ADDIS ABABA UNIVERSITY

SCHOOL OF GRADUATE STUDIES

A STUDY ON
THE EFFECT OF SOIL-STRUCTURE INTERACTION ON SEISMIC RESPONSE OF REINFORCED CONCRETE BUILDINGS SUPPORTED ON PILE FOUNDATIONS

By

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A STUDY ON THE EFFECT OF SOIL-STRUCTURE INTERACTION ON SEISMIC RESPONSE OF REINFORCED CONCRETE BUILDINGS SUPPORTED ON PILE FOUNDATIONS

A thesis submitted to the School of Graduate Studies of Addis Ababa University in partial fulfillment of the Requirements for the Degree of Masters in Civil Engineering.

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ABSTRACT

In this study, the effects of soil-foundation-structure interaction effects are investigated for five reinforced concrete building structural models supported on pile foundations. The building models are assumed to be regular both in geometry and stiffness to avoid unnecessary complications due to torsion. The study is conducted using the sub structure method of soil-foundation-structure interaction analysis which idealizes the superstructure to be supported by springs and dashpots. The spring and dashpot coefficients, which respectively represent the stiffness and damping of the foundation system, are obtained from the dynamic impedance functions.

Using SAP structural analysis software the flexible-base and fixed-base building models are analyzed and differences in their seismic responses systematically compared. Base shears, overturning moments, bending moments and shear forces along selected beams, axial forces in selected columns, bending moments in shear walls are the main parameters used to compare the analyses results of the flexible-base and fixed-base models to assess the influence of soil flexibility.

In this study it is observed that the seismic responses of buildings supported on pile foundations are influenced when the effects of soil-structure interaction are considered. In most of the cases considering soil-structure interaction results in lower seismic responses which is an economic advantage. However, it should be stated that in some cases higher seismic responses of buildings are observed. The seismic responses of buildings supported on pile foundations are observed to depend mainly on soil type, bearing mechanism of piles, and height of buildings when the effects of soil-structure interaction are considered.

Key words: Seismic Soil-Pile-Structure Interaction (SSPSI), dynamic impedance function, flexible-base model, fixed-base model.

1. INTRODUCTION

1.1. Background

When analyzing the seismic response of a structure founded on solid rock, it is natural to assume the base of the structure to be fixed and hence the input motion from a potential earthquake at the foundation level of the structure is merely that of the free-field. This gross assumption is realistic only when the relative stiffness of the foundation soil compared to the structure is high. However, if the same structure is supported on soft compliant soil, the earthquake motion at the base of the building i.e. the Foundation Input Motion (FIM), is not likely to be identical with the Free-Field Motion (FFM), which is motion of the ground with the absence of any structure or deep excavation (Wolf 1985). Besides, the inertia of the structure during its vibration in response to the seismic excitation will induce additional soil deformation which in turn modifies the seismic response of the structure.

In the literature, the former interaction effect occurring between the supporting soil and the foundation is called kinematic interaction and modifies the FFM. Whereas, the later interaction effect between the foundation and the supported superstructure is known as inertial interaction, and develops base shears and overturning moments at the foundation level making the foundation to translate and rotate thereby inducing additional soil deformation and energy dissipation. The whole interdependent phenomenon of interaction between the soil-foundation-superstructure system attributed to the compliance of the soil during earthquake shaking is called Seismic-Soil–Foundation-Superstructure–Interaction (SSFSI) or interchangeably termed Soil-Structure Interaction (SSI). For pile-supported structures, it is commonly termed as Seismic-Soil–Pile-Superstructure–Interaction (SSPSI).

The effects of SSPSI on buildings can be assessed by comparing seismic response analyses results of building models with different base conditions viz. fixed-base and flexible-base models, whereby the effects of SSPSI are ignored in the former case and considered in the later. For the fixed-base model the FFM is taken as a FIM, however for the flexible-base model the FFM is modified due to kinematic interaction effects to yield the FIM following one of SSPSI analysis methods, viz. the substructure approach. Flexible-base models are idealized by assigning springs and dashpots at foundation support levels of the structures to simulate soil-foundation stiffness and damping respectively.

This study basically assesses the influence of SSPSI on the seismic response of buildings supported on pile foundations using the substructure approach. Five idealized building models of different heights and lateral force resisting systems are considered in this study. In order to avoid unnecessary complications due to torsion, the building models considered are regular both in geometry and stiffness. These models are (G+5) – ordinary moment resisting framed building and (G+10), (G+15), (G+20), and (G+30) – dual frame-shear wall structures. The structural analyses of the building models are done using the commercial structural analysis software SAP V-14 using dynamic response spectrum analysis method. A sensitivity study is conducted to identify the key factors which influence the seismic responses of building structures supported on pile foundations when soil deformability is considered. Finally important conclusions and recommendations are drawn.

1.2. Research Motivation

Use of pile foundations for buildings is becoming increasing in Ethiopia, especially in the capital city Addis Ababa, where soft soil deposits are prevalent and taller buildings are emerging. It is known that the type and stiffness of foundations affect seismic responses of buildings besides other factors like seismic source conditions, seismic-wave travel path and site soil effects. The dynamic stiffness and damping characteristics and hence the seismic response behaviors of piles are quite complex and different from shallow foundations. Hence, seismic responses of pile supported structures are also expected to be different from similar structures supported on shallow foundations. This necessitates studying the influence of soil flexibility on the seismic responses and internal force distributions of pile supported buildings. Besides, it is worth identifying the key parameters which influence the same.

1.3. Research objectives

General objective:

The main objective of this work is to study the influence of soil flexibility on the seismic response and internal force and moment distributions, the natural periods, base shears, and overturning moments of pile supported building structures.

Specific objectives:

The specific objectives include:

- To investigate the influence of SSPSI on the seismic response of pile supported buildings,
- To identify the key parameters which influence the seismic response of pile supported building structures when the effects of SSPSI are considered.

1.4. Scope of the study

In this study, dynamic Response Spectrum Analysis (RSA) approach for regular reinforced concrete building models is followed. Both floating (friction) and end-bearing piles are included. Only a (2X2) pile configuration is considered for the purpose of this study. It is partly due to complications in computing impedance functions for bigger pile group configurations as dynamic pile-to-pile interactions develop among piles of adjacent pile groups which cannot be easily handled and partly because rotational impedance functions for other pile group configurations are not readily available.

1.5. Research Methodology

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

1. Idealized building models of (G+5) – framed structure, (G+10), (G+15), (G+20), and (G+30) - dual frame-shear wall structures, assumed to be located in Addis Ababa are employed. All the models are symmetrical with respect to plan, elevation and lateral rigidity in order to avoid unnecessary complications due to torsion. As a continuation to previous studies of Gashaye (2005) and Solomon (2007) their building models, except the (G+15) model are adopted here.
2. Pile foundations are designed for the building models. Floating piles are designed for the (G+5) and (G+10) building models and end-bearing piles are designed for the rest.

3. Ethiopian Building Code Standard (EBCS) 8, 1995, Eurocode (EC) 8, 2004, and other relevant research journals are studied to relate the soil classification methods of EBCS 8 with EC8 codes so that EC8 response spectra is used with due consideration of the EBCS 8 provisions for structural analyses of the building models using the commercial structural analysis software SAP.
4. Impedance functions for both floating and end-bearing piles and pile groups are calculated using formulas compiled from literature.
5. The fixed-base building models are analyzed using SAP V-14 structural analysis software and their seismic responses obtained.
6. The flexible-base building models assumed to be supported on pile foundations are analyzed using SAP V-14 structural analysis software and their seismic responses obtained.
7. Finally analyses results obtained from the fixed-base and flexible-base model buildings are systematically compared. Besides, the critical parameters which influence seismic responses of buildings supported on pile foundations when soil flexibility is considered are identified through sensitivity study. Finally, conclusions are drawn and recommendations made.

1.6. Organization of the Thesis

The thesis is organized in five chapters. Chapter 1 covers the background statement of the problem, motivations, objectives, scope of the study and methodology. A comprehensive literature review on Seismic Soil-Pile-Structure Interaction (SSPSI) analysis, its effects and methods of accounting for it are covered in Chapter 2. In Chapter 3, the building models studied, provisions of SSPSI in building codes and relation of EBCS 8 with EC 8 (2004) are presented. Chapter 4 is devoted to sensitivity study, interpretation of results and comparisons of the analyses results. Finally conclusions are drawn and recommendations for future works are presented in Chapter 5.

2. LITERATURE REVIEW

2.1. General

The response of a structure to earthquake shaking is affected by interactions between three linked systems: the structure, the foundation and the geologic media underlying and surrounding the foundation. A SSFSI analysis evaluates the collective response of these systems to a specified free-field ground motion. In general, three SSFSI effects can be important in engineering (Bozorgnia and Bertero 2004; NIST GCR 12-917-21 2012).

Foundation stiffness and damping: Inertia developed in a vibrating structure gives rise to base shear, moment and torsional excitation, and these loads in turn cause displacements and rotations of the foundation relative to the free field. These relative displacements and rotations are only possible because of compliance in the soil-foundation system, which can significantly contribute to the overall structural flexibility in some cases. Moreover, the relative foundation-free field motions give rise to energy dissipation via radiation damping (i.e., damping associated with wave propagation into the ground away from the foundation, which acts as the wave source) and hysteretic soil damping, and this energy dissipation can significantly affect the overall system damping. Since these effects are rooted in the structural inertia, they are referred to as inertial interaction effects. Its effect in most structures is to increase total displacement due to the additional soil deformation, and to decrease the base shear demand due to the associated reduced structural inertia forces as a result of the additional energy dissipation into the soil (Worku 2014a).

