs in very soft to soft clay layers were varied. In addition this study observed also the sensitivity of the seismic response of structure to variable stiffness of the soil. The analysis was done by changing the factor to be multiplied with S_u in the empirical equation of modulus of subgrade reaction leading to the calculation of equivalent spring stiffness as shown in Equations (3.3) and (3.4).

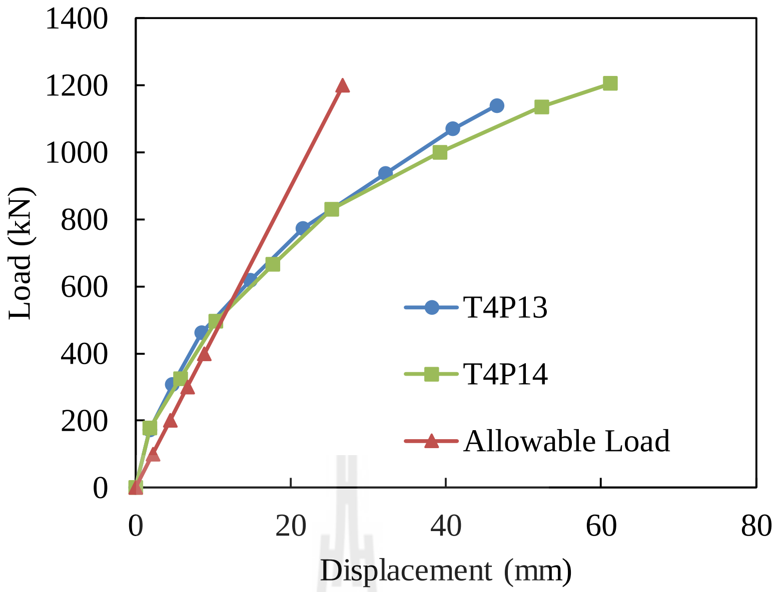


Figure 3.11 Lateral load test data of bored pile with diameter of 1.65m (T4P13) and 2.0 m (T4P14). (Submaneewong, 2009)

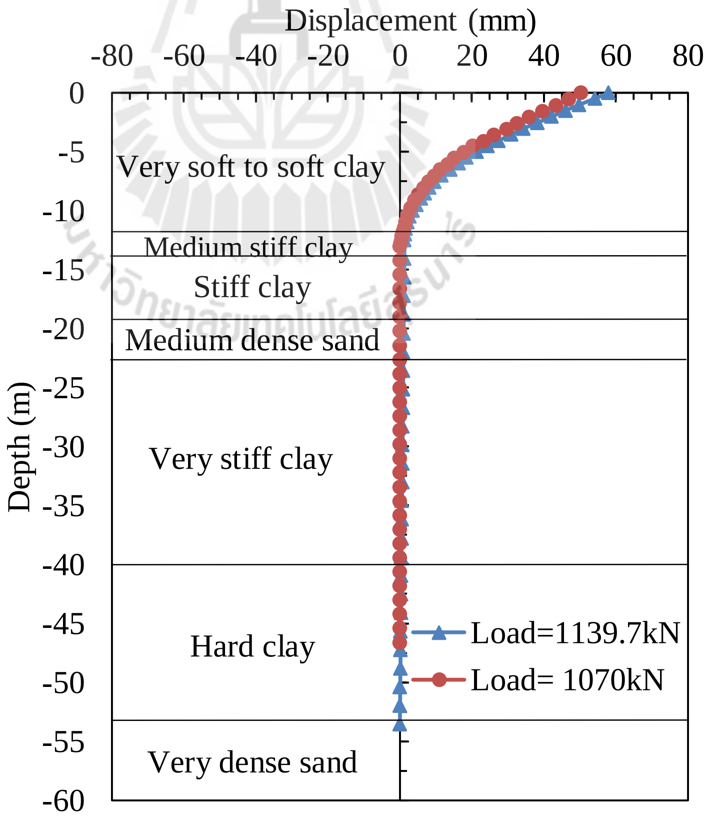


Figure 3.12 Lateral deflection profiles of tested bored pile. (Submaneewong, 2009)

3.1.3 Vertical spring stiffness

The vertical spring stiffness of the piles was that created by the elasticity of the concrete used to cast to the piles. The concrete exhibits its elasticity which works as its stiffness while the analysis is performed in the elastic range. Hence within the frame of this study, the stiffness is expressed in Equation (3.6).

$$K_v = \frac{EA}{L} = 367422.283\ kN / m \tag{3.6}$$

CHAPTER IV

EFFECTS OF SOIL-STRUCTURE INTERACTION ON SEISMIC RESPONSE OF BUILDINGS

4.1 Introduction

From day to day, buildings are constructed everywhere. At the same, some are also demolished and damaged due to its long service age, excess loading, natural disaster such typhoon or earthquake. Hence engineers are doing challenging work in order to fight against those disastrous phenomena by designing a sustainable structures after being exposed to those unexpected extreme loads while maintaining reasonable cost of building. One of the most detrimental disaster is earthquake which always hits most parts of the world and causes massive damage and casualties.

Bangkok, the capital of Thailand, is still classified as earthquake prone area even though it locates at a remote distance from seismic sources (Warnitchai et al., 2000). The case of Bangkok capital is similar to that found in Mexico City (regarding to geological situation) during the 1985 Michoacan earthquake which claimed thousands of people's lives and caused serious damage to the Greater Mexico City area. Much of the destruction was due to significant amplification of earthquake ground motions by thick soft surficial deposits in the downtown area of the City (H. B. Seed, 1987). While Bangkok is reported to situate on also such soft deposits with depth ranging from 10 to 18 m, the amplification of earthquake ground motions can also occur and may cause enormous destruction of the city.

It is therefore of a great attention for engineers especially structural designer while performing structural design. In conventional structural analysis and design, structural models with fixed bases are often adopted and performed in design software. However, it may not be the case while the structures are built on their corresponding soil profile since soil provides more or less flexibility to the foundations as well as the structures. The flexibility is relatively low if the soil is soft deposit such those under Mexico City or Bangkok metropolitan area.

Therefore this study have been done with the aim of studying the effect of the underlying soil on its foundation and superstructure. Two models with different support conditions have been studied and their responses including modal periods, story displacement, story drift, story shear and overturning moment have been compared. In addition, the investigation of seismic response of the structure with respect to different soil stiffness has also been introduced.

4.2 Horizontal response of the pile

Figure 4.1 shows the model of the analyzed pile with its lateral springs represented soil resistance.

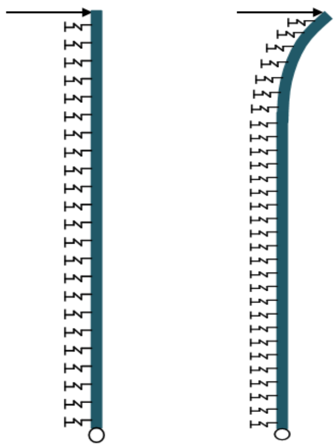


Figure 4.1 Original and deformed shapes of tested pile

The model of the pushover analysis of the pile was done by using a conventional design software, ETABS. The dimensions of the pile was set to be the same as that tested pile in the field. The forces to be applied on pile head were taken from the values given in the pile load test. After running the analysis, lateral response of the pile can be plotted as shown in Figure 4.2.

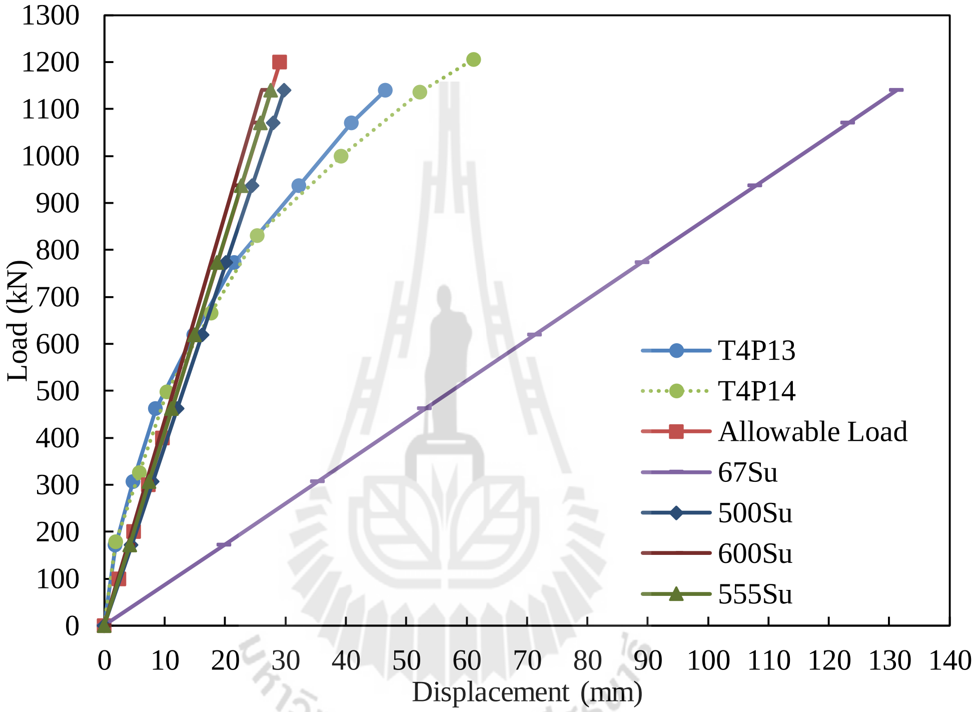


Figure 4.2 Lateral response of the pile.

Based on Equations (3.3), the spring stiffness can be referred to as the function of the undrained shear strength (S_u). Whereas the recommended factor is taken to be 67 as shown in Equation (3.3).

Equation (3.3) $k_s = 67\frac{S_u}{B}$

However as seen in Figure 4.2, the response of the pile is far different from that of test data. This is because, with the recommended factor, the stiffness calculated cannot well represent the characteristic of the soil due to high flexibility of the soil. Hence trial values were assumed and analyses were done to check for a corresponding tested pile's performance. After doing some trial tests, the factor of 555 is the best fit to represent pile's response. Therefore the new stiffness of the springs were calculated by using the factor of 555. On the other hand Equation (3.3) can be modified as follow:

$$k_s = 555\frac{S_u}{B} \tag{3}$$

Table 4.1 summarizes the new stiffness for the analysis. However only the spring stiffness calculated for very soft to soft clay layer were subjected to be changed since, according to the deflection profile of the pile from pile load test, only the upper part of the pile deflected because the stiffness of soft clay is very low compared to other layer.

Table 4.1 New spring stiffness for the analysis.

Spring No	Soil type	S_u (kN/m^2)	n_h (kN/m^3)	K (kN/m)	Label
0	-	0	0	0	PSpr0
1	Crust	35	-	19425	PSpr1
2	Very soft clay	18	-	9990	PSpr2
3					
4					
5					
6					
7	Soft clay	22	-	12210	PSpr3
8					
9					
10					
11					
12					
13					
14	Medium stiff clay	35	-	2345	PSpr4
15		35	-	2345	
16	Stiff clay	100	-	6700	PSpr5

Table 4.1 New spring stiffness for the analysis (Continued)

17	Stiff clay	100	-	6700	
18		100	-	6700	
19		100	-	6700	
20		100	-	6700	
21		100	-	3350	PSpr6
22	Medium dense sand	-	2180	22890	PSpr7
23		-	2180	47960	PSpr8
24		-	2180	50140	PSpr9
25		-	2180	25615	PSpr10
26	Very stiff clay	110	-	3685	PSpr11
27		110	-	7370	PSpr12
28		110	-	7370	
29		110	-	7370	
30		110	-	7370	
31		110	-	7370	
32		110	-	7370	
33		110	-	7370	
34		110	-	7370	
35		110	-	7370	
36		110	-	7370	
37		110	-	7370	
38		110	-	7370	
39		110	-	7370	
40		110	-	7370	
41		110	-	7370	
42		110	-	7370	
43	Hard clay	200	-	13400	PSpr13
44		200	-	13400	
45		200	-	13400	
46		200	-	13400	
47		200	-	13400	
48		200	-	13400	
49		200	-	13400	
50		200	-	13400	
51		200	-	13400	
52		200	-	13400	
53		200	-	13400	

Table 4.1 New spring stiffness for the analysis (Continued)

54	Hard clay	200	-	13400	
55		200	-	13400	
56	Very dense sand	-	7380	398520	PSpr14
57		-	7380	405900	PSpr15

4.3 Complete model for analysis

The complete 3D model of the studied structure with horizontal springs along the piles is shown in Figure 4.3, while the 3D model with fixed support is illustrated in Figure 4.4. Parameters to be investigated are modal analysis of the structure, story displacement, story drift, story shear, and overturning moment.

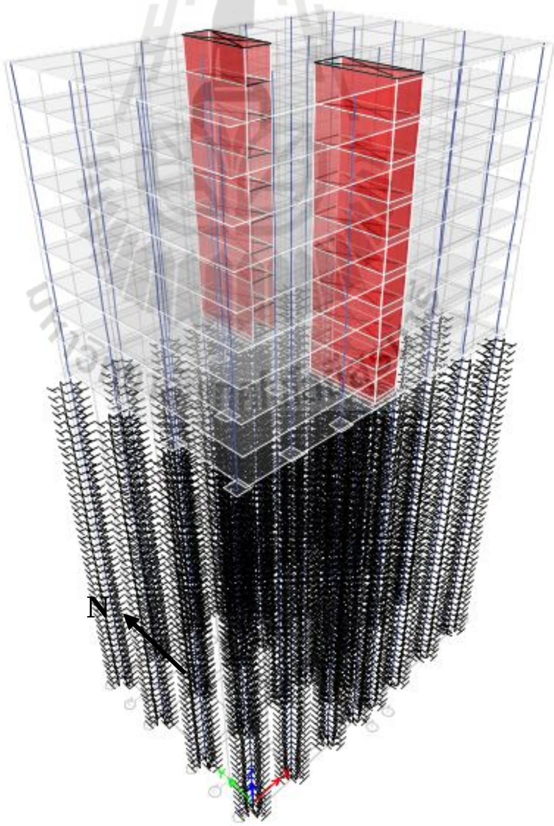


Figure 4.3 Complete 3D model of the studied building with pile foundation

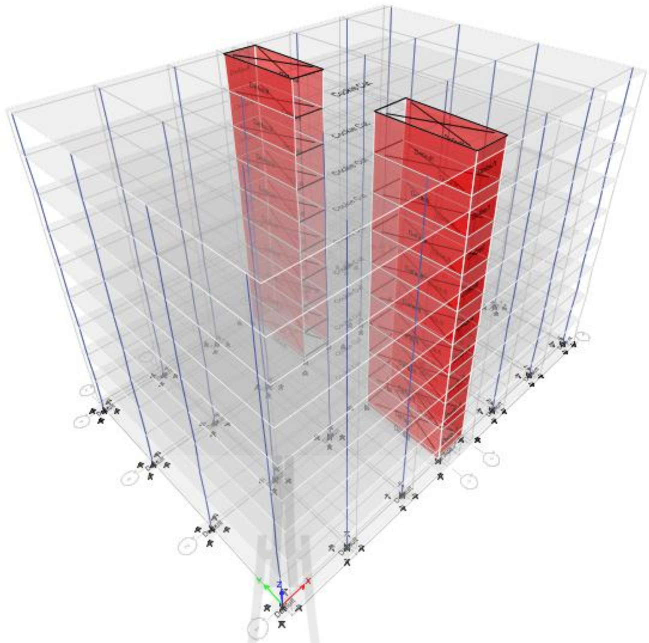


Figure 4.4 Complete 3D model of the studied building with fixed support

4.4 Modal analysis

Modal analysis is the process of determining the dynamic characteristics of a system in the form of natural frequencies, damping factors and mode shapes. Table 4.1 shows the comparison between the modal periods, circular frequencies, and Eigenvalue of the structures with fixed and flexible supports where mode expresses the possible shapes that the structure vibrates. Based on the study results, there is an increase in periods of the structure with consideration of soil-structure interaction.

Table 4.2 Comparison of modal periods and frequencies of fixed and SSI models.

Mode	Period (sec)		Circular Freq. (rad/sec)		Eigenvalue (rad^2/sec^2)	
	Fixed	SSI	Fixed	SSI	Fixed	SSI
1	0.865	1.557	7.264	4.034	52.771	16.276
2	0.758	1.378	8.285	4.561	68.646	20.801
3	0.491	1.239	12.785	5.072	163.448	25.727
4	0.229	0.362	27.466	17.366	754.359	301.576
5	0.210	0.357	29.987	17.607	899.199	310.014
6	0.115	0.317	54.420	19.835	2961.540	393.445
7	0.111	0.200	56.655	31.484	3209.740	991.245
8	0.111	0.186	56.714	33.743	3216.500	1138.570
9	0.080	0.117	78.388	53.603	6144.738	2873.241
10	0.079	0.116	79.514	54.385	6322.489	2957.754
11	0.065	0.111	96.349	56.575	9283.200	3200.754
12	0.064	0.093	98.439	67.609	9690.266	4570.939

The periods of the structure with SSI model is about 2 times of the periods of the structure with fixed base model. This is due to the flexibility of the soil provided to the pile foundation through the form of spring stiffness. Figure 4.5 is the plot of the first 3 structural periods of both fixed and SSI support on the graph of response spectrum. The dash lines represent the periods of the fixed base, while the dot lines represent the periods of the SSI base. According to the response spectrum acceleration shown in Figure 3.4, these higher periods locates their corresponding spectral acceleration near to the highest spectral acceleration which tends to create higher response of the structure. Comparing to conventional spectral acceleration, these higher periods are obviously at the decreasing branch of the acceleration.