Variations between free-field and foundation-level ground motions: The differences between foundation and free-field motions result from two processes. The first is known as kinematic interaction, and results from the presence of stiff foundation elements on or in soil, which cause foundation motions to deviate from free-field motion as a result of base-slab averaging and embedment effects. Ideally, the foundation motion should be used as input motion in the analysis of structures. However, studies have shown that the difference between the two motions can be regarded as negligible and hence, the free-field motion is used as the input ground motion in practice (Worku 2014a). The second process is related to the structure and foundation inertia, and consists of the relative foundation-free field displacements and rotations described above.

Foundation deformations: Flexural, axial and shear deformations of foundation elements occur as a result of loads applied by the superstructure and the supporting soil medium. Such deformations represent the seismic demand for which foundation components should be designed. These deformations can also significantly affect the overall system behavior, especially with respect to damping.

In almost every seismic building code, the structure response and foundation loads are computed neglecting SSFSI and the dynamic response is obtained merely from a fixed base analysis of the structure. The belief is that SSFSI always plays a favorable role in decreasing the inertia forces which is clearly related to the standard shape of code spectra which almost invariably possess a gently descending branch beyond a constant spectral acceleration plateau. Lengthening of the period, due to SSFSI, moves the response to a region of smaller spectral accelerations. However, in certain seismic and soil environments the perceived beneficial role of SSFSI is an oversimplification that may become detrimental in certain seismic and soil environments and there is evidence that some structures founded on unusual soils are vulnerable to SSFSI (Gazetas 2006; Gazetas and Mylonakis 1998). Due to this uncertainty, it is recommended to conduct a case by case study of the effects of SSFSI instead of generalizing that consideration of SSFSI is beneficial or detrimental a priori (Gazetas 1994).

2.2. Kinematic interaction

In the free-field, an earthquake will cause soil displacements in both the horizontal and vertical directions. The kinematic interaction or scattering effect happens due to the inability of the foundation, owing to its stiffness and rigidity, to conform to the free-field ground motion causing variation of ground motion with depth and scattering of waves at the corners of the foundation. The other causes of these deviations are base-slab averaging, in which spatially variable ground motions within the building envelope are averaged within the foundation footprint due to the stiffness and strength of the foundation system and embedment effect, in which foundation-level motions are reduced as a result of ground motion reduction with depth below the free surface (Kramer 1996).

Fan et al. (1991) summarized the results of a series of previous numerical studies on the kinematic response of vertical piles in elastic soil subjected to vertically incident harmonic shear waves and bonded to a massless rigid cap suspended above the ground. The results were presented as a set of dimensionless graphs that enable evaluation of the effects of relative pile rigidity $\left(E_p/E_s\right)$, pile slenderness (pile length/pile diameter, L/d), soil layering, pile spacing $\left(S/d\right)$, pile head fixity and number of piles, where, E_p and E_s are the Young's modulus of pile and soil respectively, and S is centre-to centre spacing between piles . These results generally indicate significant effects of $\left(E_p/E_s\right)$, head fixity and soil layering on the kinematic response of single free-head piles subject to vertically incident shear waves. The effect of L/d was relatively minor.

Horizontally propagating waves have negligible impacts in kinematic interaction effects on pile foundations, as wave length of practical interest, $\lambda > 6d$ where d is the pile diameter, so that the base slab averaging effect is negligible for piles, moreover, the axial stiffness of piles is considerably high (Dobry and Gazetas 1988). But, vertically propagating SH waves have considerable impact on piles and it has been found that the kinematic interaction between pile and soil has, in general, two consequences (Dobry and Gazetas 1998):

i. It filters out low-period (i.e. high frequency) components of the motion while at the same time it induces a rotational component at the pile head.
ii. It induces axial, bending, and shear deformation on piles. Bending is significant at two locations: at the top of fixed-head piles and at the interfaces of soil layers with sharply different stiffnesses.

So, the kinematic response of a pile or pile group needs to be calculated at the pile head and at depth separately. The response at the pile head is an input into the inertial response analysis; and the response at depth may be used to assess the structural requirement of the pile in the intermediate and deep zones.

Kinematic interaction effects are exactly zero for shallow foundations in a seismic environment consisting exclusively of vertically propagating shear waves or dilatational waves; but more pronounced in pile and very stiff embedded shallow foundations of structures having two or more subterranean levels (Kramer & Stewart 2004; NIST GCR 12-917-21 2012). Gazetas (1984) has demonstrated that when piles are flexible with respect to the surrounding soil, kinematic interaction is significant for small to medium frequencies.

2.3. Inertial interaction

Inertial interaction is manifested when the superstructure starts to vibrate as a result of inertial forces triggered by the excitation at the foundation level. The inertial forces distributed over the height of the structure cause a resultant base shear and an overturning moment at the foundation, which in turn cause deformation of the soil. This deformation initiates new waves propagating into the soil mass. These waves carry away part of the energy imparted on the structure by the incoming earthquake waves and act as a means of energy dissipation in addition to the material/hysteretic damping inherent in the system. This form of SSFSI is known as inertial SSFSI. Inertial displacements and rotations can be a significant source of flexibility and energy dissipation in the soil-structure system (NIST GCR 12-917-21 2004).

The structure-to-soil stiffness ratio, $h/(V_s T)$ can be used as a relative measure for determining when SSFSI effects will become significant. In this expression, h is the structure height, V_s is the soil shear wave velocity; and T is the fixed-base building period. When $h/(V_s T) > 0.1$, SSI can significantly lengthen the building period and modify (i.e., generally increase) damping in the system. This will modify the design base shear (up or down, depending on spectral shape) and the distribution of force and deformation demands within the structure, relative to a fixed-base analysis. When using the structure-to-soil stiffness ratio, it is important to recognize that the ratio is an approximate relative measure, and not an absolute criterion. Even when $h/(V_s T) < 0.1$, relative distributions of moments and shear forces in a building can be modified relative to the fixed-base condition, particularly in dual systems, structures with significant higher-mode responses, and subterranean levels of structures (NIST GCR 12-917-21, 2012).

2.4. Analysis of Seismic Soil-Pile-Structure Interaction (SSPSI)

Generally there are two approaches to evaluate the effects of SSPSI: the direct or complete interaction analysis and the multistep or substructure method of analysis. In a direct analysis, the soil and structure are included within the same model and analyzed in a single step. The soil is often discretized with solid finite elements and the structure with finite beam elements. Because assumptions of superposition are not required, true nonlinear analyses are possible although the analyses are more typically performed using equivalent linear soil properties. The need for a large model, energy absorbing boundaries, and detailed soil properties, makes its use prohibitive for all but the most extreme analysis demands (Kramer 1996; Wilson 2002).

In a substructure analysis, the SSPSI problem is broken down into three distinct parts that are combined to formulate the complete solution. In this method the effective input motion is expressed in terms of FFM of the soil layer initially. In continuation to this step, the soil/foundation medium and the superstructure are represented as two independent mathematical models or substructures as shown in Figures 2.1 – 2.3. The connection between them is provided by interaction forces of equal amplitude, acting in opposite directions of the two sub-structures.

The total motions developed at the interface are the sum of the FFM at the interface of the soil without the added structure and the additional motions resulting from the interaction. As the principle of superposition is employed, this method is limited to the analysis of linear systems. However, it is often applied to nonlinear systems using strain-compatible equivalent-linear systems (Gazetas 1991). The substructure method is easy to handle than the direct method, as it allows breaking down the complicated soil-foundation-structure system into more manageable parts which can be more easily solved and checked. The Substructure method is followed in this study and briefly discussed in the sequel.

2.5. Analysis of SSPSI using substructure approach

For computational convenience and conceptual simplicity, the two stages of interactions are subdivided into two independent analyses steps following the principle of superposition, as shown in Figures 2.1 -2.3 (Mylonakis et al 1997; Gazetas et al 1993).

i. **For the kinematic interaction response:**

   (a1) Analysis of the free-field soil response (i.e. without the presence of piles) to vertically propagating S-waves, as other waves have less influence on the kinematic interaction of pile foundations; and

   (a2) Analysis of the interaction of single pile or pile group with the surrounding soil, driven by the free-field response of step (a1) without the presence of the superstructure,

ii. For the inertial interaction response:

(b1) Computation of the dynamic impedances ('springs' and 'dashpots') at the pile head or the pile-group cap, associated with the swaying ($\Im_x$ and $\Im_y$), rocking ($\Im_{ry}$ and $\Im_{rx}$), cross-swaying-rocking ($\Im_{xry}$ and $\Im_{yrx}$), axial ($\Im_z$) and torsional ($\Im_t$) (i.e. rotation about the vertical axis) motions of the foundation; and

(b2) Analysis of the dynamic response of the superstructure supported on the 'springs' and 'dashpots' of step (b1) subjected to the kinematic pile-head motion of step (a2). The latter is also called Foundation Input Motion (FIM).