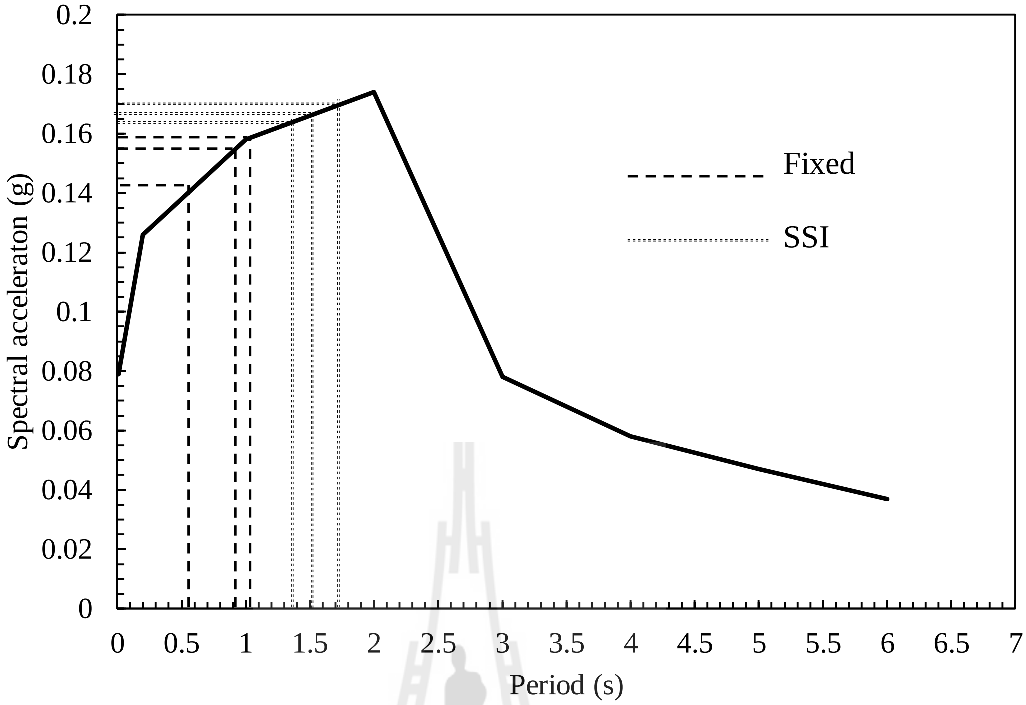


Figure 4.5 Bangkok Response Spectrum

Moreover the result of the variation of the stiffness of the springs as seen in Figure 4.6 reveals that the period of the structure decreases with the increase of the stiffness of the support. The stiffness of the support was done by increasing the factor of undrained shear strength which leads to higher equivalent spring stiffness. Because of the high stiffness of the springs, rigidity of the supports increases and tends to restrain the structure from vibrating. Hence short periods are observed.

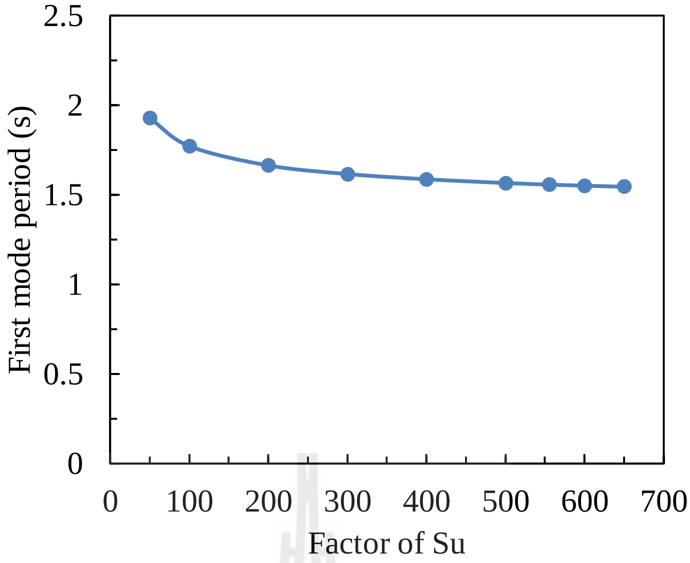


Figure 4.6 Variation of structural periods with spring stiffness

4.5 Story displacements and drifts

Lateral displacement and drift are all important design factors to be considered in the design process. Design for drift and lateral stability is an issue that should be addressed in the early stages of the design development (Naeim, 2001). There are 3 significant perspective for which the lateral displacement or drift of a structural system under lateral loads such as wind or earthquake forces: (1) structural stability; (2) architectural integrity and potential damage to various non-structural components; and (3) human comfort during, and after, the building experiences these motions. Excessive and uncontrolled lateral displacement can lead to structural problems. Scholl (1975) revealed by doing empirical observations and theoretical dynamic response studies that there is a strong correlation between the magnitude of inter story drift and building damage potential. Based on this potential problem, the requirements of drift control are including also in the design provisions in most building codes. Figure 4.5 shows the determination of drift by ASCE standard.

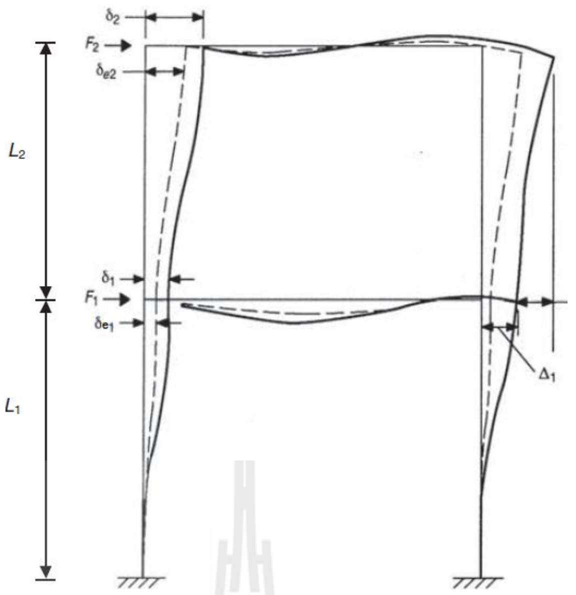


Figure 4.7 Story drift determination (ASCE7-05, 2006)

The design story drift (Δ_i) shall be computed as the difference of the deflections at the centers of mass at the top and bottom of the story under consideration (ASCE7-05, 2006). The design drift as determined in Equation (4.3) shall not exceed the allowable story drift (Δ_a) which is taken to be 0.015 h_{si} (where h_{si} is the height of the considered story) as defined by ASCE standard for this considered structure. However in many cases, the story drift ratio (Δ_i / h_{si}) which is the ratio of the design story drift with the corresponding story height (h_{si}) is preferably used and expressed in no unit.

$$\Delta_i = \frac{(\delta_{ei} - \delta_{ei-1})\mathrm{C}_d}{I_E} \leq \Delta_a \tag{4.3}$$

where δ_{ei} and δ_{ei-1} are elastic displacements of two adjacent stories.

C_d and I_E are the deflection amplification factor and the importance factor determined in ASCE, respectively.

Hence the responses of the studied models in term of story displacement and story drift were compared in order to check for critical state while SSI is taken into consideration. It is revealed from the results that the maximum story displacements increase significantly for the structure with SSI in both directions as illustrated in Figures 4.8 and 4.9. These changes in story displacements may lead to dramatically changes in internal forces of elements in the structure. It may possibly cause the instability or damage to the structure. On the other hand, the displacements of the first stories in both direction reveal a significant different between fixed and SSI supports. While the structure with fixed support exhibits zero displacement at first story, the structure with SSI support draws some 3.5 mm in both E-W and N-S direction. However there is a big difference of displacement from 1st story to the top one. In E-W direction, the difference is about 4 times comparing between fixed and SSI models. Where as in N-S direction, 8-time difference is observed comparing between fixed and SSI models. It is obvious that the increasing displacement and drift are caused by the flexibility of the support which is, in this case, the foundation surrounded by very soft to soft clay layers. In E-W direction, columns play a role of lateral resisting elements besides their vertical load support and reinforced concrete-core walls help resist lateral load in addition to columns in N-S direction. It is reason why displacements in N-S direction are smaller than that in E-W direction. This reveals the importance of lateral structural load resisting elements in maintain safe performance of overall structure.

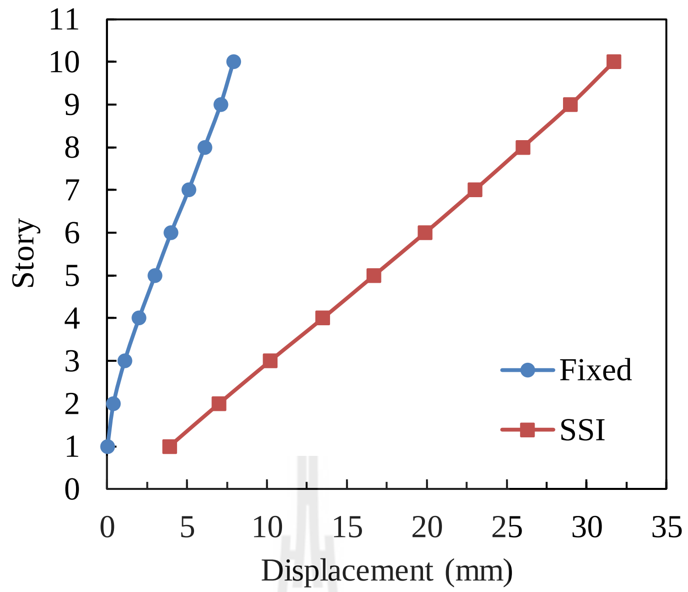


Figure 4.8 Story displacements of fixed base and flexible base structures – E-W direction

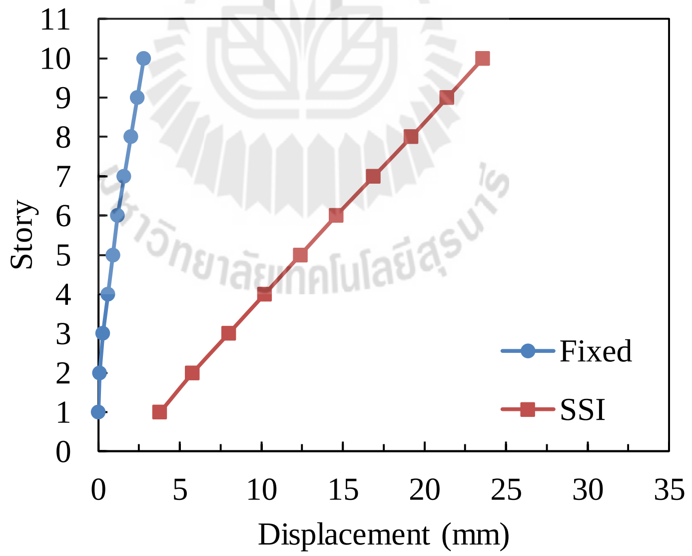


Figure 4.9 Story displacements of fixed base and flexible base structures- N-S direction

It is noticed similarly for the inter-story drifts. The values of drift ratio shown in Figures 4.10 and 4.11 indicate that drift ratios of SSI-support structure are higher than that of fixed-support structure. The maximum drift in E-W direction is about 0.395% and the maximum drift in N-S direction is about 0.272%.

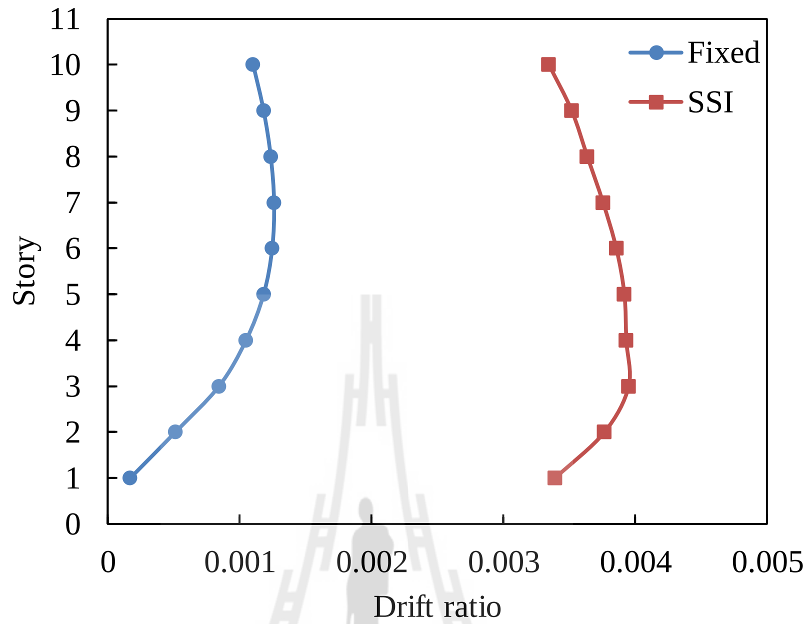


Figure 4.10 Story drifts of fixed base and flexible base structures – E-W direction

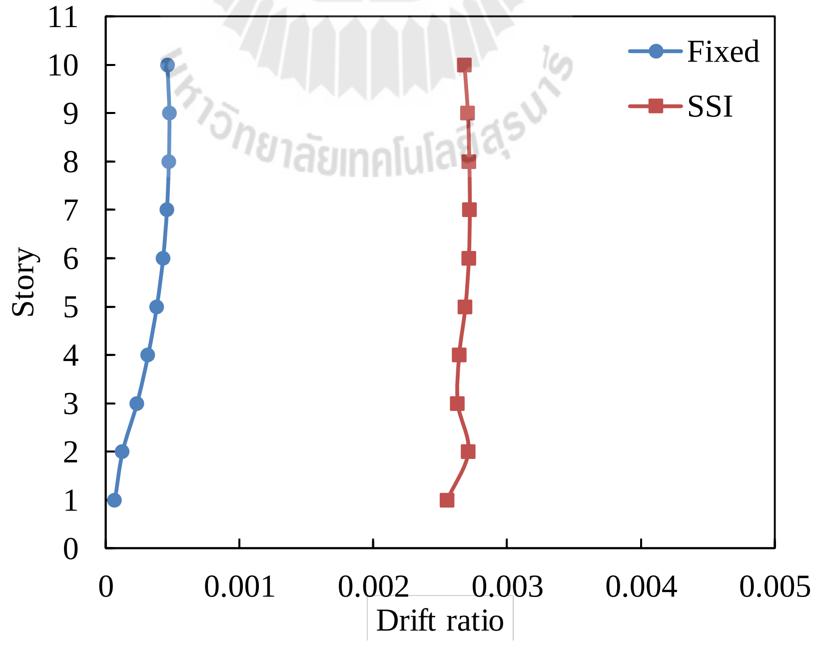


Figure 4.11 Story drifts of fixed and flexible base structures – N-S direction

However the maximum drift ratios in both directions are smaller than the limit or allowable drift ratio which is equal to 0.015 specified in ASCE standard. Even though the maximum analyzed drift ratios are smaller than that of the standard, the damage may occur in structural elements since this change in drift causes the change of internal forces of the structural elements. Moreover it may pose to non-structural elements. As indicated by Naeim (2001), larger drift can cause damage to non-structural elements as well as affect human behavior and psychology (Ohta & Omote, 1977).

Figures 4.12 and 4.13 show the variation of the maximum story displacement and drift ratio respectively with variation of spring stiffness. The maximum story displacement are those recorded at the top story. The same as the period of the structure, story displacement and drift ratio in both direction decrease with the increase of the spring stiffness. The values of displacements indicate high fluctuation at low spring stiffness. However at larger stiffness, the displacements decrease slightly and tend to be constant. Whereas the drift ratio does not show much difference in both directions.

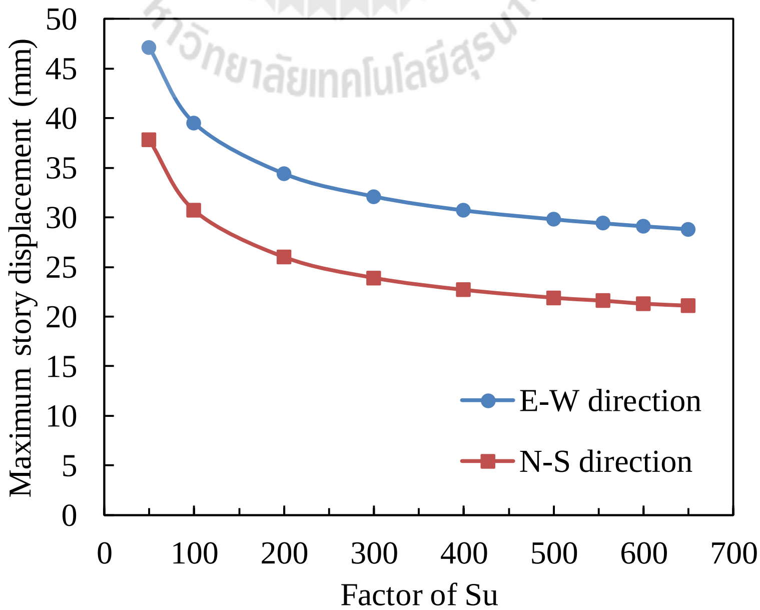


Figure 4.12 Variation of maximum story displacements with spring stiffness

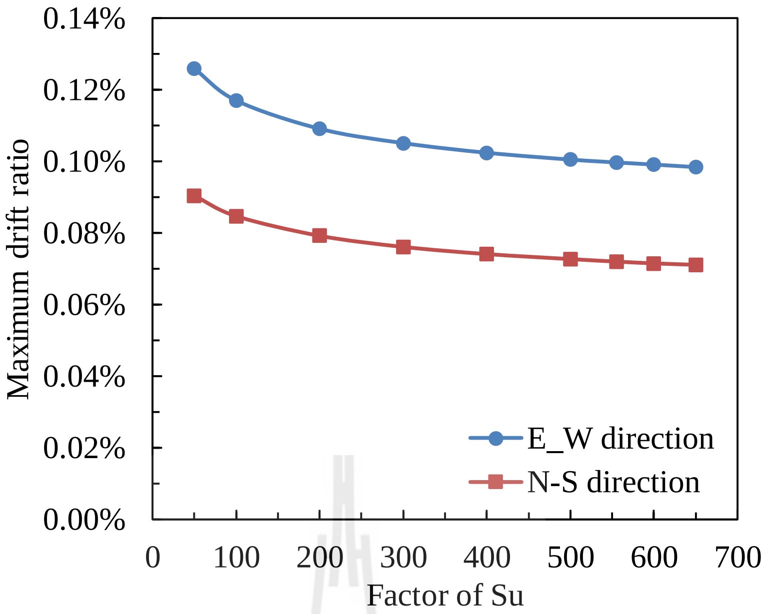


Figure 4.13 Variation of maximum drift ratio with spring stiffness

4.6 Story shears

As addressed in the modal response about the increase of the period due to the effect of soil on the foundation, this increased period have dramatically alters value of force occurring in the structural elements. Story shears of the structure with flexible support are higher than that of fixed-base structure in all considered directions. In E-W direction (Figure 4.14), the difference is about 1.4 time and SSI model exhibits higher value. Moreover the difference goes to about 1.6 time in N-S direction as shown in Figure 4.15.

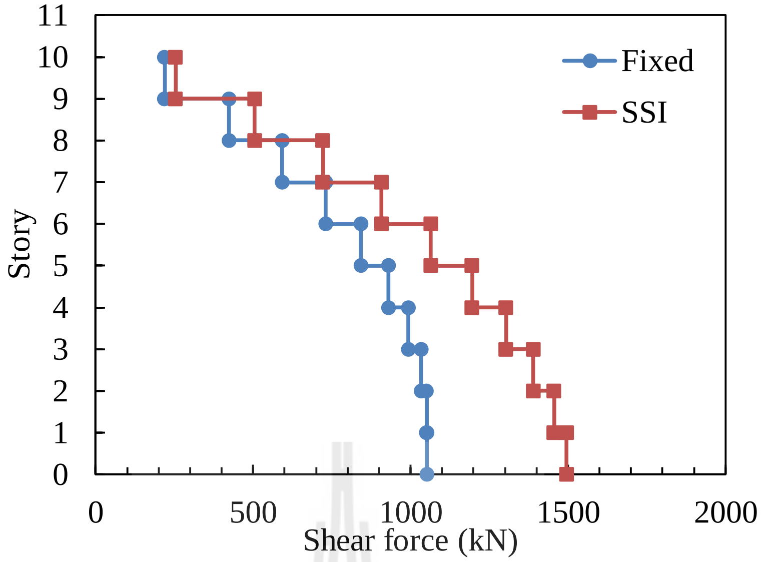


Figure 4.14 Story shears of fixed and flexible base structures – E-W direction

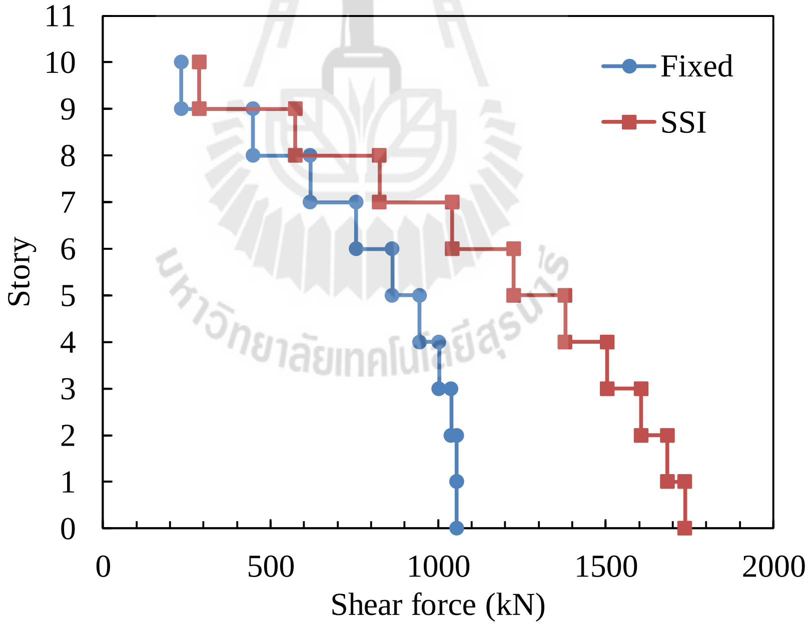


Figure 4.15 Story shears of fixed and flexible base structures – N-S direction

Figure 4.16 shows the plot of maximum shear forces in term of spring stiffness presented in the force of factor of S_u as in Equation (3.3). These maximum shears are a series of shear forces at the footing level where the superstructure and substructure are connected. Due to the flexibility of the soil, shear forces of the structure should be decreased. However in the application of the response spectrum of this study, low flexibility of the soil causes period lengthening which leads to higher spectral acceleration in the analysis calculation. The curves of shear forces in both direction flatten out while the spring stiffness gets higher. This is reasonable since when the rigidity of the support is big enough, the support can be considered as fixed and period of the structure obviously decreases and shear force of fixed-base structure will be observed. Even though lower spring stiffness creates higher shear forces, maximum allowable shear force specified in the code of practice should be considered for safe performance of the structure.

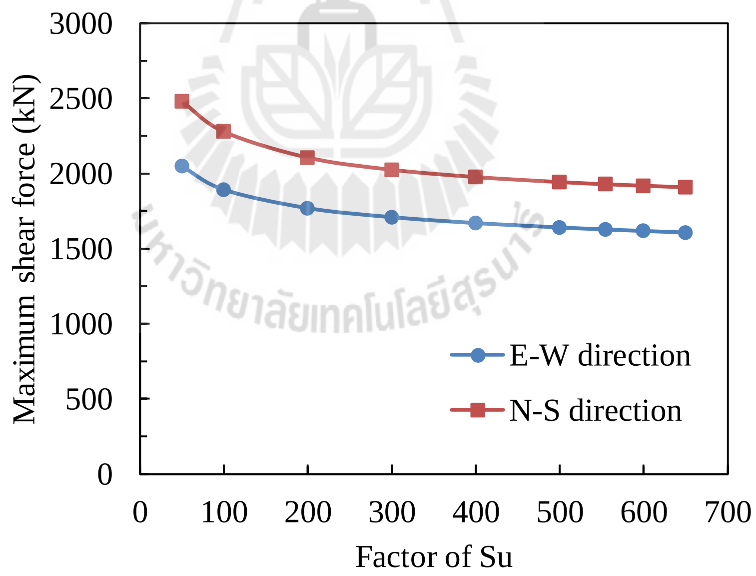


Figure 4.16 Variation of maximum shear force with spring stiffness

4.7 Overturning moments

In addition to increase of shear force, overturning moments also increase to significant values. The moments at the tops of both structural models are equally zero.

The values tend to separate from each other to higher values until it reaches its maximum value which is at the base of the structure. The maximum values as seen in Figures 4.17 and 4.18 are about 1.3 time and 1.5 time differences in E-W and N-S directions when SSI is taken into account, respectively.

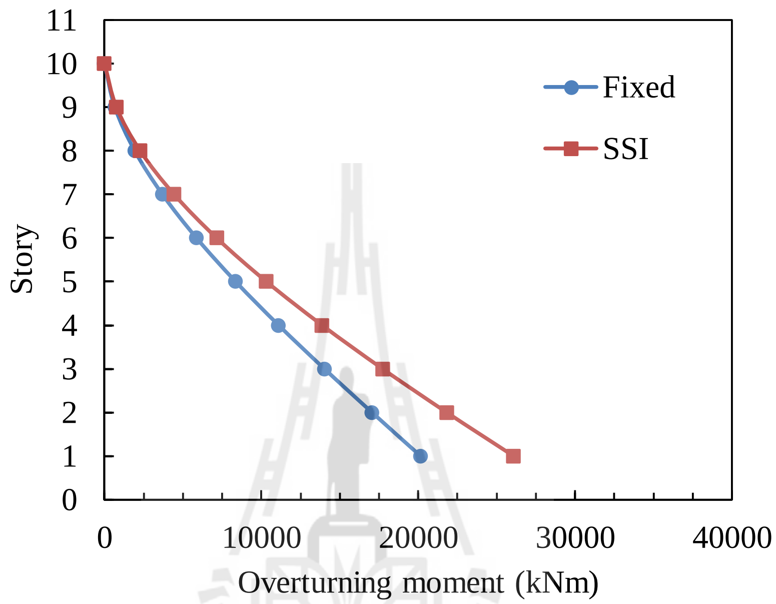


Figure 4.17 Overturning moments of fixed base and SSI models – E-W direction

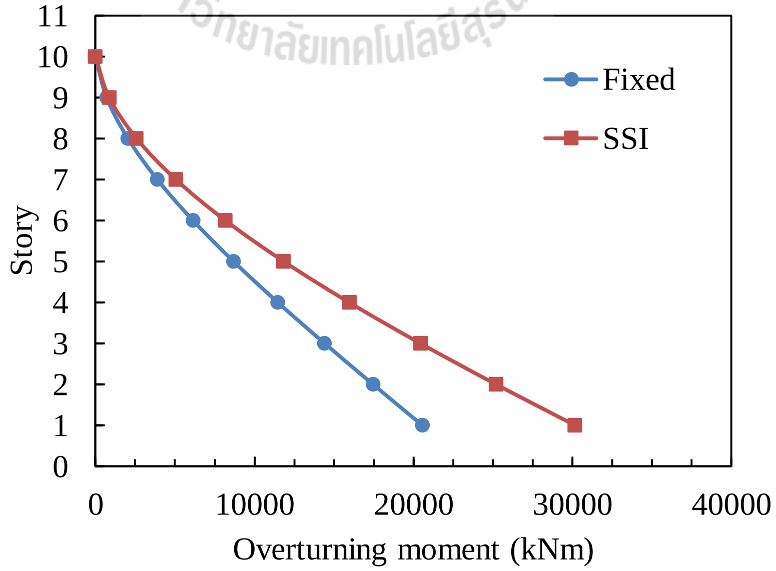


Figure 4.18 Overturning moments of fixed base and SSI models–N-S direction

While the shear forces change, the moments also change. The same other response parameters, the moments of the building exhibit higher value at low stiffness of the spring. The curves of the moments in both direction flatten out and the value of the moment converges to a constant when the spring stiffness becomes higher.

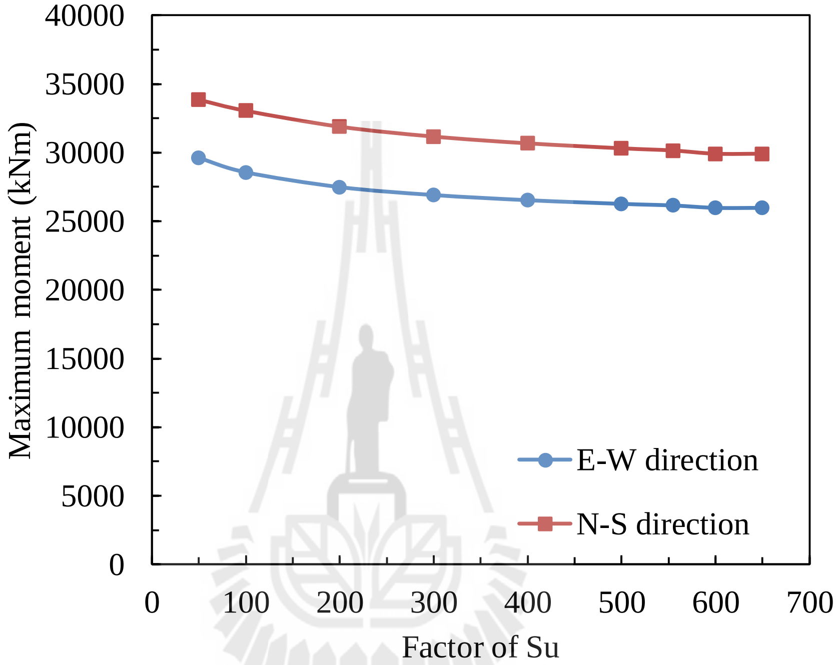


Figure 4.19 Variation of the maximum moments with spring stiffness

4.8 Response at footing level

Pile cap which is located at footing level and the connection between the superstructure and substructure, distributes the loads of the building to the piles. Overall loads such as axial loads, lateral loads, moment, direct shear, punching shear occur and exert onto the pile caps supporting the structure. The analysis and design of the pile caps must be done and checked properly since there are many kind of loads as well as the combination of internal forces acting on it. In addition the connections between the pile caps to piles and columns are crucial. This part aims to check for the forces happening at footing level and the upper part of the piles of both fixed and SSI models. Figure 4.20 shows an elevation view of the studied model. As seen in the Figure, under the story 1, there are stump columns which are linked to the piles supported by series of lateral springs via pile caps. Figure 4.21 shows a typical pile cap layout.

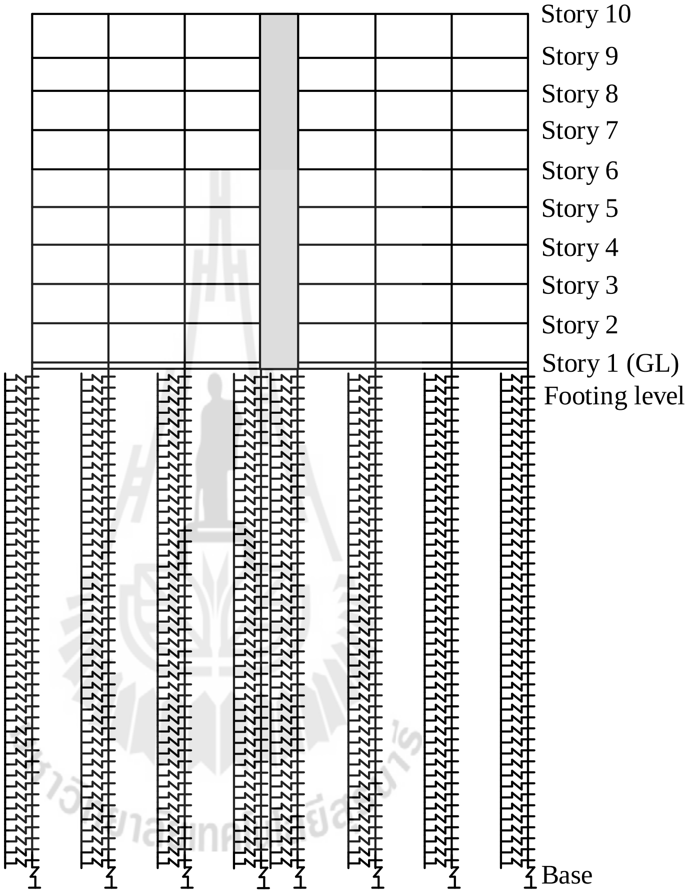


Figure 4.20 An elevation view of the studied model

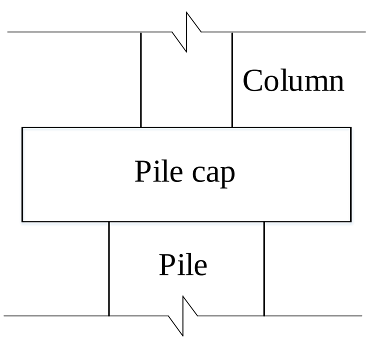
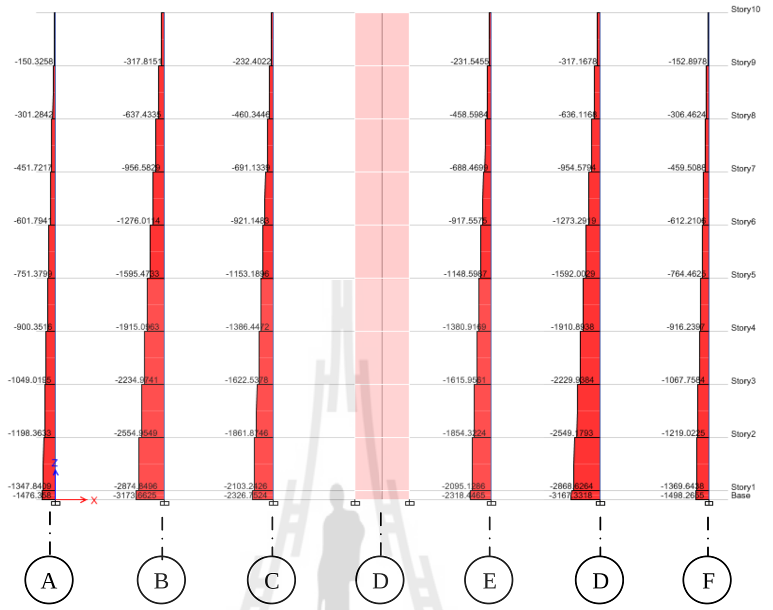


Figure 4.21 Typical pile cap

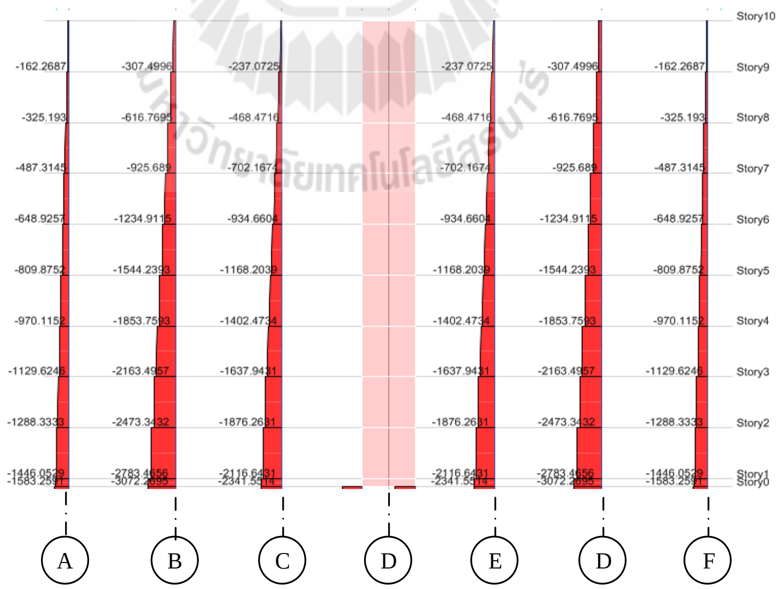
Based on the internal force diagrams shown from Figures 4.22 to 4.27, it indicates that the internal forces of the structural elements change when SSI interaction is taken into account. Excluding axial force, shear force and moment are increased. Figures 4.22 and 4.25 show the axial force diagrams in Elevation 1 and B, respectively. The axial force does not exhibit any change in value because it is not affected by the lateral force which, in this study, is the earthquake force. Obviously shear forces shown in Figures 4.23 and 4.26 in Elevation 1 and B, respectively, indicates that there is an increase in shear force while SSI is taken into consideration. The increase of the force is caused by the period lengthening of the structure which locates the response spectral acceleration at higher value compared to fixed base model. In addition some structural elements are subjected to change in working function or additional working function such additional compression or tension force occurs in structural elements. Also lateral load from earthquake excitation causes higher moments in the structural elements, on pile caps and piles (Figures 4.24 and 4.27). Pile caps are subjected to additional forces which are transferred to piles (Figures 4.23b, 4.24b, 4.26b, and 4.27b). However only the upper part of the pile are influenced by the lateral force. It complies with the lateral pile load test in which the upper part of the pile deflects while lower part of the pile started from medium stiff clay are less influenced by the lateral load. The changing of these internal forces makes significance in structural design of each member including structural piles and pile caps particularly the joints connected pile with pile cap and pile caps with columns. Therefore the forces created by the interaction of soil and structure provide considerable change in the design process which should be paid attention to by engineers.

4.8.1 Internal force diagrams of structural elements in Elevation 1

Figures 4.22 shows axial force diagrams in Elevation 1.