2.5.1. Analysis of kinematic interaction of pile foundations

Kinematic interaction effects are described by a frequency dependent transfer function relating the FFM to the FIM of a hypothetical system which differs from the complete actual system in that, the mass of the superstructure is set equal to zero. The kinematic interaction is solved using the displacement and rotation kinematic interaction factors given by Gazetas (1984) for three soil models. So, calculation of the kinematic response of piles comprises of finding the FFM and calculation of the interaction factors to modify this FFM to get the FIM. The FIM is then simply obtained by multiplying the free-field response spectra with the calculated interaction factors.

a) Free-field ground motion

Normally the free field motion at a specific location is obtained by 1D, 2D or 3D wave propagation theories. However, in the frequency domain analysis response spectra can be used as a FFM (Han 2004).

b) Pile head motion

Kinematic interaction is expressed in terms of kinematic amplification factors as (Gazetas 1984):

$$A_u = \frac{u_p}{u_g} \tag{2.1}$$

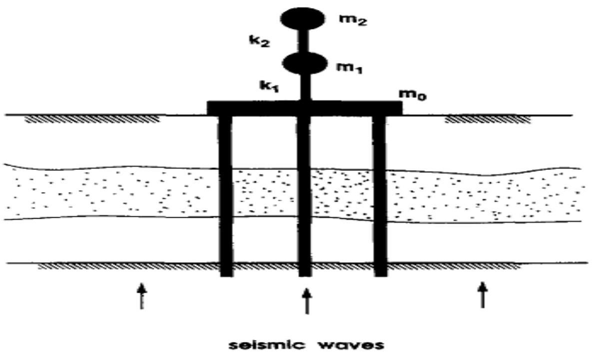


Figure 2.1: Seismic soil-pile-foundation-structure interaction: the whole system (Gazetas et al 1993)

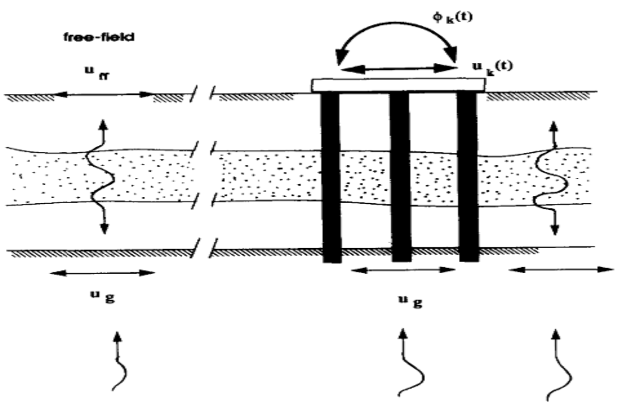


Figure 2.2: Seismic soil-pile foundation-structure interaction: kinematic seismic response analysis (Gazetas et al 1993)

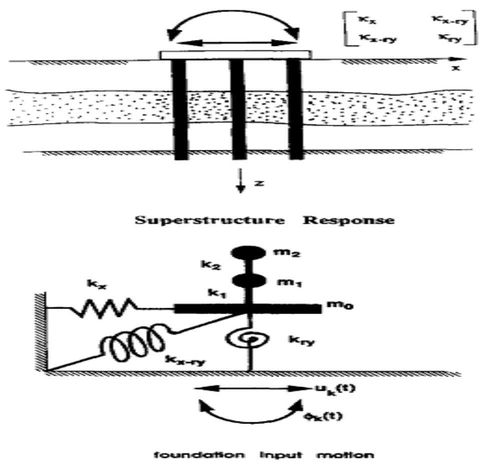


Figure 2.3: Seismic soil-pile-foundation-structure interaction: inertial response analyses (Gazetas et al 1993)

N.B. The dynamic impedance functions designated in the figure above by $k_{\alpha\beta}$ are replaced with $\Im_{\alpha\beta}$ in this study.

$$A_\theta = \frac{\theta_p D}{2u_o} \tag{2.2}$$

Or it can be better expressed in terms of kinematic interaction factors as:

$$I_u = \frac{u_p}{u_o} \tag{2.3}$$

$$I_\theta = \frac{D\theta_p}{2u_o} \tag{2.4}$$

Where: u_p = the amplitude of the horizontal displacement of the pile head relative to the input at the base of the pile u_g; the total displacement of the pile head is $u_g(t)+u_p(t)$

u_o = the amplitude of the free field motion of the ground surface adjacent to the pile head.

D = diameter of the pile

θ_p = the rotation of the pile head.

Simplified kinematic interaction factors for the three types of soil models as shown in Figure 2.4, which idealize different distributions of Young's modulus with depth are given as a function of the following frequency parameters (Gazetas 1984). However, this study focuses on constant stiffness distribution model, i.e. Soil model C shown in Figure 2.4.

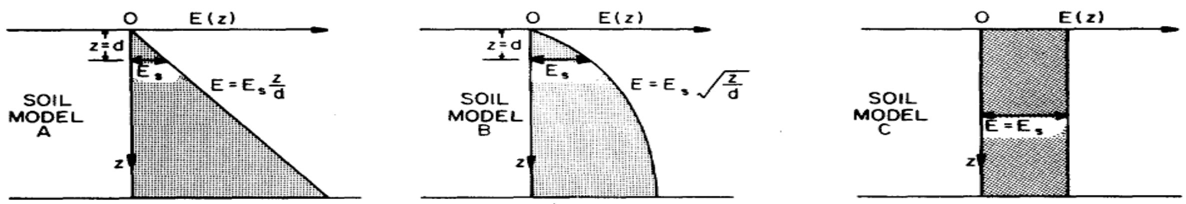


Figure 2.4: Idealized Soil models (Gazetas 1984)

$$\left.\begin{aligned} F_A &= \left[\frac{f}{f_1}\right] k^{0.10} \mathcal{L}^{-0.40} \\ F_B &= \left[\frac{f}{f_1}\right] k^{0.16} \mathcal{L}^{-0.35} \\ F_C &= \left[\frac{f}{f_1}\right] k^{0.30} \mathcal{L}^{-0.50} \end{aligned}\right\} \qquad (2.5)$$

Where: F_A, F_B, F_C = Non dimensional kinematic interaction factors for Linear, Parabolic and Constant soil stiffness distribution models respectively as depicted in Figure 2.5.

f = excitation frequency

f_1 = first natural shear frequency of the layer

k = the ratio E_p/E_{SD} ,

E_{SD} = the soil modulus at depth equal to the pile diameter, D.

$\mathcal{L} = L/D$

L = length of pile

The first natural shear frequencies for the three soil profile models are given below (Gazetas 1984).

$$\left.\begin{aligned} f_{1A} &= 1.21V_s/H \\ f_{1B} &= 0.56V_s/H \\ f_{1C} &= 0.25V_s/H \end{aligned}\right\} \qquad (2.6)$$

Where: V_s = S-wave velocity at depth $z = d$ below the ground surface

f_1 = first mode frequency

H = layer thickness

Using the formulas given in equation 2.5, values of F are calculated for frequency ranges of interest and then the interaction factor I_u read from the curves depicted in Figure 2.5 for the pertinent soil models. Alternatively, the interaction factor I_u can also be calculated from equation 2.7. However, the interaction factors produced by this procedure are strictly applicable only to a Fourier spectrum (Pender 1993; Madabhushi et al 2009).

$$I_u = aF^4 + bF^3 + cF^2 + e \qquad (2.7)$$

With a minimum value of $I_u = 0.5$,

Where: F represents the value from equation 2.5.

The values of the coefficients a, b, c, and e which give a reasonable fit to the curves in Figure 2.5 are given in Table 2.1.

Table 2.1: Coefficients for horizontal kinematic interaction factors (Gazetas 1984)

Coefficient	Soil Stiffness distribution		
	Constant	Parabolic	Linear
a	0	3.64 X 10-6	-6.75 X 10-5
b	0	-4.36 X 10-4	-0.007

c	-0.21	0.006	0.033
d	1	1	1

Studies reveal that the free head results presented in Figure 2.5 and given in equation 2.7 are conservative assessments of the kinematic interaction factors for fixed head piles and also the Kinematic interaction factor for a pile group is similar to that for an individual pile in the group. So, for kinematic seismic loading the group effect for pile foundations could for all practical purposes be neglected; and the rotational kinematic interaction factors, I_θ, are sufficiently small to be neglected (Pender 1993; Madabhushi et al 2009). As seen from Figure 2.5, the higher frequencies are damped by the piles as a result of the kinematic interaction effect and also the interaction factors are reduced with soil inhomogeniety.

2.5.2. Analysis of inertial interaction of pile foundations

In the substructure method of SSPSI analysis the flexibility of the foundation soil and associated damping with foundation-soil interaction is described by frequency dependent dynamic impedance functions. Impedance functions are defined as the complex amplitudes of harmonic forces (or moments) that have to be applied at the pile head in order to generate a harmonic motion with a unit amplitude in the specified direction. The pile head impedances are often used as foundation spring and dashpot parameters in the analysis of superstructures subjected to FIM. The effects of inertial interaction can be assessed by comparing the seismic response analyses results of the fixed-base models with the flexible-base models which are supported on springs and dashpots representing the stiffness and damping, respectively of the soil-foundation system. So, the first step of inertial interaction analysis is computation of the "spring and dashpot" of soil-foundation system which are combined in complex notation to result impedance functions.

2.6. Dynamic impedance functions of foundations

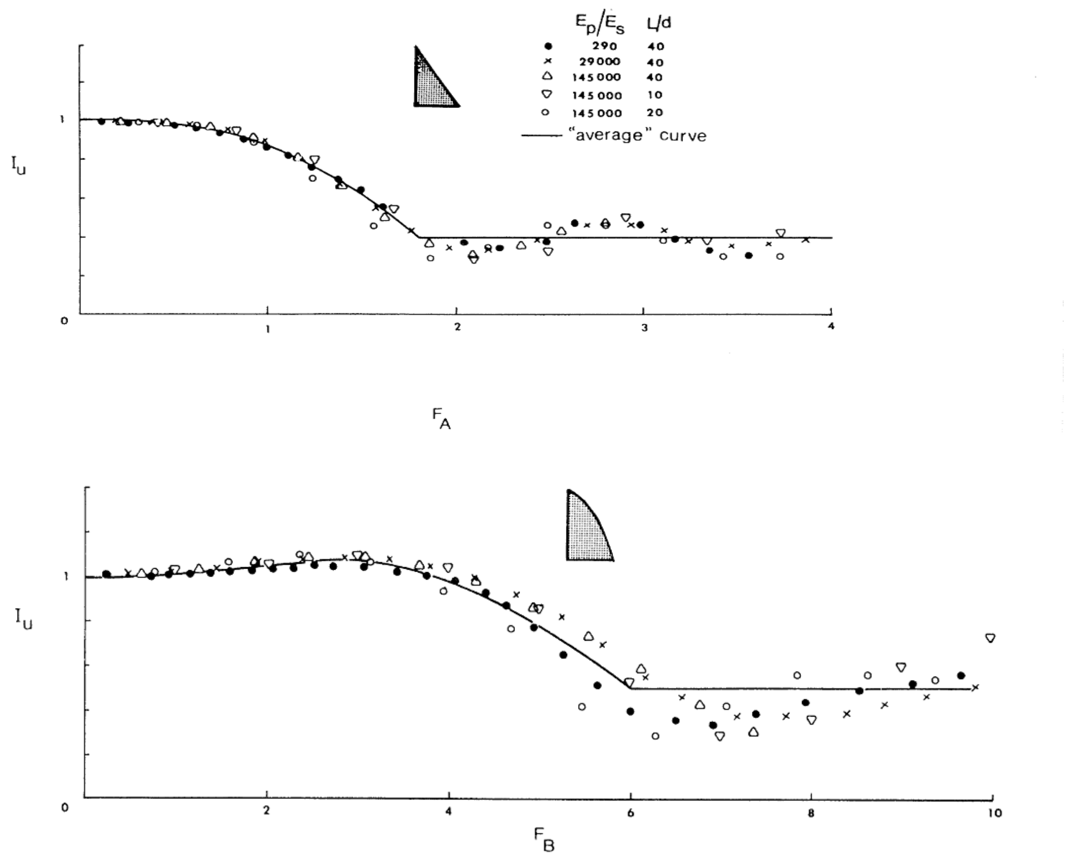
In substructuring method of SSPSI analysis the response of a foundation to horizontal inertial loading and moments is determined by combining the stiffness and damping terms using impedance functions. Impedance functions are conveniently expressed as a complex variable because the damping component, being a function of velocity, is out of phase with the elastic stiffness.

Terms in the impedance function are expressed in the form (Pender 1993; Worku 2014a):

$$p(t) = \Im(\omega)u(t) \qquad (2.8)$$

With $\Im(\omega) = \bar{K}(\omega) + i\omega C \qquad (2.9)$

Where: $\Im$ is impedance function for mode of response (sliding, rocking etc.), $\bar{K}$ is the dynamic pile stiffness, C is the damping coefficient, ω is circular frequency in radians/second ($\omega = 2\pi f$ where f is frequency of excitation), i is an imaginary number in complex number notation = $\sqrt{-1}$, $P(t)$ is the dynamic excitation force, and $u(t)$ is the dynamic response, displacement or rotation.



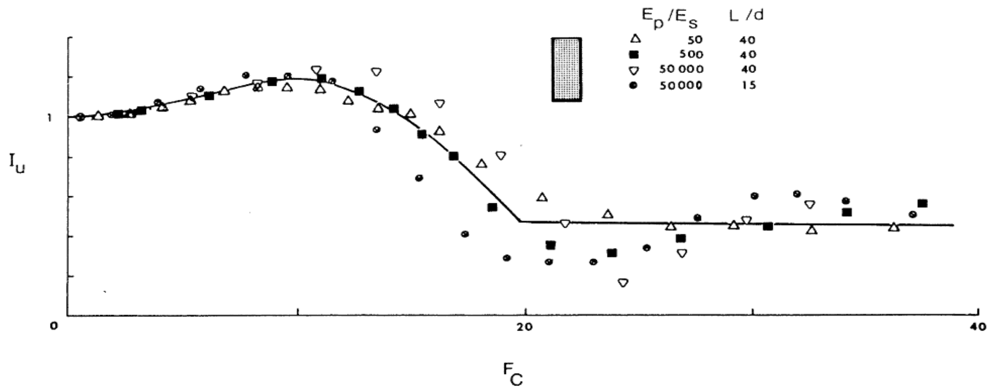


Figure 2.5: Kinematic interaction factors (I_u) for free head piles in terms of dimensionless frequency factors F_A, F_B, and F_C (Gazetas 1984)

The damping may also be expressed as dimensionless frequency dependent coefficients, $\zeta(\omega)$ for the various modes of response where:

$$\zeta(\omega) = \frac{\pi f C}{K} = \frac{\omega C}{2K} \tag{2.10}$$

This enables an alternative expression for the impedance to be developed:

$$\Im(\omega) = K_{\alpha\beta}\left[k_{\alpha\beta}(\omega) + 2\zeta_{\alpha\beta}(\omega)i\right] \tag{2.11}$$

where: $K_{\alpha\beta}$ is a component of the static pile head stiffness matrix, $k_{\alpha\beta}(\omega)$ is a frequency dependent dynamic stiffness coefficient relating the real part of $\Im$ to $K_{\alpha\beta}$, $\zeta_{\alpha\beta}(\omega)$ is the effective damping ratio of the system and $\alpha\beta$ refers to the various modes of vibrations, axial, rocking, torsional, etc.

In equation 2.9, the real component reflects the stiffness and inertia of the supporting soil; its dependence on frequency is attributed solely to the influence which frequency has on inertia, since soil properties are essentially frequency independent (Gazetas 1983). The imaginary component reflects the radiation and material damping of the system, the former being the result of energy dissipation by waves propagating away from the foundation, and it is frequency dependent. The latter, arising chiefly from the hysteretic cyclic behavior of soil, depends on strain but it is practically frequency independent (Gazetas 1983; Pender et al 2010). While the single pile stiffness is not sensitive to frequency, the pile group interaction terms and the radiation damping are frequency dependent.

Numerical studies undertaken by Gazetas (1984) show that $k_{\alpha\beta}(\omega)$, is approximately unity for most practical values of pile – soil stiffness ratio over the frequencies of interest and for the horizontal, rocking and vertical modes.

2.6.1. Dynamic impedance functions of pile foundations

The foundation system is idealized with six degrees of freedom system, three translational and three rotational: dynamic displacements along the axes, x, y, and z, and dynamic rotations around the same axes. The steady-state responses of foundations to dynamic forces and moments transmitted from the superstructure can be easily computed once the dynamic impedances, $\Im_x, \Im_y, \Im_z, \Im_{rx}, \Im_{ry}$ and $\Im_t$ associated with swaying (in x and y axes), vertical (in z axis), rocking (about x and y axes), and torsional (rotation about z axis) oscillations, respectively, have been derived. Moreover, in embedded foundations and piles, horizontal forces along principal axes induce rotational in addition to translational oscillations; hence, two more "cross-coupling" horizontal-rocking impedances exist: $\Im_{xry}$ and $\Im_{yrx}$. They are usually negligibly small in shallow foundations, but their effects may become appreciable for greater depths of embedment, owing to the moments about the base axes produced by horizontal soil reactions against the sidewalls. In piles the "cross-coupling" impedances are as important as the "direct" impedances (Gazetas 1991). The eight impedance terms are represented by a 6 X 6 matrix relating the force-moment vector with the displacement-rotation vector.

Several researchers suggest different approaches to compute dynamic stiffnesses of single piles and pile groups. The approaches differ mostly in the assumptions made and modeling of the soil-pile-structure problem. Though it is not the focus of this study to exhaustively summarize different formulations of computing dynamic stiffnesses of piles and pile groups, some approaches are discussed as follow.

a) Novak (1974) formulated a simple approach based on plane strain soil reactions, which can be interpreted as dynamic Winkler springs and dashpots attached directly to the pile. Piles are considered as frequency independent equivalent springs based on Novak's (1974) formulation. Novak has shown that the pile stiffness and the damping are almost the same for fixed and pinned pile heads if the slenderness ratio (L/R) of the pile is larger than about 25, where L and R are respectively the length and radius of the pile. The application of the same approach to a vertically vibrating pile (Novak 1977) indicated the great sensitivity of the pile behavior to tip condition.

b) Kaynia and Kausel (1982) developed rigorous methods of solution to three-dimensional dynamic boundary-value problems for piles and pile groups interacting with soil. The method is described as a boundary-integral-type formulation, which uses Green's functions, defining the displacement fields due to uniform unit loads acting on an elemental cylindrical surface and on a circular disk. The Green's functions are computed by solving the wave equations through Fourier and Hankel transformations. The solutions describe the dynamic soil flexibility matrix that is combined with the pile flexibility matrix. The method gives the complex valued impedance functions for rotation and horizontal translation for a given soil profile which can be implemented into standard structural programs (Bentley, 1999).