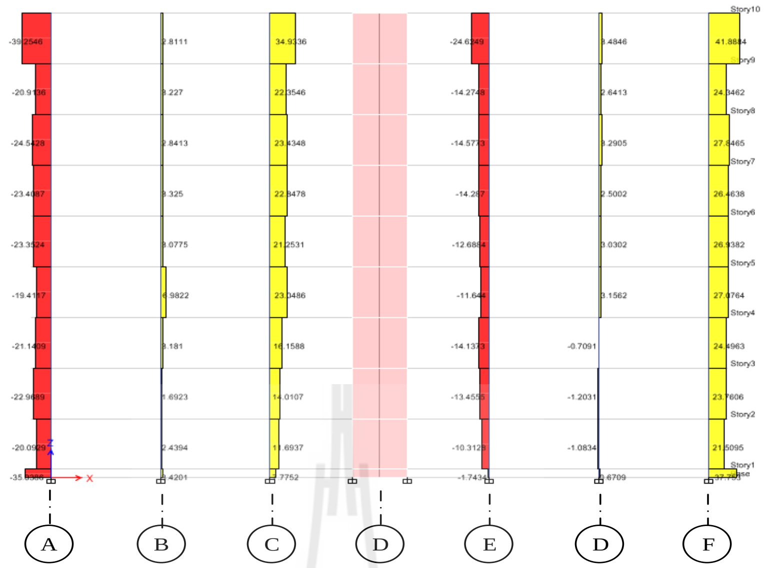
(a) Fixed base model



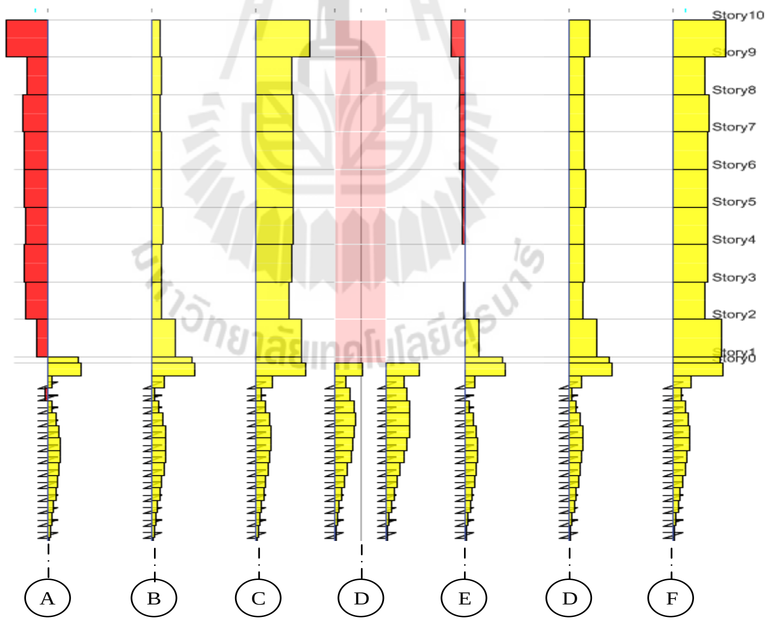
(b) SSI Model

Figure 4.22 Axial force diagrams in Elevation 1 (a) fixed base model; (b) SSI model

Figure 4.23 shows shear force diagrams of in Elevation 1.



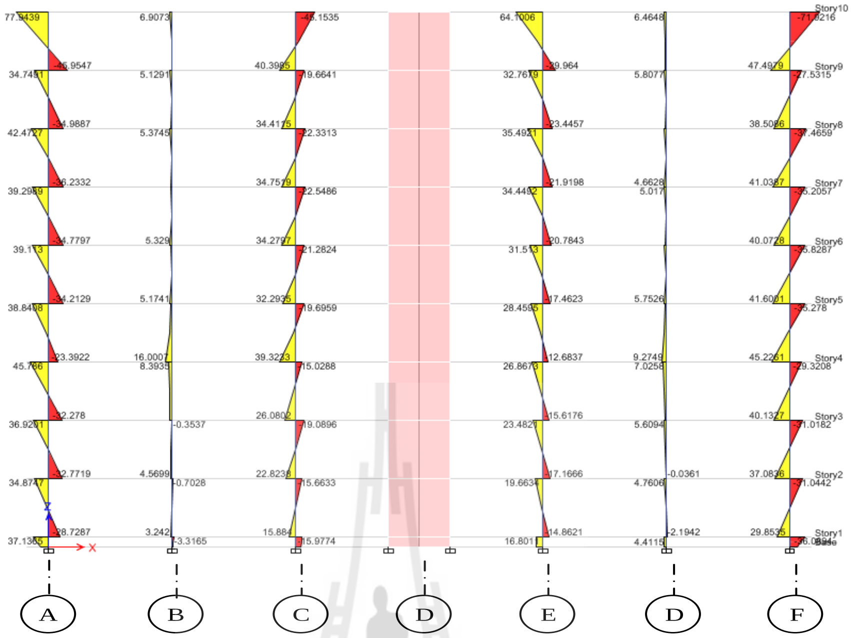
(a) Fixed base model



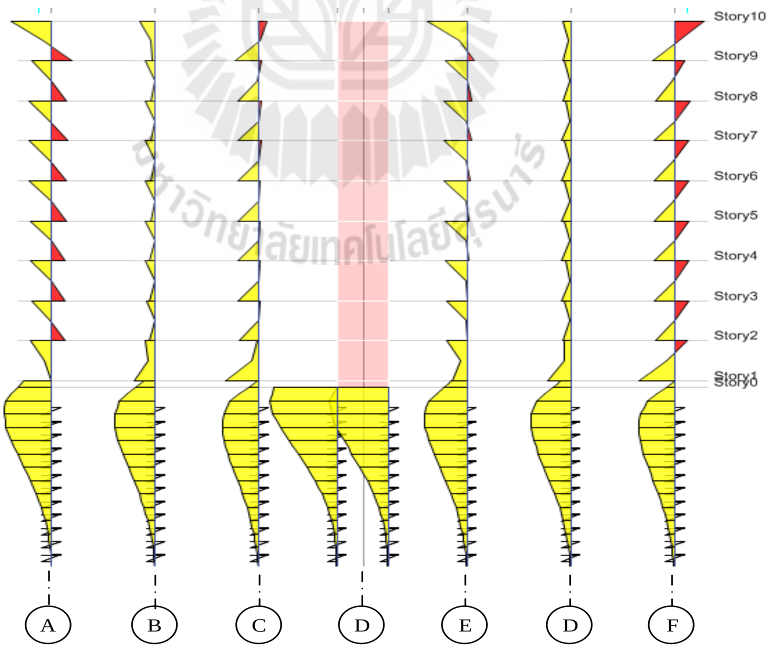
(b) SSI model

Figure 4. 23 Shear forces diagrams in Elevation 1 (a) Fixed base model; (b) SSI model

Figure 4.24 shows moment diagrams in Elevation 1.



(a) Fixed base model

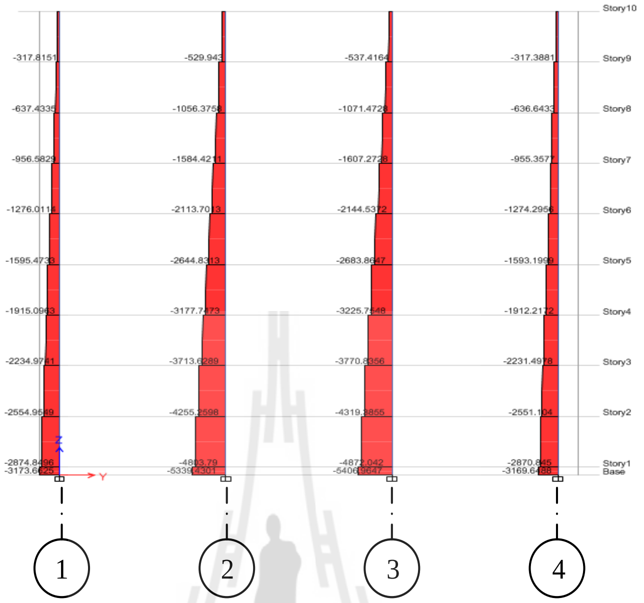


(b) SSI model

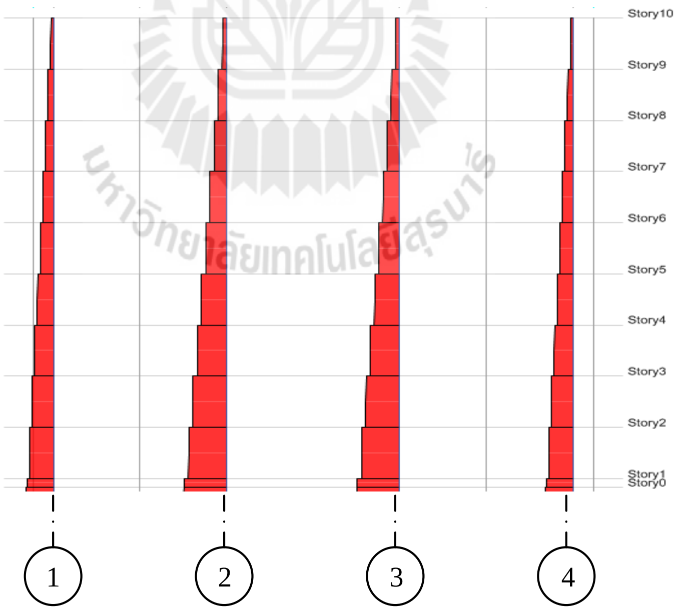
Figure 4.24 Moment diagrams in Elevation 1 (a) Fixed base model; (b) SSI model

4.8.2 Internal force diagrams of structural elements in Elevation B.

Figures 4.25 shows axial force diagrams in Elevation B.



(a) Fixed base model



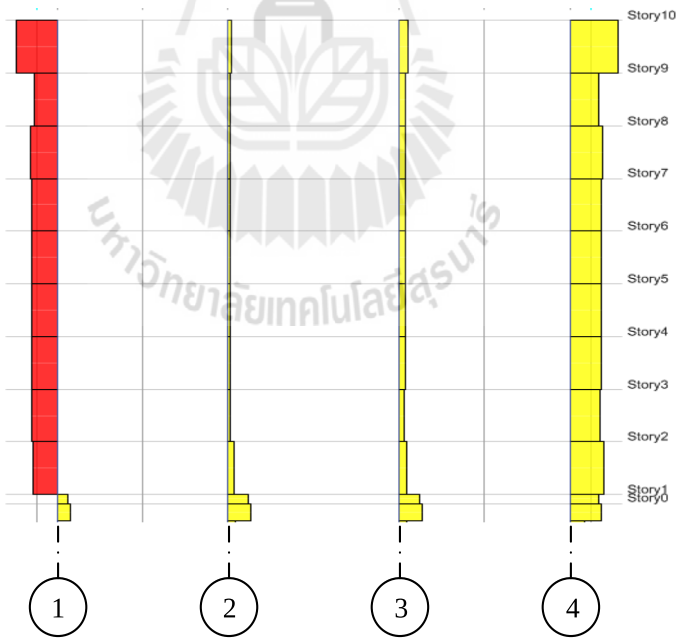
(b) SSI model

Figure 4.25 Axial force diagrams in Elevation B (a) Fixed base model

(a) SSI models

Figure 4.26 shows shear force diagrams in Elevation B.

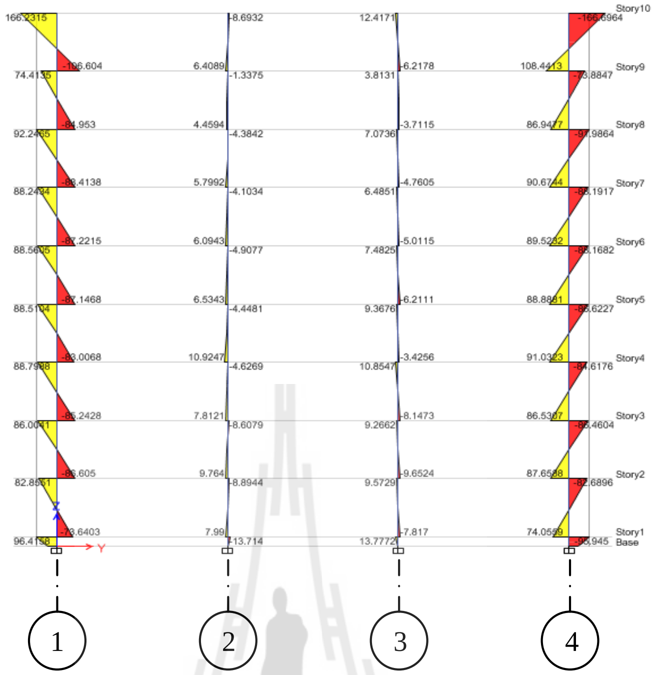
(a) Fixed base model



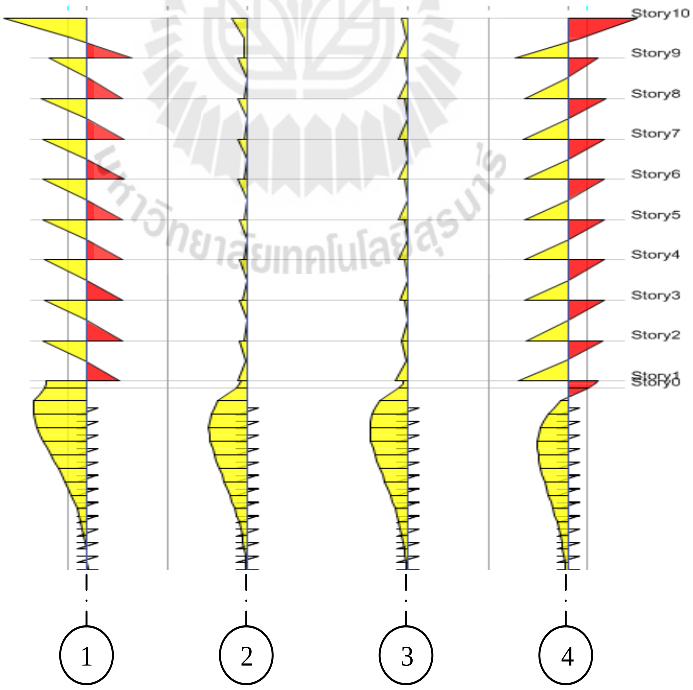
(b) SSI model

Figure 4.26 Shear force diagrams in Elevation B (a) Fixed base model; (b) SSI model

Figure 4.27 shows moment diagrams in Elevation B.



(a) Fixed base model



(b) SSI model

Figure 4.27 Moment diagrams in Elevation B (a) Fixed base model; (b) SSI model

CHAPTER V

CONCLUSIONS

In the conventional design, structures are analyzed and designed with assumption of having fixed support. However this assumption can lead to unrealistic response due to the flexibility of the soil underlying and surrounding the foundation supporting the superstructures. It is reported that for the earthquake prone areas such Bangkok (Warnitchai et al., 2000) and Mexico City (Seed, 1987) where soft soil deposits occupy for a large depth, the amplification of earthquake force leads to hazardous problems to these far-fault earthquake sites. To improve the analysis as well as the design consideration in a detail manner, the effect of soil underlying the structure should be taken into consideration and thorough study should be done in order to maintain an effective and conservative design.

The study aiming to investigate the effect of soil on the seismic response of buildings has been done. The following conclusions can be drawn from the study:

1. Based on pushover analysis of the pile, it is noticed that the empirical equations used to determine the equivalent spring stiffness of the soil are not sufficient to implement in the analysis. Pile load test data is needed to calibrate between the analyzed and test data to get a more realistic force-displacement curve.

2. The period of the structures increases when SSI is introduced into the analysis. This 2-time increase of the period lead to the increase spectral acceleration.

Hence increase in other structural response parameters. The period of the structure decreases when the stiffness of the springs increase.

3. The story displacements and story drift are significantly modified. The model of the structures with SSI shows large values of both displacement and drift in all considered directions. However the obtained results are smaller than the allowable value. The story displacement and the drift ratio also decrease when the stiffness of the spring increase.

4. Not only are the story displacement and story drift changed, the shear force and overturning moment are dramatically altered. By taking into account of the effect of soil, overall force and moment increase to higher values. Story shear and overturning moment decrease as well when the stiffness of the springs increases by changing the factor of S_u. Story shear shows a steep decrease at very small stiffness but it flattens when the stiffness get higher and higher. The value tends to converge to a constant corresponding to a very high rigidity of the support or in an ideal case of the assumption of fixed base.

Due to this study, the consideration of SSI in the design process is significant and make a considerable change for the design of structural elements. While these effects are caused by lateral loads created by earthquake ground motion, the lateral resisting elements such shear walls, core walls, bigger dimensions for corresponding structural elements, and bracings should be provided in order to absorb those lateral loads and prevent damages on the structures. However this study is a part of the work on SSI research. It is recommended to perform other case studies on different soil properties, site locations, structure configurations and other related parameters in order to get all aspects of possible solution.

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APPENDIX A

PUSHOVER ANALYSIS OF A PILE USING ETABS

Pushover analysis of the pile using ETABS

1. Create Material properties: Define → Material properties

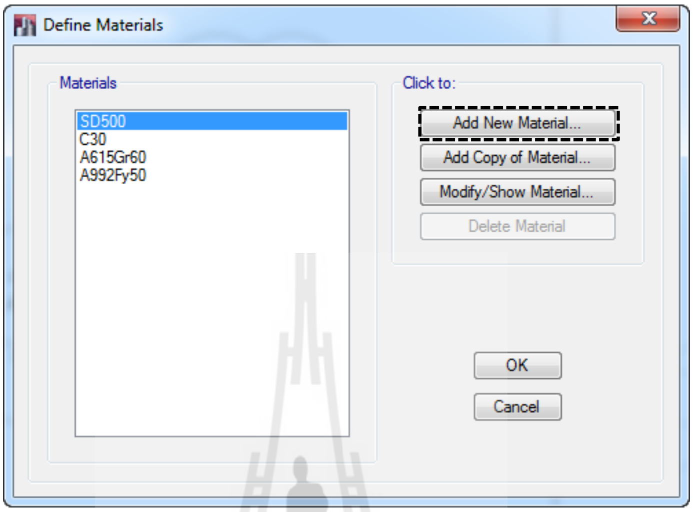

Figure A.1 Material Properties in pile analysis

- Concrete

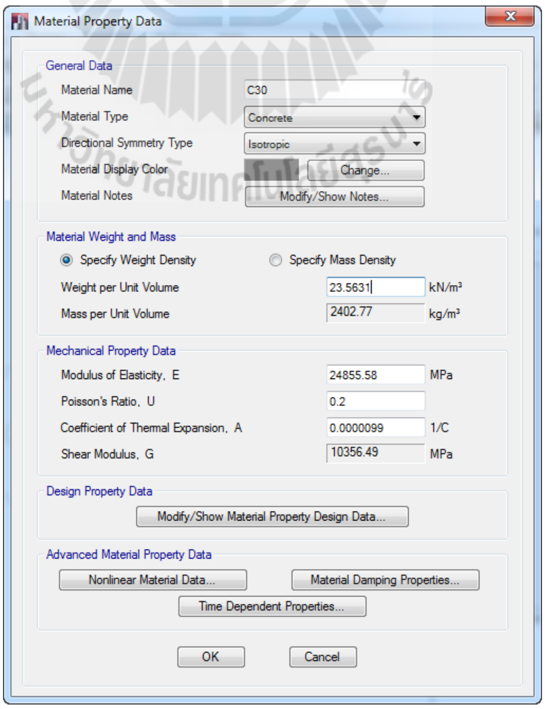

Figure A.2 Concrete Properties in pile analysis

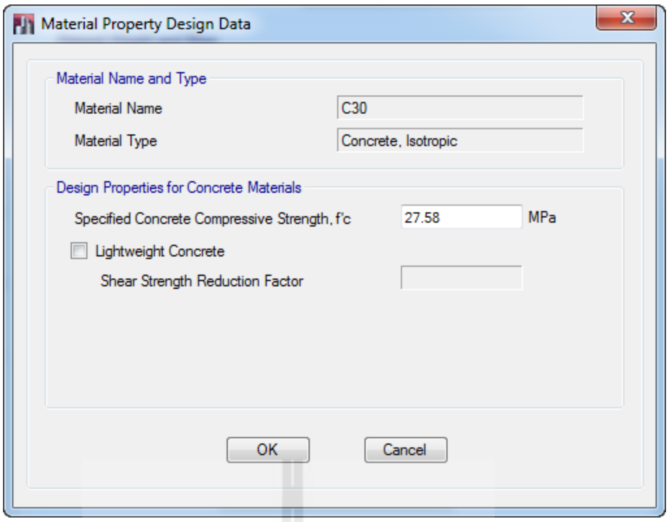

Figure A.3 Concrete Design Data in pile analysis

- Steel

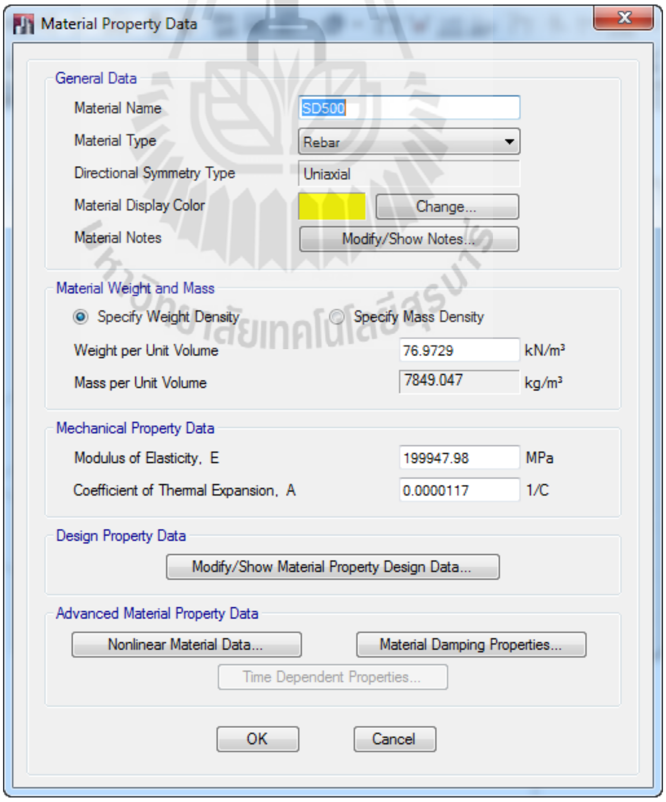

Figure A.4 Steel Properties in pile analysis

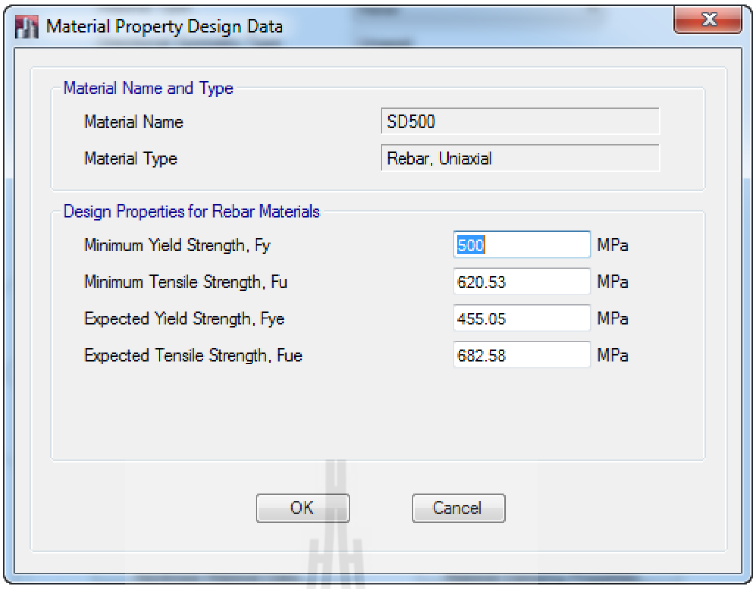

Figure A.5 Steel Design Data in pile analysis

2. Create Section properties: Define → Section properties → Frame Sections

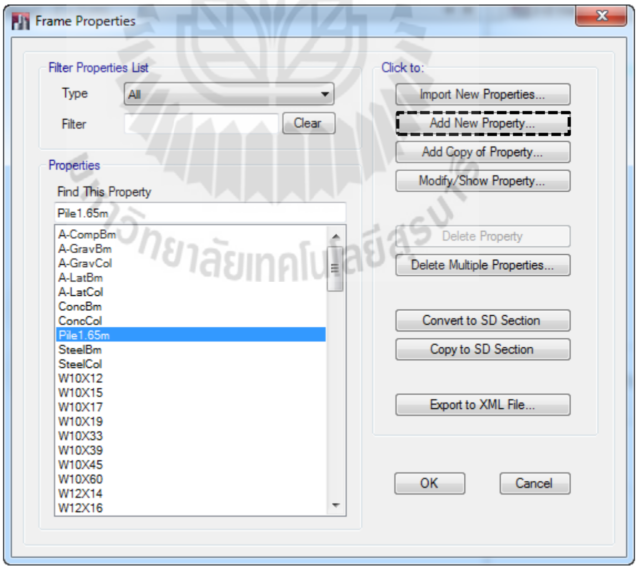

Figure A.6 Frame Properties in pile analysis

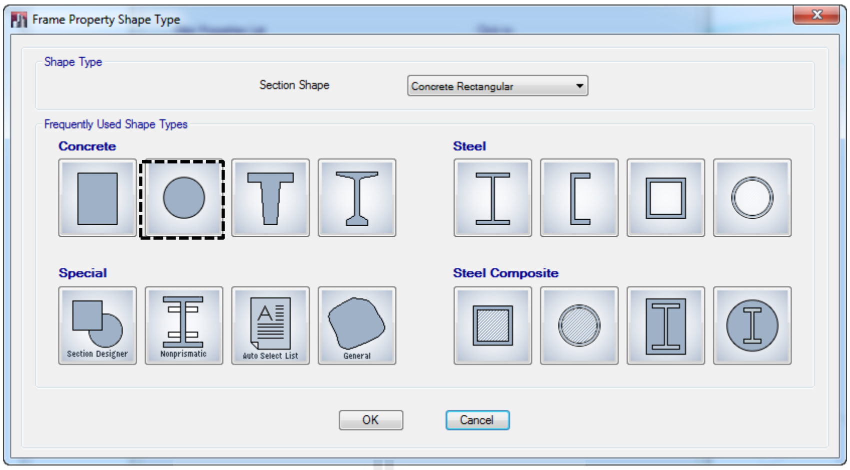

Figure A.7 Frame Property Shape Type in pile analysis

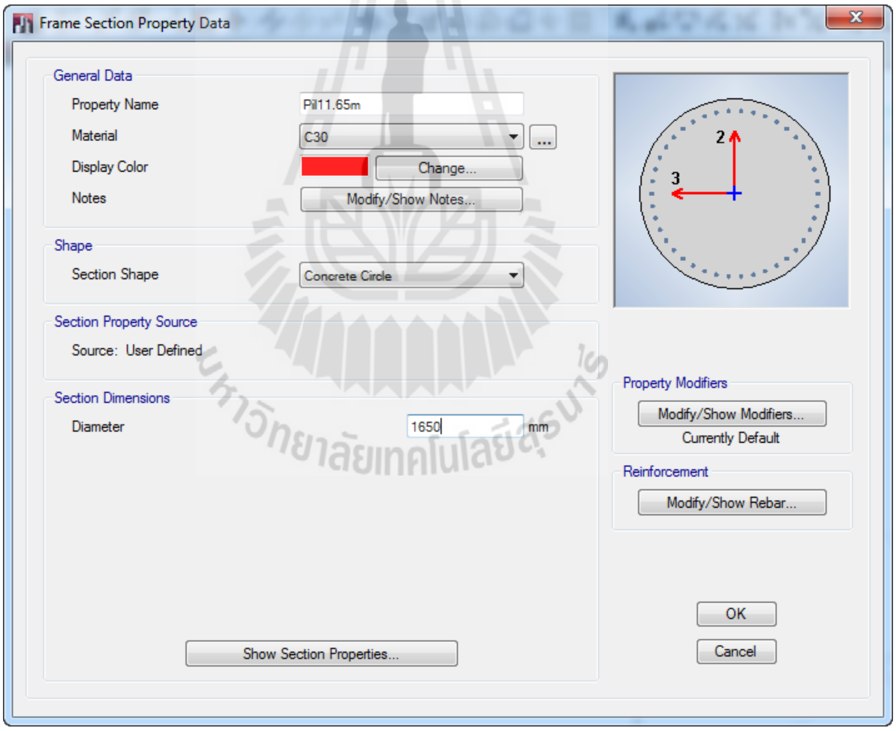

Figure A.8 Frame Section Property Data in pile analysis

3. Draw pile: Draw → Draw Beam/Column/Brace Object → Draw Beam/Column/Brace Object (Plan, Elev, 3D)

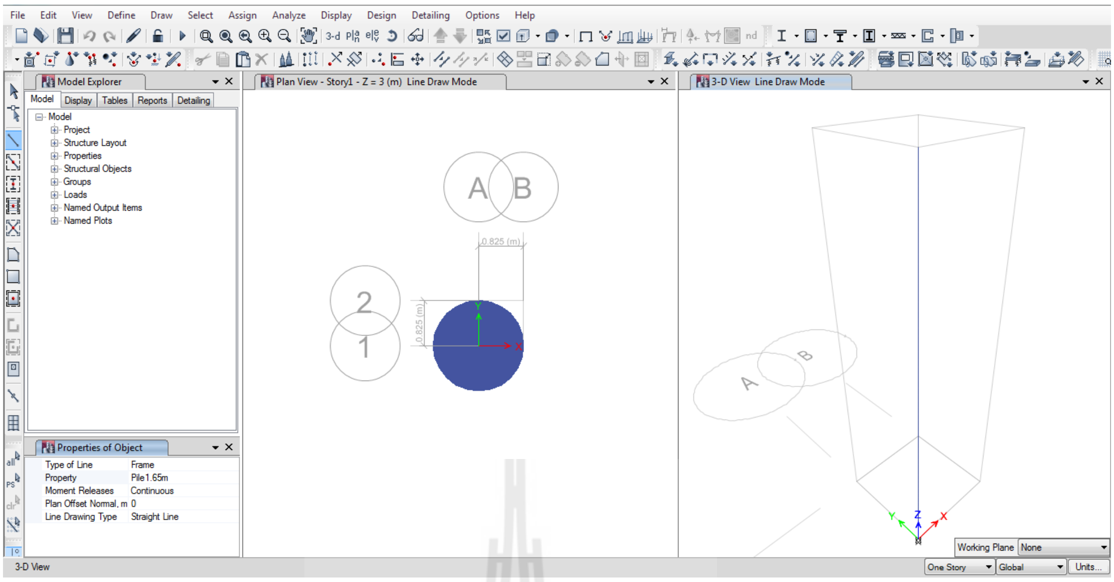

Figure A.9 Draw pile Element

4. Create Spring properties: Define → Spring Properties → Point Springs

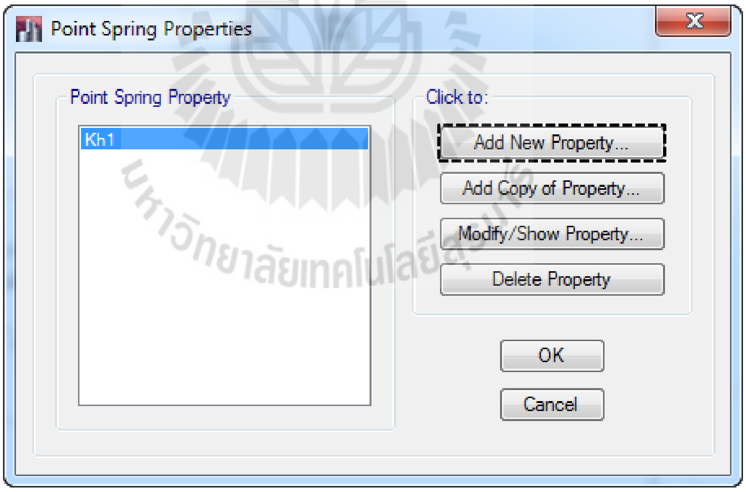

Figure A.10 Point Spring Properties in pile analysis

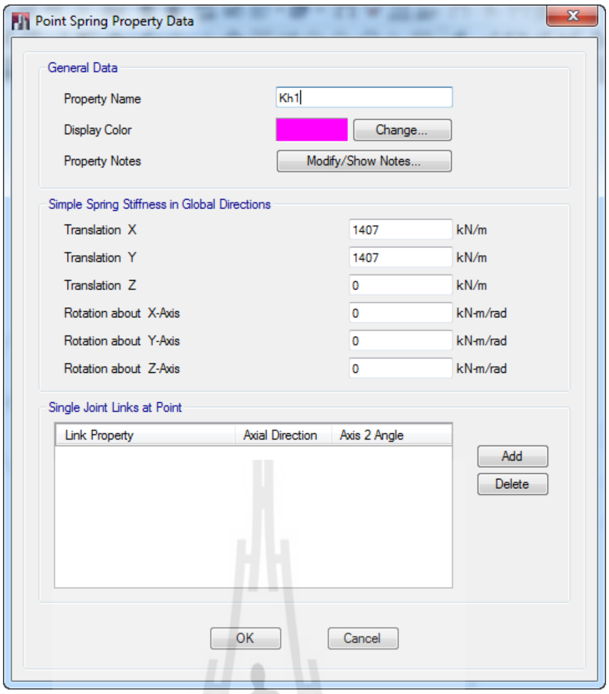

Figure A.11 Point Spring Property Data in pile analysis

5. Assign Springs: Assign → Joint → Springs

Figure A.12 Joint Assignment in pile analysis

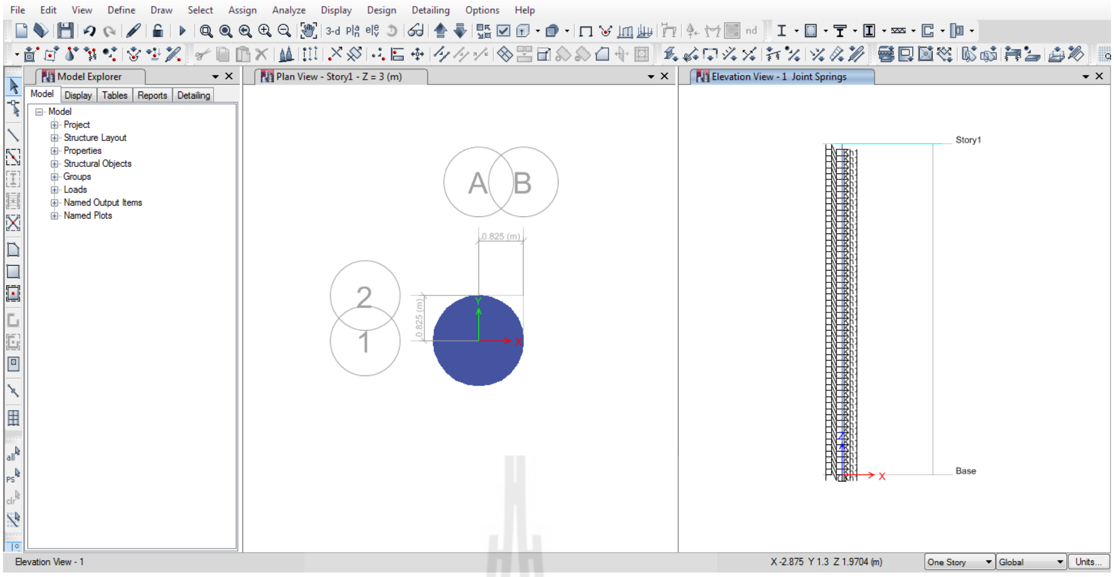

Figure A.13 Section and Lateral springs of the pile in ETABS

6. Create Load Patterns: Define → Load Patterns

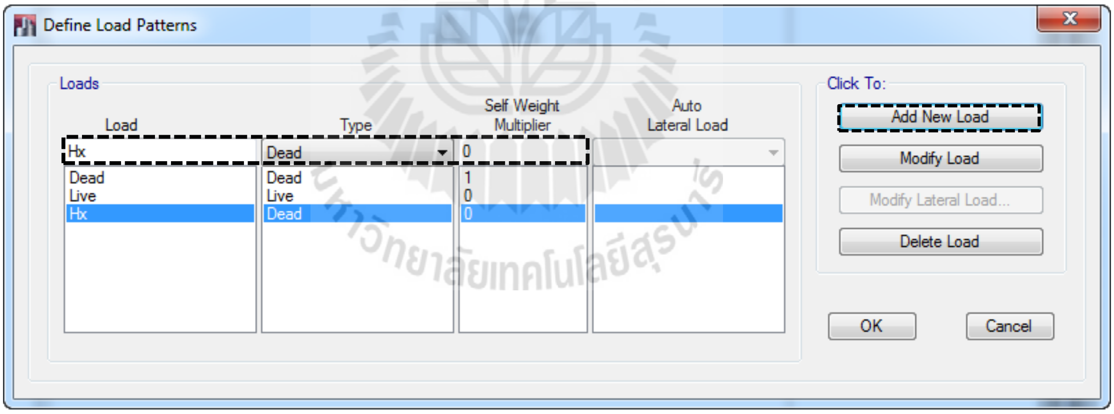

Figure A.14 Define Load Patterns in pile analysis

7. Assign Loads: Assign → Joint Loads → Forces

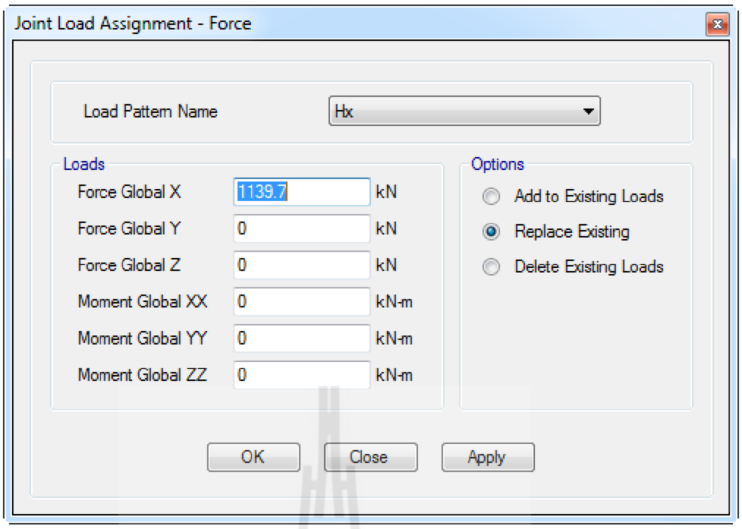

Figure A.15 Joint Load Assignment in pile analysis

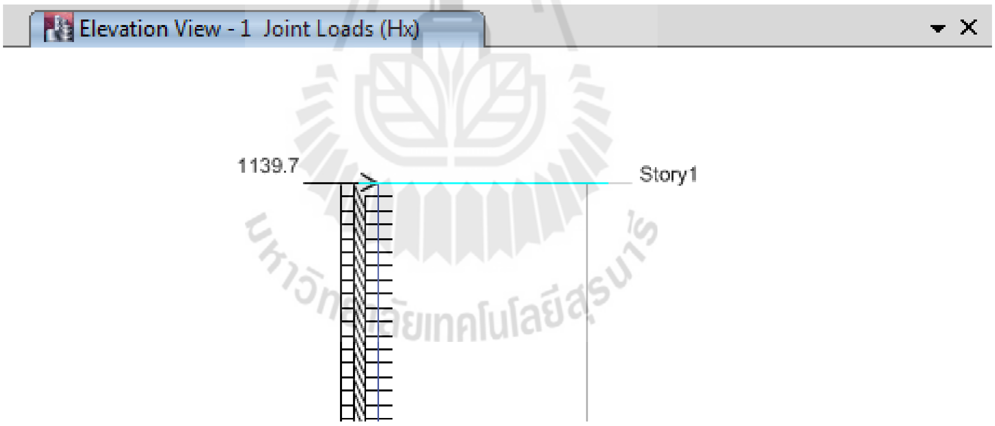

Figure A.16 Joint Load view in pile analysis

8. Run Analysis: Analyze → Run Analysis

9. Check analysis results

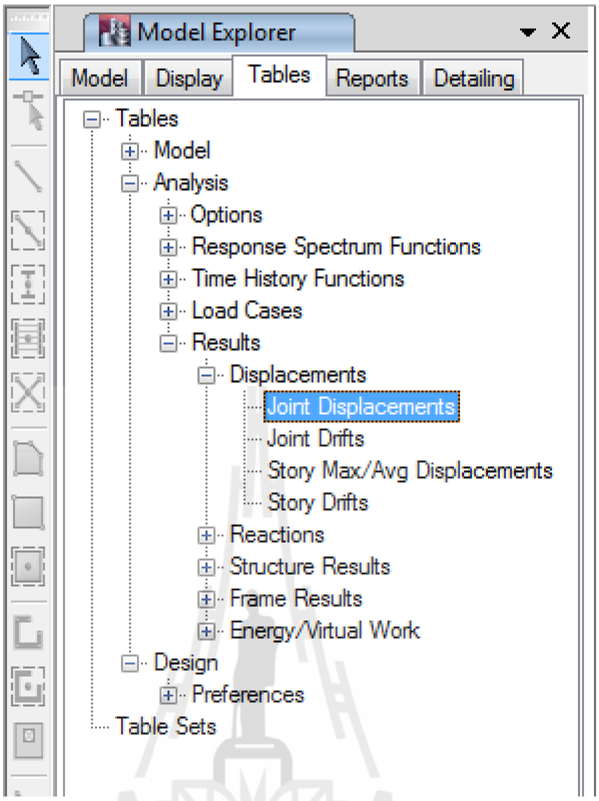

Figure A.17 Result check for pile analysis

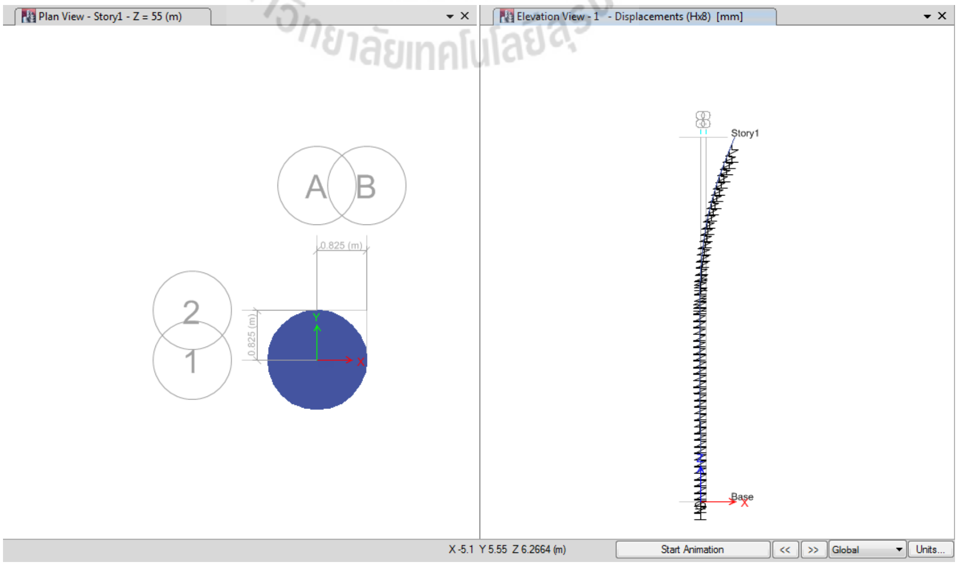

Figure A.18 Section and deflection of the pile

APPENDIX B

STRUCTURAL MODELLING USING ETABS

Structural modelling using ETABS

1. Create Material Properties: Define → Material properties

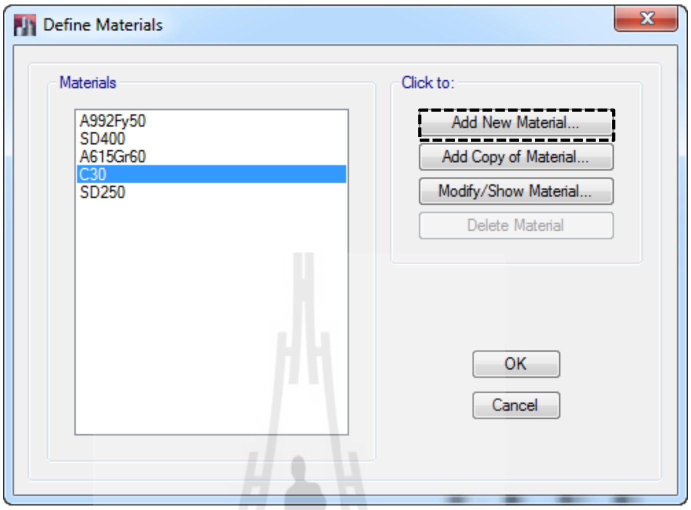

Figure B.1 Define Materials