Kaynia and kausel (1982) give dynamic stiffness and damping of pile groups graphically for S/d ratios of 2, 5 and 10; and square pile configurations of 2X2, 3X3 and 4X4 piles groups. They assumed that the soil medium is a viscoelastic halfspace with υ_s = 0.40 and β_s = 0.05, and the piles are made of elastic materials with υ_p = 0.00. In addition, it is assumed that ρ_s/ρ_p = 0.70 and L/D = 15, as well as two soil conditions (soft soil: E_s/E_p = 10^{-3}; stiff soil E_s/E_p = 10^{-2}) and the two types of pile –to - cap connections (fixed or hinged).

c) Gazetas (1991) made a complete survey of foundation vibration problems and for practical use, summarized simplified closed form algebraic formulas and dimensionless graphs for static and dynamic stiffness, and dashpot coefficients to be used for computation of impedance functions of piles and shallow foundations having a wide range of geometries for different modes of oscillations, and embedment conditions for the three soil profiles mentioned above.

Gazetas and co-workers (Dobry and Gazetas 1988; Makris & Gazetas 1992; Gazetas 1991; Gazetas et al 1993) developed a Beam-on-Dynamic-Winkler-Foundation (BDWF) simplified model to determine the impedance functions of single piles and pile groups. The soil is modeled as a Winkler foundation resisting the pile motion by continuously distributed frequency dependent linear springs and dashpots along the pile length. The coefficients of these springs and dashpots are frequency dependent and determined using algebraic expressions, developed by matching the dynamic pile-head displacements from Winkler and finite element analyses. The form of the damping coefficients is first determined from the cone model by Wolf et al. (1992) and the numerical values are then calibrated by curve-fitting finite element results. The spring constants are derived solely through curve fitting to rigorous numerical results.

d) Taherzadeh et al (2002) give simple formulas for computing dynamic stiffness of pile groups based on the construction of a general model of impedance matrices as the condensation of matrices of mass, damping and stiffness, and on the identification of the values of these matrices on an extensive database of numerical experiments computed using coupled Finite Element-Boundary Element (FE-BE) models. The formulations obtained can be readily used for design of both floating piles on homogeneous half-space and end-bearing piles, and are applicable for a wide range of mechanical and geometrical parameters of the soil and piles, in particular for large pile groups.

Each of the above approaches has its own limitations and is not complete by itself to fully compute the pile stiffness matrix. For further and complete review of different approaches refer to Gazetas and Mylonakis (1998), Mylonakis and Gazetas (1998), or Pender (1993).

2.6.1.1. Dynamic impedance functions of floating single piles

Studies reveal that, similar to static loads, dynamic lateral loads at pile heads do not deform piles over their entire length. Instead, pile deformations and stresses reduce to negligible proportions below a distance l_a, called dynamic "active length", from the ground surface. For piles of length $L > l_a$ commonly termed as flexible piles, the exact pile length L, have no influence on their response to lateral loads. The active pile length is in the order of 10 to 20 pile diameters, depending on pile-soil stiffness contrast, soil non-homogeneity, and fixity conditions at the pile head (Randolph 1981; Gazetas 1991; Pender 1993; Syngros 2004; Karatzia and Mylonakis 2012).

Flexible piles essentially behave as infinitely long beams, and the actual length does not affect flexural response. Active lengths tend to be greater for dynamic loading than for static loading, due to the ability of elastic waves to travel further down the pile than a static stress field. Axially loaded piles tend to respond to much greater depths (in excess of 50 pile diameters), and tip reaction is almost always mobilized. Accordingly, an axially-loaded pile cannot usually be approximated by an infinite rod.

Expressions for active pile length for lateral deformations can be cast in the form (NIST GCR 12-917-21 2012):

$$l_a = \Lambda \left(\frac{E_p}{E_s} \right)^{\mu} d \qquad (2.12)$$

Where Λ and μ are dimensionless constants, and all other terms are as previously defined. For fixed-head piles in homogeneous soil under static loading, Randolph (1981) and Fleming et al. (1992) recommend Λ = 1.8 and μ = 0.25. For dynamic loading, Gazetas (1991) recommends Λ = 2 and μ= 0.25. Based on a more accurate set of finite-element analyses, Syngros (2004) recommends Λ = 2.4 and μ = 0.25. Approximate values of active pile length l_a, are 10d to 20d for lateral loading, and the actual pile length, L, for axial loading (NIST GCR 12-917-21 2012).

Worku (2013) gives a set of analytical formulas for the estimation of foundation model parameters by synthesizing mechanical models at three different levels, viz. Winkler, Pasternak and Kerr, with corresponding variants of a generalized continuum model. The generalized continuum model was derived using a unified approach on the basis of a subgrade idealized as an elastic layer of finite thickness overlying a rigid base without making prior simplifying assumptions. It has been demonstrated that the calibration factor for each model type can be established from comparative analytical-numerical studies and the values obtained in this manner are suggested for practical use. in routine analysis of beam-like and plate-like shallow foundations and rigid pavements.

Worku's (2013) approach yields $K_x = 0.82E_s$ for floating piles whereas, Gazetas et al (1993) suggested $K_x = 1.2E_s$, where K_x is modulus of subgrade reaction in the horizontal direction, and E_s is the modulus of elasticity of the soil. This newer approach of Worku (2013) likely results in reduced single and group pile impedances, pile responses and resonant frequency of vibration as compared with results obtained using inflated modulus as suggested by Gazetas et al (1993). Thus using of this relatively reduced modulus might solve the discrepancies between the observed and computed responses of piles under dynamic loads as reported by Puri and Prakash.

Due to its relative simplicity for practical use, and its availability of impedance functions for almost all modes of vibrations, the approaches of Gazetas are followed in this work.

Gazetas (1991) gives simple closed form expressions for static stiffness, dynamic stiffness coefficients, and dashpot coefficients to be used for computation of impedance functions of piles for different modes of oscillations using curve fits to rigorous numerical results as presented in Table 2.2. Gazetas (1984) has shown that for frequencies less than the natural frequency of the soil stratum there is no radiation damping; and only material damping exists in the soil adjacent to the pile. For frequencies higher than the natural frequency of the layer, radiation damping, which increases with increasing frequency, is added to the material damping. Pender (1993) stated that equations given by Gazetas (1991) for calculation of damping are conservative and under-predict the damping for swaying, rocking and swaying-rocking vibrations by 30%. Pender (1993) recommends increasing the damping obtained using the equations given by Gazetas (1991), by 30% for those vibration modes described herein above.

2.6.1.2. Dynamic impedance functions of floating pile groups

The impedance of a pile group cannot be determined by simple addition of individual pile impedances because grouped piles interact through the soil by "pushing" or "pulling" each other through waves emitted from their periphery. This is called a group effect, and it can significantly affect the impedance of a pile group as well as the distribution of head loads among individual piles in the group. Group effects depend primarily on pile spacing, frequency, and number of piles. They are more pronounced in the elastic range, and dynamic group effects decrease in the presence of material nonlinearity (NIST GCR 12-977-21 2012).

In engineering practice, three approaches are used to obtain the dynamic impedances of pile groups (Gazetas et al 1993): (a) the superposition method using the static interaction factors and ignoring the frequency dependence of pile-pile interaction; (b) superposition using (simplified or rigorous) dynamic interaction factors; and (c) direct numerical solutions.

In principle the last method is more rigorous, but a sophisticated computer code is usually needed. The first and second methods are conceptually simple. They use the dynamic impedances of single piles as the basis and account for the group effect by means of interaction factors (static or dynamic) (Gazetas et al 1993). The group impedance function is then used in a global structural analysis, which produces forces and deflections on the pile group. Pile group interaction factor has been observed to have significant effect on the dynamic response of the system specifically when the pile spacing is between 2.5D to 3D, where D is the diameter of the pile (Chowdhury and Shambhu 2009).

The dynamic stiffness of a pile group, in any vibration mode, can be calculated using the dynamic stiffness of single pile in conjunction with dynamic interaction factors. This method, originally introduced for static loads by Poulos (1968), and later validated for dynamic loads by Kaynia & Kausel (1982), Sanchez-Salinero (1983) and Roesset (1984), can be used with confidence - at least for groups of fewer than 50 piles. Dynamic interaction factors for various modes of loading are available in the form of non-dimensional graphs (Gazetas et al 1991) and, in some cases, closed-form expressions derived from a beam on a Winkler foundation model in conjunction with simplified wave propagation theory (Makris & Gazetas 1992).

Kaynia and Kausel (1982) derived dynamic interaction factors for vertical floating pile groups. They defined a dynamic interaction factor for two piles in which the first ('active pile') is loaded with a unit harmonic load, and the displacements are observed on the second ('passive') pile as:

$$\text{Interaction factor} = \frac{\text{Dynamic displ.of pile 2}}{\text{Static displ.of pile 1,considered individually}} \qquad (2.13)$$

In which the word displacement stands for either a translation or a rotation. Their method is based on the observation that

$$\frac{Displ. of\ pile\ 1}{Displ.\ of\ pile\ 1, considered\ individually} \approx 1$$

That is, the second pile hardly affects the displacements of the loaded pile.