2. Create Section Properties: Define → Section Properties

- Frame Section

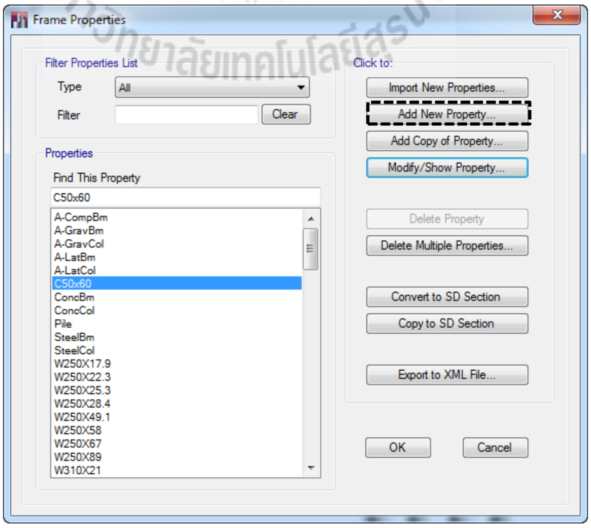

Figure B.2 Frame Properties

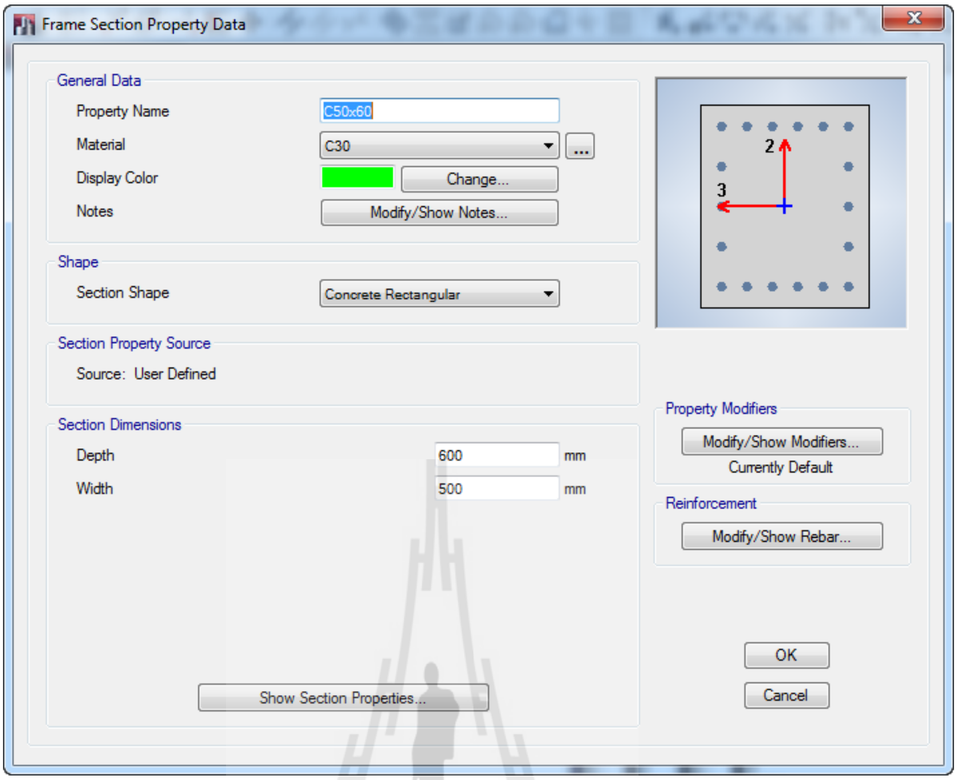

Figure B.3 Frame Section Property Data

- Slab Section

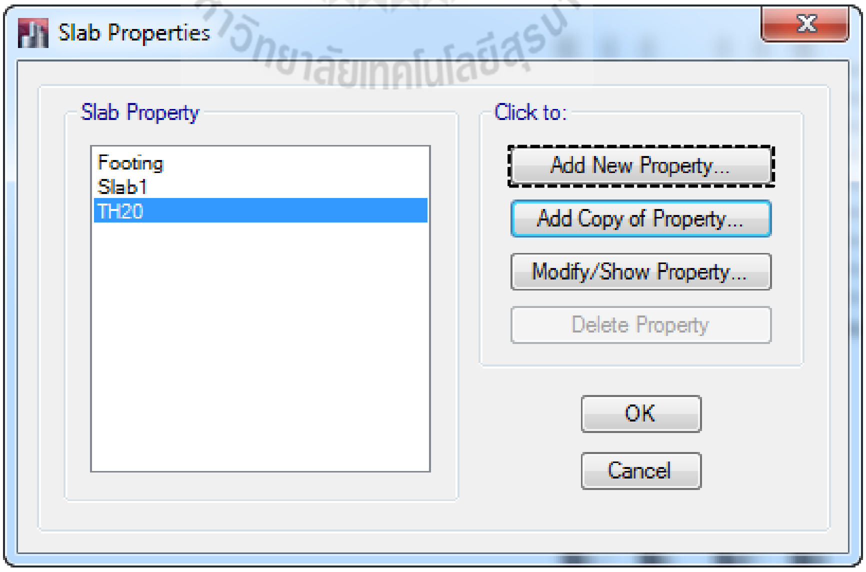

Figure B.4 Slab Properties

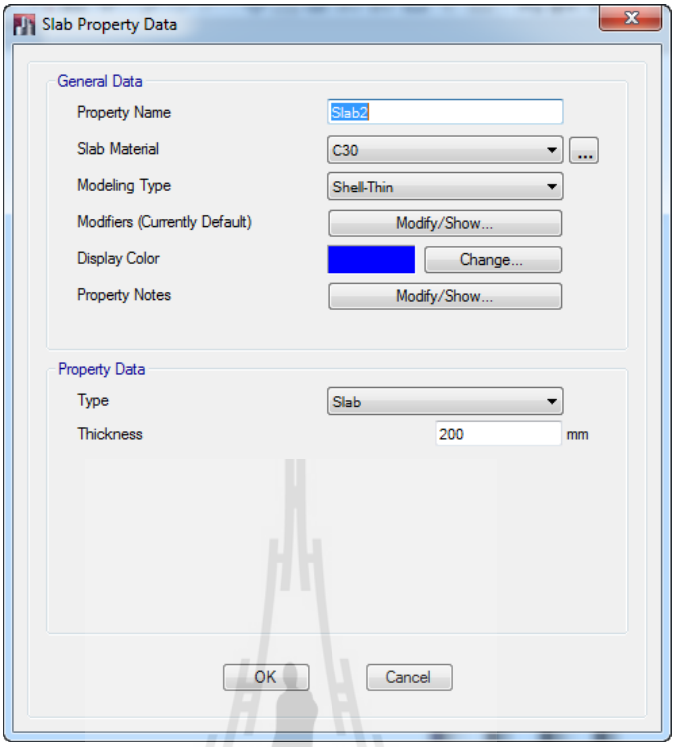

Figure B.5 Slab Property Data

- Wall Section

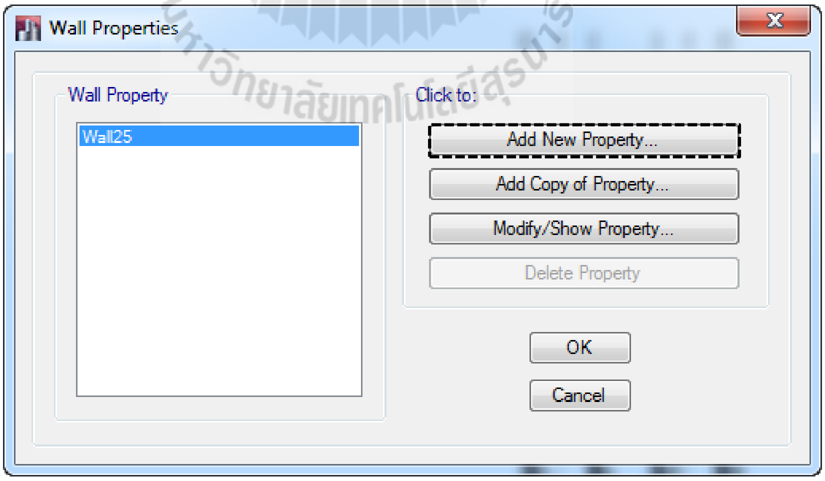

Figure B.6 Wall Properties

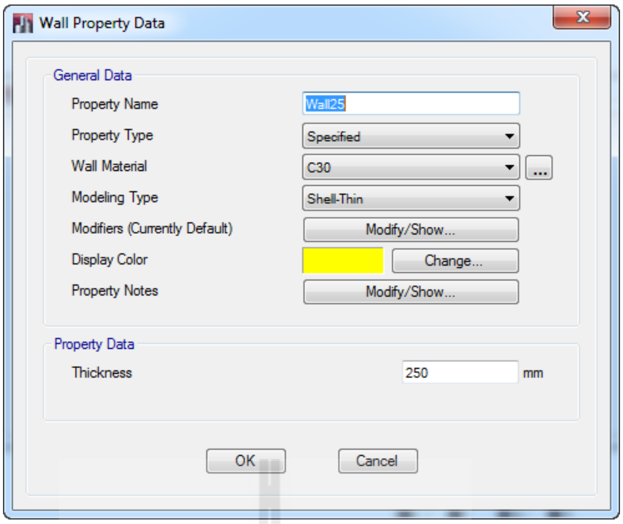

Figure B.7 Wall Property Data

3. Model the structural elements as indicated in the drawings

4. Create Spring properties: Define → Spring Properties → Point Springs

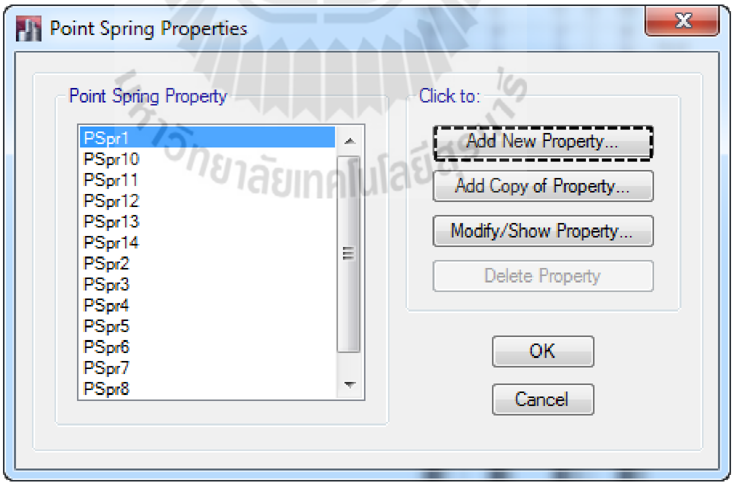

Figure B.8 Point Spring Properties

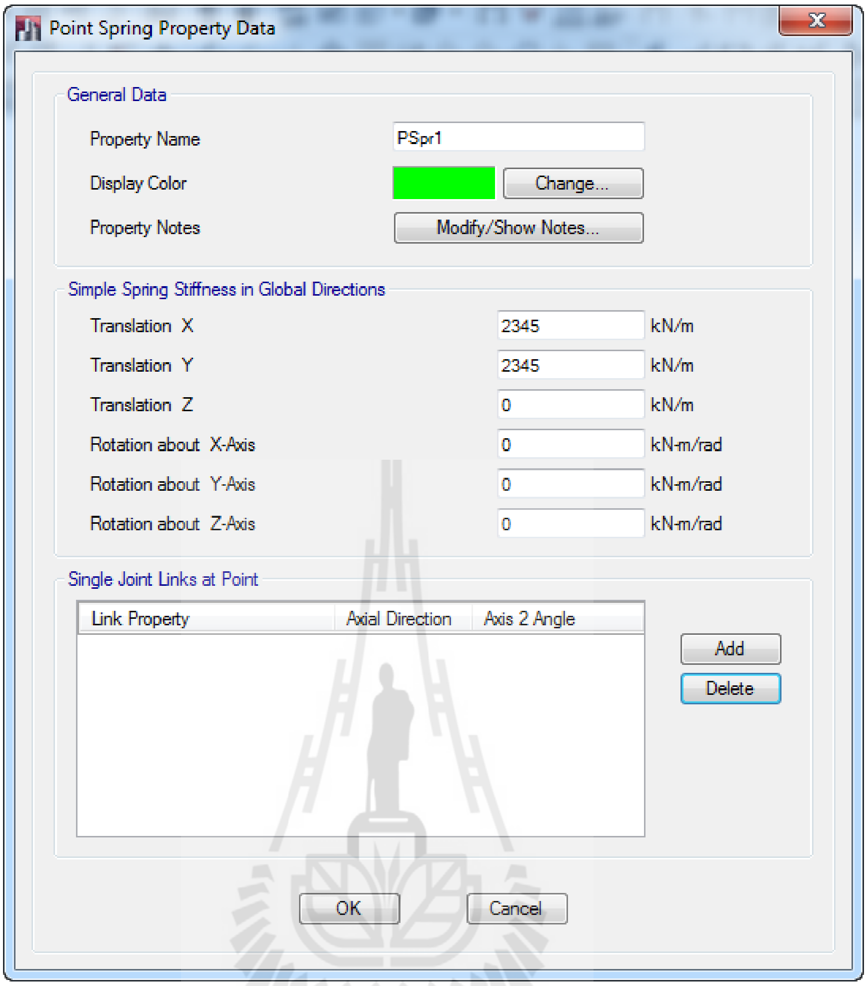

Figure B.9 Point Spring Property Data

5. Create Load Patterns: Define → Load Patterns

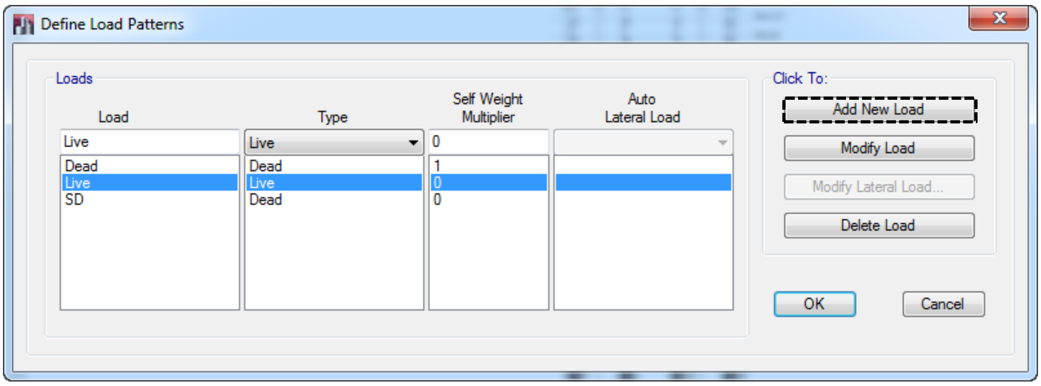

Figure B.10 Define Load Patterns

6. Create Function: Define → Function → Response Spectrum

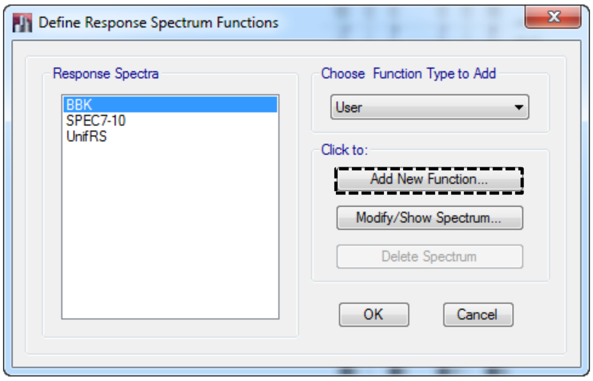

Figure B.11 Define Response Spectrum Functions

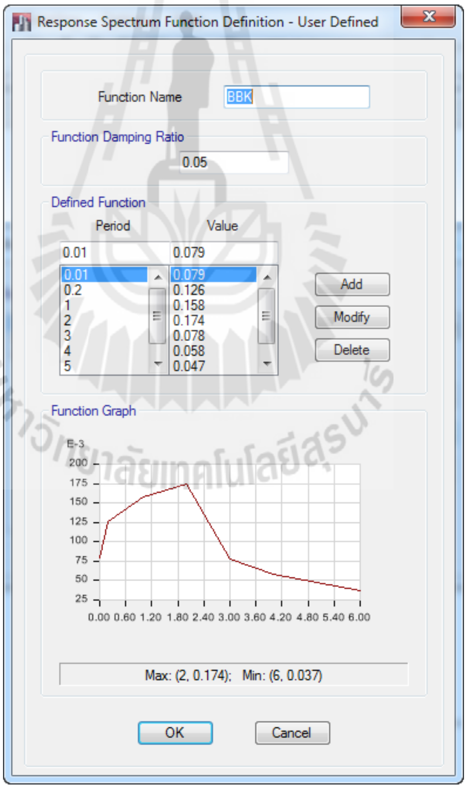

Figure B.12 Response Spectrum Function Definition

7. Create Load Cases: Define → Load Cases

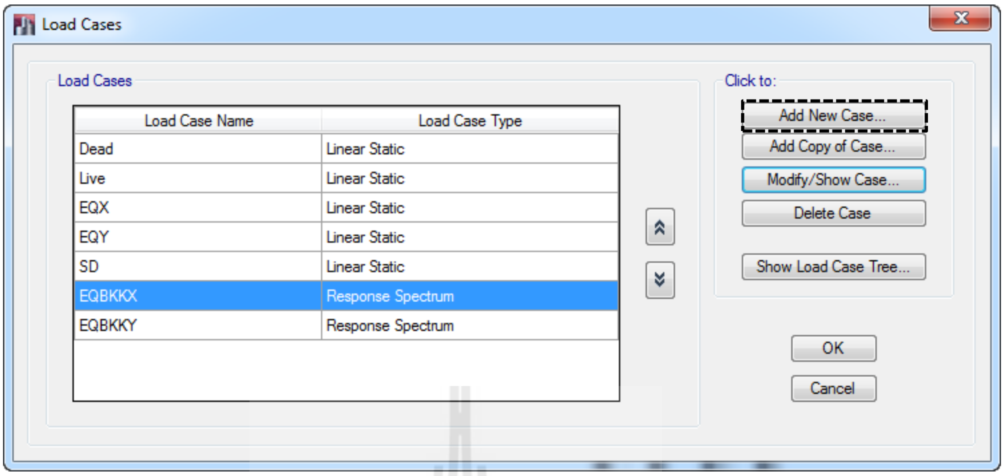

Figure B.13 Load Cases

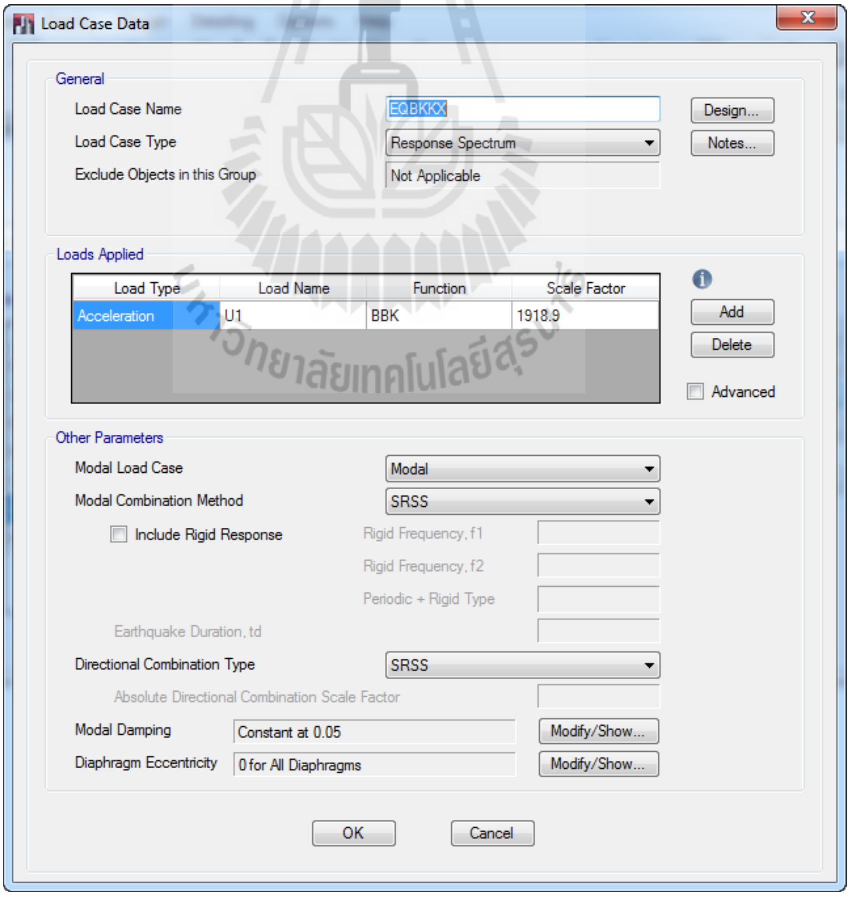

Figure B.14 Load Case Data

8. Assign Spring Properties: Select Node(s) → Assign → Joint → Springs

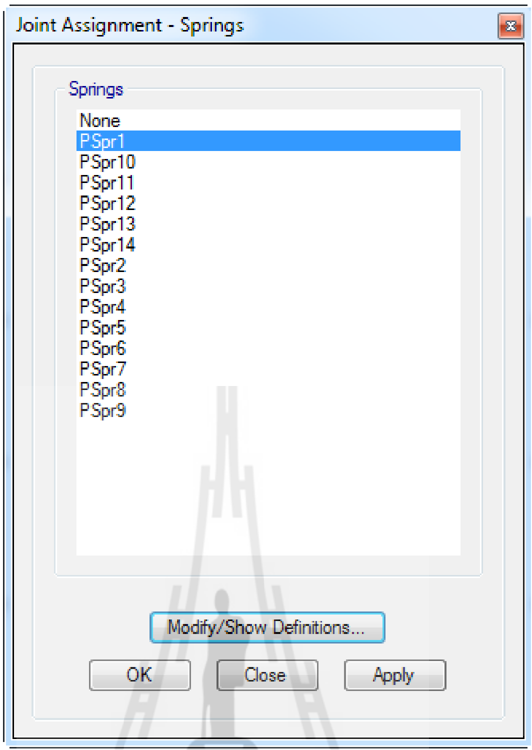

Figure B.15 Joint Assignment - Springs

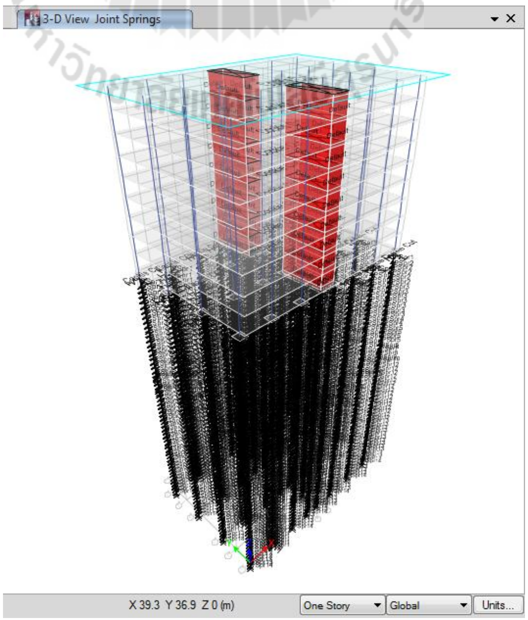

Figure B.16 3-D View Joint Springs

9. Run analysis: Analyze → Run analysis

10. Check analysis results.

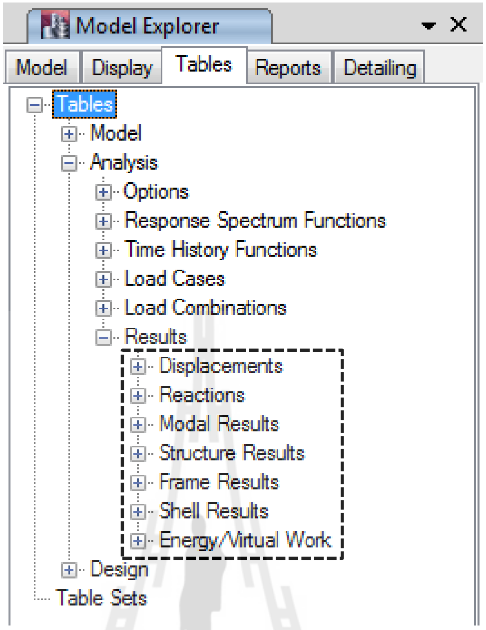

Figure B.17 Model Explorer - Analysis check

11. Display results of stories: Display → Story Response Plots

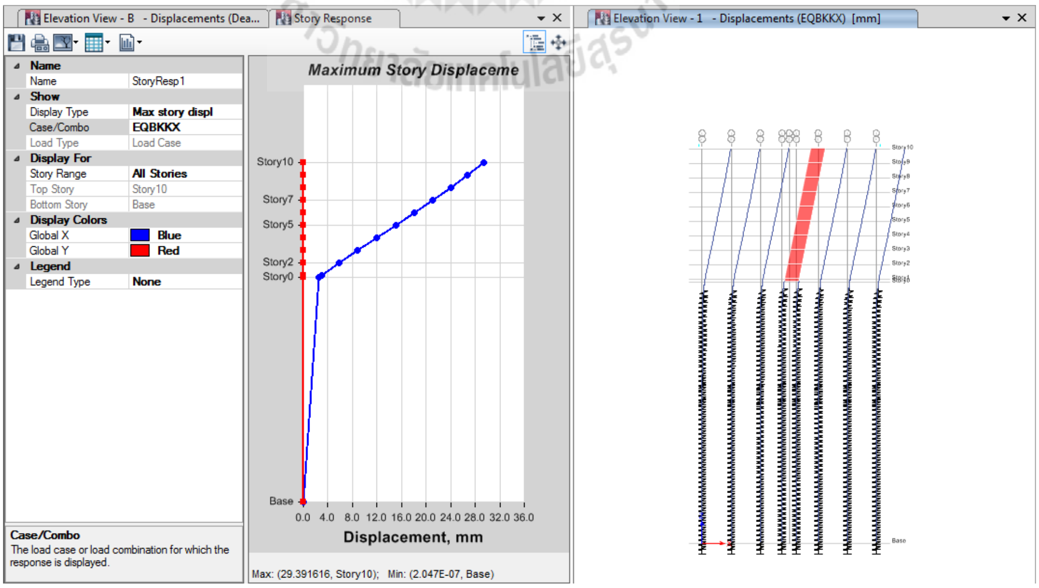

Figure B.18 Story Response Plots

APPENDIX C

PUBLICATION

Publication

Prum, S. and Jiravacharadet, M. (2014). **Effects of Soil Structure Interaction on Seismic Response of Buildings.** Paper presented in the Proceedings of the First International Conference on Advances in Civil Engineering for Sustainable Development, Nakhon Ratchasima, Thailand. August 27-29, 2014. pp. 549-554.

International Conference on Advances in Civil Engineering for Sustainable Development
Suranaree University of Technology
27-29 August 2014

Effects of Soil Structure Interaction on Seismic Response of Buildings