The horizontal, vertical and rotation interaction factors are presented as complex-valued frequency dependent ratios of the dynamic displacement of active pile to the static displacement of passive pile, due to a unit harmonic load on passive pile. Kaynia and Kausel (1982) concluded that, the superposition scheme gives reasonable results for dynamic loads as for static loads. Their study also shows that pile groups are less influenced by near-surface ground conditions than isolated piles, group interaction effects are stronger for softer soils, and radiation damping increases with foundation size. They also gave charts that show the dynamic load distributions among piles in a group. Contrary to static loading, in which the corner piles carried the greatest load and the centre piles the smallest, they showed that the load sharing depends on the dimensionless frequency, a_o that for certain frequency ranges the piles closest to the center take the largest portion of the load.

Dobry and Gazetas (1988) developed a simplified analytical method for computing dynamic impedance functions of floating rigidly capped pile groups taking the pile-soil-pile interactions into account through frequency dependent interaction factors, assuming that cylindrical wave propagation governs vibration of source pile and displacement of neighboring pile. Using two identical piles, p and q, separated by a distance S between axes the effect of vibration of pile p on the response of pile q can be conveniently expressed through dynamic interaction factor α_v, which is a function of frequency

$$\alpha_v = \alpha_v(\omega) = w_{qp} / w_{qq} = \frac{additional\ displacement\ of\ pile\ q\ caused\ by\ pile\ p}{displacement\ of\ pile\ q\ under\ own\ dynamic\ load} \qquad (2.14)$$

The dynamic interaction factors for different modes of vibrations are shown in Table 2.2.

Gazetas (1984) stated that the pile group impedances predicted by using static interaction factors are close to those from the numerical solutions only at very low frequencies. At intermediate and high frequencies use of static interaction factors leads to erroneous results.

So, dynamic impedances of pile groups are better predicted using dynamic interaction factors approach and the procedure is discussed as follows.

i) Dynamic pile group interaction factor method for translational, vertical and rocking vibrations

The response of a pile group subjected to an arbitrary harmonic force was obtained from the dynamic interaction factors derived from the study of only two piles at a time, assuming that the other piles, beside the two studied, are transparent (Dobry and Gazetas 1988). The inputs required in the method are the dynamic impedance of a single pile and shear wave velocity, Poisson's ratio and damping ratio of the soil.

Dobry and Gazetas (1988) assumed that, in vertical vibrations, cylindrical waves emanate from the pile perimeter along the pile length at equal time and propagate radially outward in the horizontal direction. However, in lateral vibrations, both P and SH-waves are generated, as shown in Fig. 2.6.

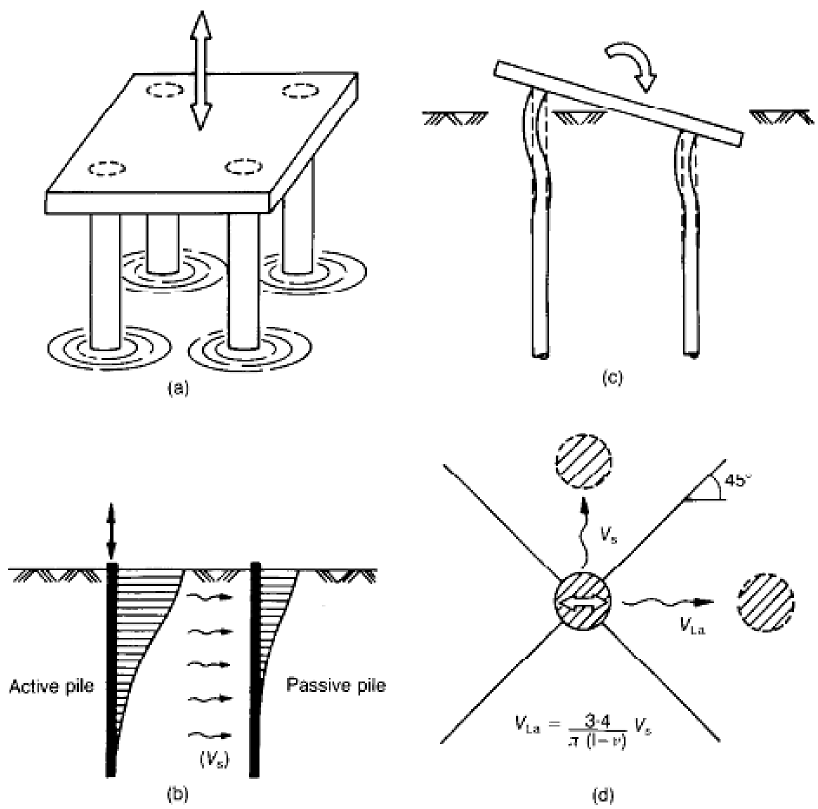


Figure 2.6 (a) analogy between the cylindrical wave assumption for group of piles in soil and cylindrical water waves; (b) distribution of displacement amplitudes along the shaft of an oscillating (active) pile and of a neighboring (passive) pile are assumed to be of the same shape; (c) pile-head deformation and reactions during rocking; (d) assumed apparent velocities of waves emanating from a laterally oscillating pile (Dobry and Gazetas, 1988).

The P –waves are generated in the direction of shaking and propagate with apparent phase velocity which is approximately equal to the so-called Lysmer's analog velocity, $V_{La} = 3.4V_s / [\pi(1-\upsilon)]$. Whereas, the SH waves are generated perpendicular to the shaking and travel with the shear wave velocity of the soil, Vs, as shown in Figure 2.6. Gazetas and Makris (1991) and Makris and Gazetas (1992) also developed simplified methods for pile group axial and lateral dynamic responses respectively for a harmonic excitation at the pile head and a seismic-type excitation. The dynamic interaction factors for the different modes of vibrations, obtained by Dobry and Gazetas (1988) are compiled and shown in Table 2.2.

Impedance functions for axial, swaying and rocking vibrations of a (2 X 2) pile group are obtained by considering the fact that displacements or rotations experienced by all the piles are equal due to the rigidity of the pile cap. The dynamic impedance functions of pile groups for different modes of vibrations are given as under (Dobry and Gazetas 1988).

- **For axial vibration**

$$\Im_z^G = \frac{4\Im_z^S}{1+2\alpha_v(S)+\alpha_v\left(S\sqrt{2}\right)} \tag{2.15}$$

Where: $\Im_z^s$ is vertical impedance function for single pile,

α_v is the interaction factor for axial vibration (obtained from Table 2.3),

S is center to center spacing between adjacent piles,

$S\sqrt{2}$ is the distance between the diagonal piles and

$\alpha_V(S)$ and $\alpha_V(S\sqrt{2})$ are interaction factors to be calculated at spacings of S and $S\sqrt{2}$ respectively.

- **For rocking vibration**

$$\Im_{rx}^{G} = 4\Im_{rx}^{s} + \Im_{z}^{s} S^{2} \frac{1}{1-\alpha_{v}\left(S\sqrt{2}\right)} \tag{2.16}$$

Where: $\Im_{rx}^{s}$ is rocking impedance function for a single pile

$\Im_{z}^{s}$ is axial impedance function for a single pile as defined above.

- **For lateral vibration**

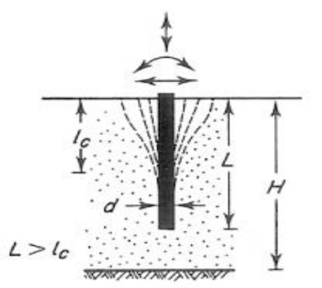
$$\Im_{h}^{G} = \frac{4\Im_{h}^{s}}{1+\alpha_{v}\left(S\right)+0.5\alpha_{v}\left(S\sqrt{2}\right)+\alpha_{ho}\left(S\right)+0.5\alpha_{ho}\left(S\sqrt{2}\right)} \tag{2.17}$$

Where: $\Im_{h}^{s}$ is lateral impedance function for a single pile.

α_{ho} is interaction factor for lateral vibration at an angle of 0°.

Note that, for the cross swaying-rocking modes of vibration, the dynamic interaction factors are zero.

Table 2.2 Dynamic Stiffness And Damping Coefficients For Flexible Piles ($L > l_c$) (Gazetas, 1991).