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ABSTRACT: In practical work, buildings are generally designed with the assumption of having fixed support. In reality, the supporting soil creates some movement of the foundation. This alters the response of the structure due to inappropriate assumption of building supports. The present study considers a reinforced concrete building resting on pile foundation. Influence of soil-structure interaction (SSI) on response of the building subjected to seismic excitation is investigated. The model of the building is created by using a conventional structural analysis and design, ETABS. Response spectrum analysis is adopted to simulate the earthquake excitation. Story displacements, story drifts, overturning moment, and story shear are observed and compared between different support conditions. Based on the study results, SSI affects the overall reponse of the structure. The results indicate that the structure exhibits larger displacements as well as story drifts caused by elastic support condition. In addition, story shears and overturning moments are modified significantly.

1 INTRODUCTION

A widely-accepted perception of soil-structure interaction in most design codes is its beneficial role in the design of structures. Design acceleration spectra resulting from actual recordings of many elastic response spectra consists normally three branches: increasing, constant and decreasing acceleration branches. Whereas the constant acceleration branch of a soft deposit soil can take up to 1 sec period (Gazetas, 2006). This long natural period may lead to smaller acceleration, bending moment, and base shear of majority of building structures and their foundation due to its position in the decreasing acceleration branch of conventional response spectra (Fardis, 2005). It is also noted similarly in ASCE7-05 (2006) that the base shear of the structure is reduced for an amount in case that soil-structure interaction is taken into account. However the beneficial role of soil in soil-structure interaction has become an unclear thing. It has been shown in many documents and case histories that over-simplification of the beneficial role of soil-structure interaction may lead to a non-conservative design of structures, hence cause destruction of structures during earthquake. The collapses of a long elevated highway section of Hanshin Expressway's Route 3 in Kobe (Mylonakis, Syngros, Gazetas, & Tazoh, 2006) and buildings in the recent Adana-Ceyhan earthquake (Celebi, 1998) were caused by detrimental role of soil.

Therefore this paper aims to study the detrimental role of soil participating in seismic response of structures. A case study of a building with its corresponding soil profile is used to observe its elastic response while it is subjected to earthquake excitation. This study is useful for understanding the performance of a structure with consideration of its underlying soil properties.

2 STRUCTURAL MODEL

2.1 *Superstructure model*

The case-study building is a concrete core-wall building with the height of 27.5 m and 36 m x 24 m floor plan (Figure 1). The column cross section is 60 cm by 50 cm and the wall thickness is 25 cm. The lateral load resisting elements of the building are reinforced concrete core wall and columns. The compressive strength of concrete is 30 MPa and yielding strength of rebar is 400 MPa. Young modulus of concrete and steel are 25,743 MPa and 200,000 MPa, respectively. The gravity-load carrying system used to support a super-imposed dead load of 1 kN/m^2 and a live load of 3 kN/m^2 is a 20 cm thick post-tension concrete flat slab resting on reinforced concrete columns and shear walls.

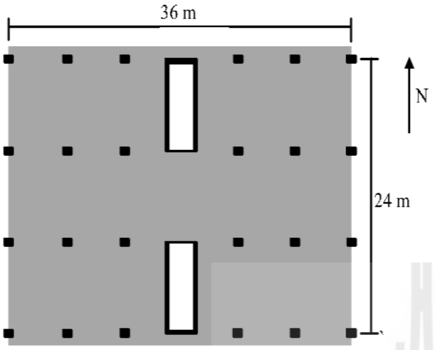

Figure 1. Floor plan of the case-study building.

The flat slab and wall are modeled as shell element, while columns are modeled as beam-column frame element. Finite element program, ETABS (2005) is used to model the considered building. 3D model of the studied fixed-base structure is shown in Figure 2.

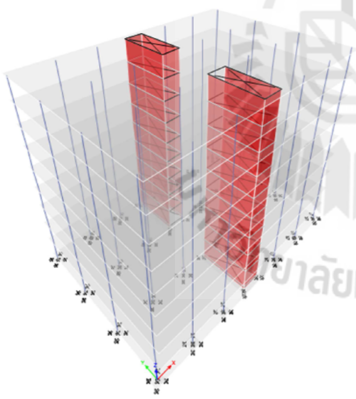

Figure 2. 3-D model in ETABS.

2.2 *Substructure model*

Footings and piles are also included the model. 1m-thickness footing with 1.5 m width by 1.5 m length is modeled under each column, while footing with 4.2 m width by 9.2 m length is modeled under each core wall. Property of the footings is defined as shell-thick element in the model. Under each column, a circular-column element with diameter of 1m and length of 39 m is modeled to represent a pile supporting the superstructure. There are totally 6 piles used to support a core wall. A 3D model of footings with piles is shown in Figure 3.

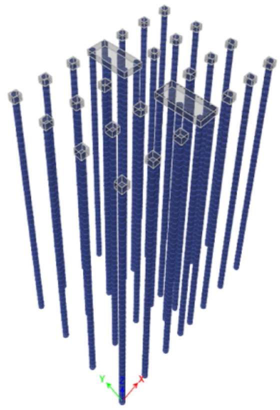

Figure 3. 3D model of substructure elements.

3 REPONSE SPECTRUM

In this study, the design response spectrum from Thai seismic design code is used. Figure 4 below shows the plot of pseudo acceleration with natural period of the structure.

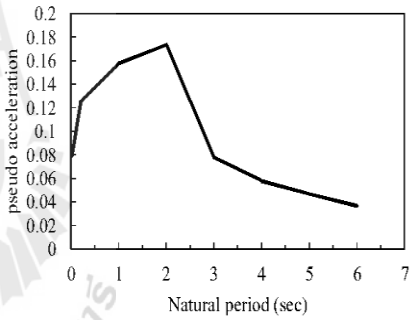

Figure 4. Design response spectrum.

4 EQUIVALENT SOIL SPRING AND VERTICAL STIFFNESS FOR PILE FOUNDATION

The analysis of the laterally loaded piles has been done by many researchers. The widely used and simple method of analysis is by using the subgrade reaction model proposed by Winkler 1867. In that method, the surrounding soil is modeled as a series of unconnected linearly-elastic springs. The spring stiffness is defined in term of modulus of horizontal subgrade reaction (k_h : kN/m^3) as shown in Equation 1.

$$K_h = k_h \times B \times \Delta L \qquad (1)$$

where k_s = modulus of horizontal subgrade reaction; B = width or diameter of the pile; and ΔL = spring spacing.

In Winkler model, the pressure p and the deflection y at a point are assumed to be related through the modulus of subgrade reaction, which for horizontal loading, is denoted as k_h and shown in Equation 2. The physical meaning of k_h is illustrated in Figure 5.

$$k_h = \frac{p}{y} \tag{2}$$

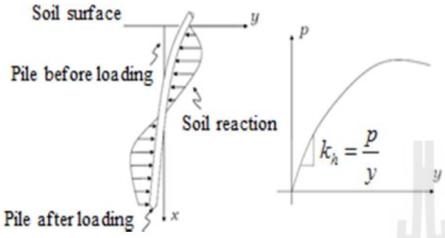

Figure 5. Horizontal subgrade reaction of soil.

4.1 *Modulus of subgrade reaction for sand*

Terzaghi (1955) presented the evaluation of modulus of subgrade reaction for sand as shown in Equation 3.

$$k_h = \frac{n_h z}{B} \tag{3}$$

where z = depth below ground surface; B = width or diameter of the pile; and n_h = constant of horizontal subgrade reaction.
Recommended values for constant of subgrade reaction are shown in Tables 1 and 2.

Table 1. Recommended values for constant of subgrade reaction (Davisson, 1970).

Soil type	n_h kN/m^3
Granular	2840-28380
Silt	110-850
Peat	60

Table 2. Recommended values for constant of subgrade reaction (Reese, Cox, & Koop, 1974; Terzaghi, 1955).

Soil property	Loose	Moderate	Hard
Author	kN/m^3	kN/m^3	kN/m^3
Terzaghi (1955)	740-2180	2180-7380	7380-14470
Reese et al. (1974)	5680	17030	35470

Tomlinson and Woodward (2008) investigated the the relationship of SPT-N value with relative density of soil (Tab. 3) by using the graphic given by Terzaghi and Reese as shown in Figure 6.

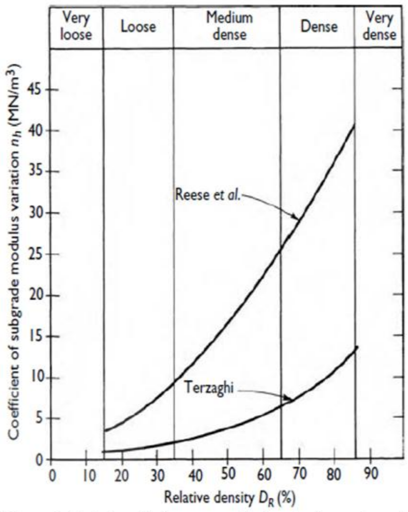

Figure 6. Relationship between relative density and coefficient of subgrade reaction.

4.2 *Modulus of subgrade reaction for clay*

Modulus of subgrade reaction for clay was introduced by Davisson (1970) as shown in Equation 4.

$$k_h = 67\frac{S_u}{B} \tag{4}$$

where S_u = undrained shear strength and B = width or diameter of the pile.

4.3 *Soil properties*

In this study, Bangkok subsoil is used. Soil profile and its correstponding properties are shown in Table 3 (Chandrasakha, 2013). As seen in Table 3, the underlying soil in Bangkok area consists of a very deep soft deposit which may be vulnerable to earthquake excitation tending to amplify the response of the structure.

Table 3. Soil layers and their properties.

Depth	Soil type	SPT-N	Su (kN/m2)	ϕ
2-15	Very soft ot soft clay	2-12	21	-
15-18	Medium clay	6-18	57	-
18-25	Stiff to very stiff clay	15-35	120	-
25-40	Dense sand	18-65	-	35
40-45	Hard clay	27-62	262	-
45-60	Very dense sand	25-80	-	36

4.4 *Calculation of spring stiffness*

Based on the calculation procedure and soil properties given above, stiffness of each spring can then be evaluated. The constant of subgrade reaction for sand is chosed from the recommended value given by Terzaghi (1955). The following table is the summary of the spring stiffness to be used in the analysis.

Table 4. Summary of spring stiffness.

Depth m	Type of soil	SPT-N	Su kN/m^2	k kN/m^3	K kN/m
1-15	Very soft to soft clay	2-12	21	1407	1407
16-18	Medium clay	6-18	57	3819	3819
19-25	Stiff to very stiff clay	15-35	120	8040	8040
26	Dense sand	18-65	-	7380	191880
27			-		199260
28			-		206640
29			-		214020
30			-		221400
31			-		228780
32			-		236160
33			-		243540
34			-		250920
35			-		258300
36			-		265680
37			-		273060
38			-		280440
39			-		287820

4.5 *Stiffness of vertical spring*

The stiffness of the vertical spring applied at the tip of the pile (K_v = 518159.58 kN/m) is used in the analysis. Equation 5 shows its relation with section modulus and length of the pile.

$$K_v = \frac{EA}{L} \tag{5}$$

where E = Young's modulus of pile; A = area of cross section of the pile; and L = length of the pile.

5 RESULTS AND DISCUSSION

Analyzed model is shown in Figure 7. According to the study results, it is found that the periods of the model with flexible base support are higher than that of fixed base support for all considered modes. The period of the first mode of the model with SSI is about 2 times of the period of the fixed base model. This is caused by the flexibility of the soil that tends to lengthen the structural period. Table 5 summarizes the modal periods and frequencies of each studied model. The notation "SSI" refers to the model of the structure with springs supporting the piles and "Fixed" refers to the model of the structure with fixed support.

Table 5. Modal periods and frequencies

Mode	Period (sec)		Frequency (rad/sec)	
	Fixed	SSI	Fixed	SSI
1	0.865	1.823	7.2644	3.4461
2	0.758	1.675	8.2853	3.752
3	0.491	1.508	12.7847	4.1667

The detrimental effect of soil structure interaction is clearly seen in Figures 8 and 9. In Figure 7, the displacement at story 1 refers to the displacement at ground level of the structure. It is noticeable at that level that the displacement of the structure with pile foundation is not zero, while that of fixed base structure is approximately zero (Figs 8 and 9). Due to the flexibility of the soil represented by a series of springs, the pile heads can move for some distance. The displacement then continues increasing from the base of the structure to its maximum value which is about 5 times compared to the displacement of fixed base structure (Figure 8). The displacements in E-W direction are larger than that in N-S direction since in N-S direction the two core wall provide large stiffness to resist the lateral load applied.

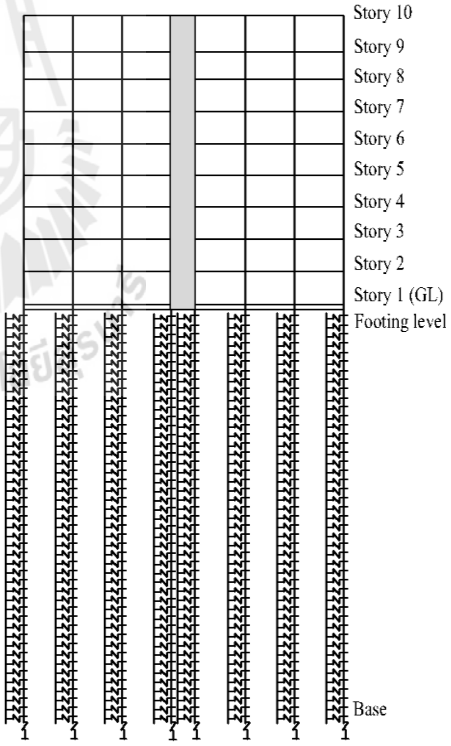

Figure 7. An elevation of the studied model.

It is similarly observed for the structure with SSI that story drifts (Figures 10 and 11) are magnified significantly.

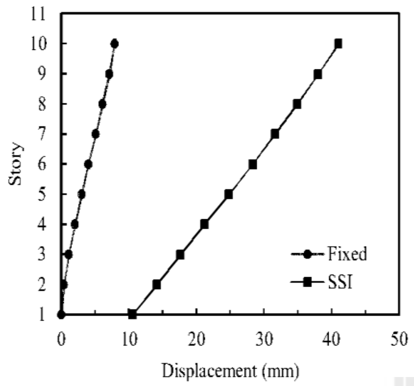

Figure 8. Story displacements of fixed and flexible base structures - E-W direction.

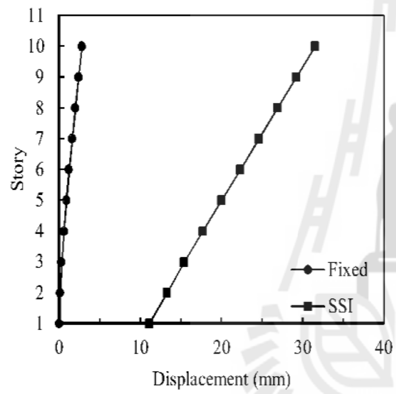

Figure 9. Story displacements of fixed and flexible base structures - N-S direction.

The maximum drifts in both directions are at the first story of the structure with flexible base, while the maximum drifts for fixed base structure are at its 7^{th} story for drift in E-W direction and at its 9^{th} story for drift in N-S direction. The difference between the two maximum drifts in E-W direction is about 3.5 times.

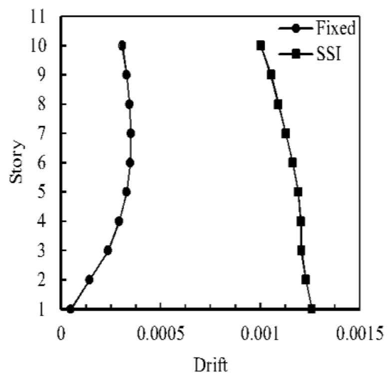

Figure 10. Story drifts of fixed and flexible base structures E-W direction.

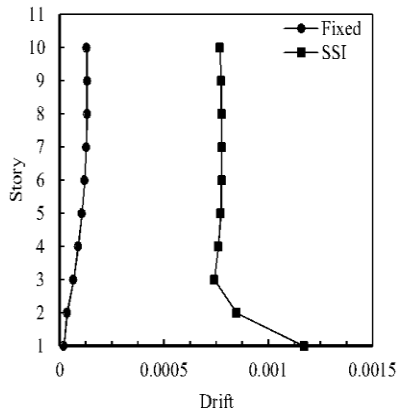

Figure 11. Story drifts of fixed and flexible base structures N-S direction.

The maximum drift among all cases is about 0.0013 which is relatively small compared to allowable drift of 0.02 mentioned in ASCE7-05 (2006). Even though the drifts do not exceed the limit, higher drifts may cause higher change of internal forces of structural elements in each story.

Refered to Figures 12 and 13, the story shears of both fixed and flexible base are compared. The story shear indicates the value of force exerting on each floor of the structure. As seen in Figures 12 and 13, shear forces at each story of flexible base structure are higher than that of fixed base structure regarless of directions. The value is more important at the base of the structrue where the shear force of the structure with SSI is about 1.5 times of that of fixed base structure in E-W direction and 1.8 times in N-S direction. According to this result, the value of base shear is almost double when SSI is taken into account. This high value of shear force should be paid attention to during analysis as well as design of resisting elements.

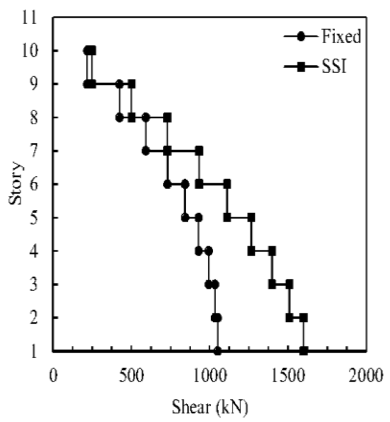

Figure 12. Story shears of fixed and flexible base structures E-W direction.

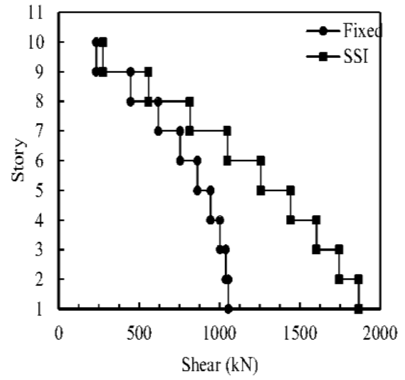

Figure 13. Story shears of fixed and flexible base structures N-S direction.

Similar to story shears, the overturning moments of each floor also increase to a significant value (Figs 14 and 15). The moments at the top of both structures are equally zero. The values then tend to separate from each other to higher values until they reach their maximums which are the base of the structures. Both values are about 1.5 times difference in both direction and the structure with SSI shows a higher value.

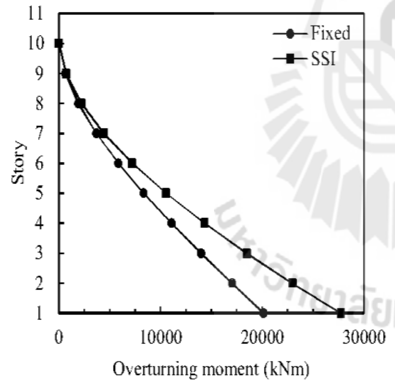

Figure 14. Overturning moments of fixed and flexible base structures E-W direction.

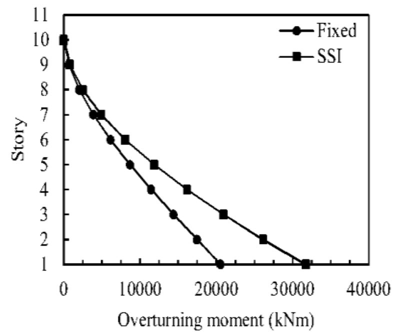

Figure 15. Overturning moments of fixed and flexible base structures N-S direction.

6 CONCLUSION

In this study, a reinforced concrete core-wall buiding is analyzed with the consideration of soil-structure interaction. The structure is modeled in conventional structural analysis and design program, ETABS. Response spectrum analysis is used in the analysis to simulate earthquake excitation. The study shows that SSI affects the overall response of the structure. The period of the structure with SSI increases to about 2 times compared to that of fixed base structure. The results also reveal that the structure exhibits larger displacements as well as story drifts while SSI interaction is taken into account. Moreover, the forces in the structure such story shears and overturning moments are magnified significantly. However, it is recommended to do further research on different building configurations and height as well as different soil profiles to understand clearly on the seismic response of buildings.

ACKNOWLEDGEMENT

The first author acknowledges a financial support from ASEA-UNINET program for his master study.

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BIOGRAPHY

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