 	Linear Increase of Soil Modulus with Depth $E = \tilde{E}_s \frac{z}{d}$ $\tilde{V}_s = \left[\tilde{E}_s / 2\rho(1+\upsilon)\right]^{1/2}$	Parabolic Increase of Soil Modulus with Depth $E_s = \tilde{E}_s \sqrt{\frac{z}{d}}$ $\tilde{V}_s = \left[\tilde{E}_s / 2\rho(1+\upsilon)\right]^{1/2}$	Constant Soil Modulus at All Depths $E_s =$ constant
"Active length"	$l_c \approx 2d\left(E_p / \tilde{E}_s\right)^{0.20}$	$l_c \approx 2d\left(E_p / \tilde{E}_s\right)^{0.22}$	$l_c \approx 2d\left(E_p / E_s\right)^{0.25}$
Natural Shear frequency of deposit	$f_s = 0.19 V_{SH} / H$ where V_{SH} = the S-wave velocity at depth Z = H (bottom of stratum)	$f_s = 0.223 V_{SH} / H$ where V_{SH} = the S-wave velocity at depth Z = H (bottom of stratum)	$f_s = 0.25 V_S / H$
Static lateral (swaying) stiffness	$K_{HH} = 0.6d\,\bar{E}_s\left(E_p / \bar{E}_s\right)^{0.35}$	$K_{HH} = 0.8d\,\bar{E}_s\left(E_p / \bar{E}_s\right)^{0.28}$	$K_{HH} = dE_s\left(E_p / E_s\right)^{0.21}$
Lateral (swaying) dynamic stiffness coefficient	$k_{HH} \approx 1$	$k_{HH} \approx 1$	$k_{HH} \approx 1$
Lateral (swaying) dashpot coefficient:			

$C_{HH} = 2K_{HH}D_{HH}/\omega$	$\begin{cases} D_{HH} \approx 0.60\beta + 1.80\, fd\tilde{V}_s,\ for \\ f > f_s \\ D_{HH} \approx 0.60\beta,\ for,\ f \le f_s \end{cases}$	$\begin{cases} D_{HH} \approx 0.70\beta + 1.20\, fd\left(\frac{E_p}{E_s}\right)^{0.08} \tilde{V}_s^{-1} \\ for,\ f > f_s \\ D_{HH} \approx 0.70\beta\ for,\ f \le f_s \end{cases}$	$\begin{cases} D_{HH} \approx 0.80\beta + 1.10\, fd\left(\frac{E_p}{E_s}\right)^{0.17} \tilde{V}_s^{-1} \\ for,\ f > f \\ D_{HH} \approx 0.80\beta\ for,\ f \le f_s \end{cases}$ s
Static rocking stiffness Rocking dynamic stiffness coefficient Rocking dashpot coefficient: $C_{MM} = 2K_{MM}D_{MM}/\omega$	$K_{MM} = 0.15d^3\tilde{E}_s\left(E_p/\tilde{E}_s\right)^{0.80}$ $k_{MM} \approx 1$ $\begin{cases} D_{MM} \approx 0.20\beta + 0.40\, fd\tilde{V}_s^{-1} \\ ,\ for,\ f > f_s \\ D_{MM} \approx 0.20\beta,\ for,\ f \le f_s \end{cases}$	$K_{MM} = 0.15d^3\tilde{E}\left(E_p/\tilde{E}_s\right)^{0.77}$ $k_{MM} \approx 1$ $\begin{cases} D_{MM} \approx 0.22\beta + 0.35\, fd\left(E_p/\tilde{E}_s\right)^{0.10} \tilde{V}_s^{-1} \\ for,\ f > f_s \\ D_{MM} \approx 0.22\beta,\ for,\ f \le f_s \end{cases}$	$K_{MM} = 0.15d^3E_s\left(E_p/\tilde{E}_s\right)^{0.75}$ $k_{MM} \approx 1$ $\begin{cases} D_{MM} \approx 0.35\beta + 0.35\, fd\left(E_p/E_s\right)^{0.20} \tilde{V}_s^{-1} \\ for,\ f > f_s \\ D_{MM} \approx 0.25\beta,\ for,\ f \le f_s \end{cases}$
Static swaying-rocking cross stiffness Swaying-rocking cross dynamic stiffness coefficient Swaying-rocking dashpot coefficient:	$K_{HM} = K_{MH} = -0.17d^2\tilde{E}_s\left(E_p/\tilde{E}_s\right)^{0.60}$ $k_{HM} = K_{MH} \approx 1$	$K_{HM} = K_{MH} = -0.24d^2\tilde{E}_s\left(E_p/\tilde{E}_s\right)^{0.53}$ $k_{HM} = K_{MH} \approx 1$	$K_{HM} = K_{MH} = -0.22d^2\tilde{E}_s\left(E_p/E_s\right)^{0.5}$ $k_{HM} = k_{MH} \approx 1$

$C_{HM} = 2K_{HM}D_{HM}/\omega$	$\begin{cases} D_{HM} \approx 0.30\beta + fd\tilde{V}_s \\ for,\ f > f_s \\ D_{HM} \approx 0.30\beta,\ for,\ f \le f_s \end{cases}$	$\begin{cases} D_{HM} \approx 0.60\beta + 0.70\,fd\left(\frac{E_p}{\tilde{E}_s}\right)^{0.05}\tilde{V}_s^{-1} \\ for,\ f > f_s \\ D_{HM} \approx 0.35\beta,\ for,\ f \le f_s \end{cases}$	$\begin{cases} D_{HM} \approx 0.80\beta + 0.85\,fd\left(\frac{E_p}{E_s}\right)^{0.18}\tilde{V}_s^{-1} \\ for,\ f > f_s \\ D_{HM} \approx 0.50\beta,\ for,\ f \le f_s \end{cases}$
Static axial stiffness	The axial stiffness of a pile depends not only on its relative compressibility (E_p/E_s) but also on the slenderness ratio L/d and the tip support conditions (end-bearing versus floating). See the pertinent geotechnical literature for a proper estimation of the static stiffness. The expressions given herein are only for estimates of the axial stiffness of floating piles in a homogeneous stratum of total thickness H ≈ 2L.		
	$K_z \cong 1.8E_{sL}d\left(\frac{L}{d}\right)^{0.55}\left(\frac{E_p}{E_{sL}}\right)^{-\left(\left(\frac{L}{d}\right)/\frac{E_p}{E_{sL}}\right)}$ $E_{sL} = \tilde{E}_s\left(L/d\right)$	$K_z \cong 1.9E_{sL}d\left(\frac{L}{d}\right)^{0.60}\left(\frac{E_p}{E_{sL}}\right)^{-\left(\left(\frac{L}{d}\right)/\frac{E_p}{E_{sL}}\right)}$ $E_{sL} = \tilde{E}_s\sqrt{\left(\frac{L}{d}\right)}$	$K_z \cong 1.9E_s d\left(\frac{L}{d}\right)^{2/3}\left(\frac{E_p}{E_s}\right)^{-\left(\left(\frac{L}{d}\right)/\frac{E_p}{E_{sL}}\right)}$
Axial radiation dashpot coefficient	$C_z = \frac{2}{3}a_o^{1/3}\rho V_{sL}\pi dLr_d$ $for,\ f > f_r$ where: $r_d \approx 1 - e^{-2\left(E_p/E_{sL}\right)\left(L/d\right)^{-2}}$ $C_z \approx 0,\ for,\ f \le f_r$	$C_z = \frac{3}{4}a_o^{-1/4}\rho V_{sL}\pi dLr_d$ $for,\ f > 1.5\,f_r$ where: $r_d \approx 1 - e^{-1.5\left(E_p/E_{sL}\right)\left(L/d\right)^{-2}}$ $C_z \approx 0,\ for,\ f \le f_r$	$C_z = a_o^{-1/5}\rho V_s\pi dLr_d,$ $for,\ f > 1.5\,f_r$ where: $r_d \approx 1 - e^{-\left(E_p/E_s\right)\left(L/d\right)^{-2}}$ $C_z \approx 0,\ for,\ f \le f_r$ linearly interpolate for

	linearly interpolate for $f_r < f < 1.5 f_r$	linearly interpolate for $f_r < f < 1.5 f_r$	$f_r < f < 1.5 f_r$
Interaction factor α_z for axial in-phase oscillations of the two piles	$\alpha_z \approx \sqrt{2}\left(\frac{S}{d}\right)^{-3/4} e^{-0.5\beta\omega S/V_{sL}} e^{-i\omega\sqrt{2}S/V_{sL}}$	$\alpha_z \approx \sqrt{2}\left(\frac{S}{d}\right)^{-2/3} e^{-(2/3)\beta\omega S/V_{sL}} e^{-i\omega\sqrt{2}S/V_{sL}}$	$\alpha_z \approx \sqrt{2}\left(\frac{S}{d}\right)^{-1/2} e^{-\beta\omega S/V_s} e^{-i\omega S/V_s}$
	V_{sL} = the S-wave velocity at depth $z = L$; $\tilde{V}_s = V_s$ at pile mid-length; S = axis-to-axis pile separation; β = soil hysteretic damping. Note: although α_z are complex numbers their use is identical to the familiar use of static interaction factors introduced by Poulos.		
Interaction factor α_{HH} for lateral in-phase oscillation	Very little information presently available	Very little information presently available	$\alpha_{HH}(90^\circ) \approx (3/4)\alpha_z$ $\alpha_{HH}(0^\circ) \approx 0.5\left(\frac{S}{d}\right)^{-1/2} e^{-\beta\omega S/V_{La}} e^{-i\omega S/}$ $\alpha_{HH}(\theta^\circ) \approx \alpha_{HH}(0^\circ)\cos^2\theta + \alpha_{HH}(90^\circ)\sin^2\theta$
Interaction factor α_{MM} for in-phase rocking, and α_{HM} for swaying-rocking	$\alpha_{MM} \approx \alpha_{MH} \approx 0$	$\alpha_{MM} \approx \alpha_{MH} \approx 0$	$\alpha_{MM} \approx \alpha_{MH} \approx 0$

- $\tilde{E}_s$ and $\tilde{V}_s$ (for the two inhomogeneous deposits) denote Young's modulus and S-wave velocity, respectively, at depth.

ii) Dynamic torsional impedance function of floating pile groups

The torsional dynamic stiffness and damping for a (2X2) pile group as given graphically by Kaynia and Kausel (1982) is presented in Figure 2.7. The torsional impedances are normalized with $\sum r_i^2 k_{xx}^s\left(a_o=0\right)$ as shown in the figure, where r_i is distance of pile i to the torsional axis, $k_{xx}^s\left(a_o=0\right)$ is the static translational stiffness of single pile and $k_{\psi\psi}^G$ and $c_{\psi\psi}^G$ are stiffness and damping of the pile group for torsional mode of vibration.

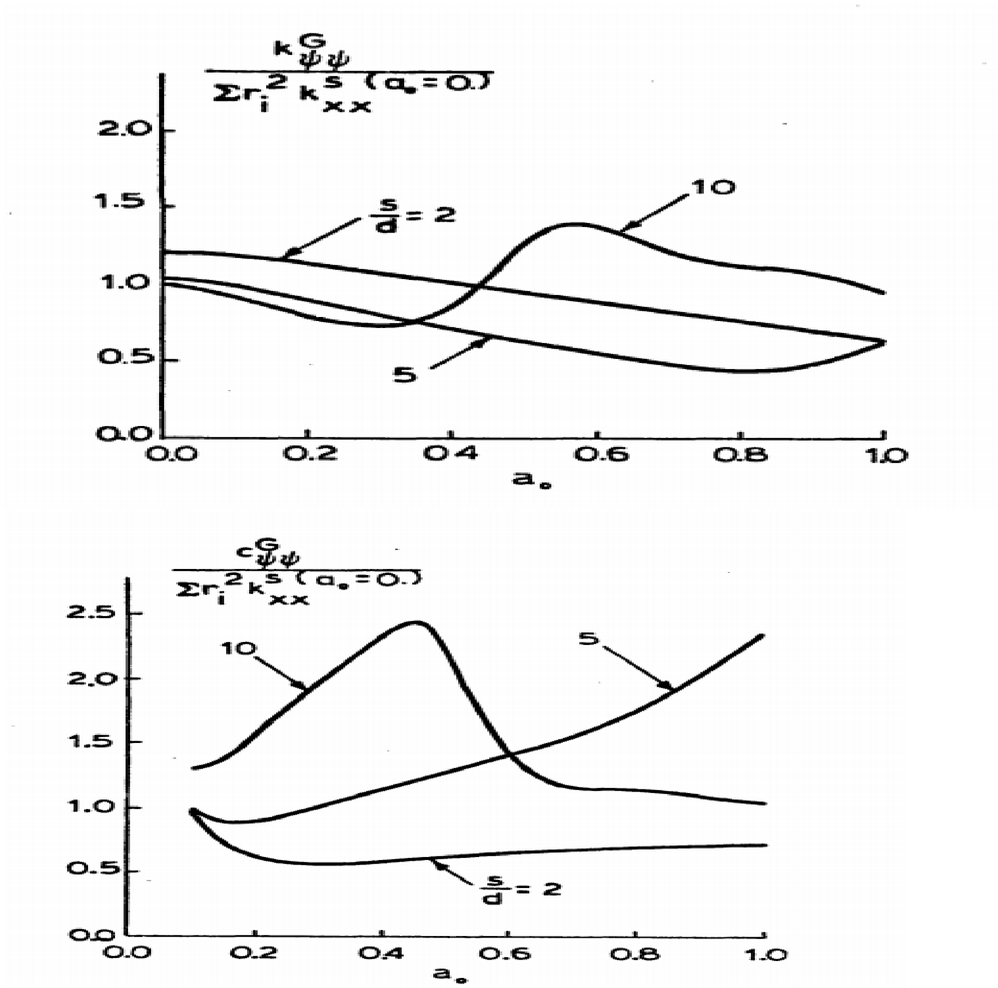


Figure 2.7: Torsional dynamic impedance of (2X2) pile group in soft soil medium (Kaynia and Kausel 1982)

2.6.1.3. Dynamic impedance functions of end-bearing piles

Gazetas (1991) stated that the closed form expressions for axial stiffness and interaction factors for different modes of vibrations are limited to floating piles though the expressions for lateral, rocking and swaying-rocking cross stiffnesses and the associated dashpot coefficients can be used for both floating and end-bearing piles. Due to the scarcity of comprehensive latest reference material on the impedance functions of end-bearing single and group impedance functions, the approach of Novak et al (1983) is followed to compute single and group pile impedance functions in this work.

Novak et al. (1983) provided charts for the dimensionless parameters for pile stiffness and damping as functions of frequency, mass ratio and pile slenderness. Novak's formulations are based on plain strain assumptions for floating piles (Novak 1974), and latter for end-bearing piles and pile groups for different modes of vibrations (Novak 1983). Novak's dynamic impedance functions formulas for end-bearing piles are presented as follow:

i) Vertical vibration

For vertical single end bearing piles undergoing vertical motion, the spring coefficients are given by (Novak et al 1983):

$$k_{z1} = \left(\frac{E_p A_p}{r_o} \right) f_{18,1} \tag{2.18}$$

Where: k_{z1} = equivalent vertical spring constant for end bearing piles,

E_p = Young's modulus of pile material,

A_p =cross sectional area of the pile,

r_o = equivalent radius of the pile, and

$f_{18,1}$ = a factor as given in Table 2.3. It depends on pile material (concrete, steel, timber etc.), ratio of embedded length l to radius r_o and V_s/V_c (ratio of shear wave velocity of the soil above the tip to compression wave velocity in pile).

The damping coefficient in vertical direction is given by:

$$C_{z1} = \left(\frac{E_p A_p}{V_s} \right) f_{18,2} \tag{2.19}$$

Where: C_{z1} = damping coefficient of the end bearing piles,

V_s =shear wave velocity of the soil through which the pile is driven,

V_c = compression wave velocity in the pile,

$f_{18,2}$ = is a factor as given in Table 2.3.

Table 2.3: Values of factor f as per Novak (1974) for axial stiffness and damping factor for single pile. For concrete piles $\left(\gamma_s/\gamma_p = 0.7\right)$ having $l/r_o > 25$ (Novak et al 1983).

Slenderness ratio	Stiffness and damping function f for vertical bearing pile
20	$f_{18,1} = 3.75(V_s/V_c)^2 - 0.05(V_s/V_c) + 0.0501$
	$f_{18,2} = 15.345(V_s/V_c)^{2.0928}$
50	$f_{18,1} = 6.25(V_s/V_c)^2 - 0.05(V_s/V_c) + 0.0199$
	$f_{18,2} = -10(V_s/V_c)^2 + 1.5(V_s/V_c) - 0.012$
100	$f_{18,1} = -3.75(V_s/V_c)^2 + 0.45(V_s/V_c) + 0.0061$
	$f_{18,2} = 1.4(V_s/V_c) - 0.0083$

Effect of pile group on the vertical stiffness and dashpot coefficients depends upon the relative distance between the piles itself and the slenderness ratio of the piles carrying the loads and is expressed as (Novak 1983).

$$k_z^g = \frac{\sum_1^N k_{z1}}{\sum_1^N \alpha_A} \tag{2.20}$$

The equivalent damping for the pile group is given by

$$C_z^g = \frac{\sum_1^N c_{z1}}{\sum_1^N \alpha_A} \tag{2.21}$$

Where: N = number of piles in a group,

α_A = displacement interaction factor (axial) for a typical reference pile in the group relative to itself and to all other piles in the group assuming the reference pile and all other piles are loaded to same magnitude.

The factor α_A can be evaluated from the expression below (Randolph and Poulos 1982) as also recommended by API 351R (Chowdhury et al 2009).

$$\alpha_A = \frac{0.5\ln\left(l_p/S\right)}{\ln\left(l_p d \rho_A\right)} \text{ for } S \le l_p \tag{2.22}$$

Where: l_p = Pile length,

S = spacing of piles,

d = diameter of pile,

$\rho_A = G_{av}/G_b$,

G_{av} = Average shear modulus along pile depth and

G_b =Shear modulus at pile base.

Alternatively the value can also be deduced from Poulos's interaction curve for static interaction (Pecker et al).

The above equations can be directly used when the pile cap is not in contact with the ground. However, if the pile cap is assumed to be in contact with the ground the equations ought to be modified to take the effect of the pile cap embedment into consideration. Neglecting the effect of pile cap embedment seems more realistic unless the soil medium is of good quality and the underside of the cap is filled with good quality granular material. This is so as both cohesive and non cohesive soils shrink and settle respectively, during seismic vibration.

ii) Translational vibration

For vibration in horizontal direction the expression for stiffness and damping is as shown below (Novak 1983):

$$k_{x1} = \left(\frac{E_p I_p}{r_o^3}\right) f_{11,1} \text{ for } l/r_o \geq 25 \tag{2.23}$$

$$c_{x1} = \left(\frac{E_p I_p}{r_o^2 v_s}\right) f_{11,2} \text{ for } l/r_o \geq 25 \tag{2.24}$$

Here I_p is the moment of inertia of the pile cross section about the centroidal axis perpendicular to the x-direction, which is the direction of motion. $f_{11,1}$ and $f_{11,2}$ are factors for fixed headed piles as furnished in Table 2.4.

The group effect for horizontal direction is expressed as:

$$k_x^g = \frac{\sum_1^N k_{x1}}{\sum_1^N \alpha_L} \text{ and} \tag{2.25}$$

$$c_x^g = \frac{\sum_1^N C_{x1}}{\sum_1^N \alpha_L} \tag{2.26}$$

Table 2.4: Values of factor f - as per Novak (1974) stiffness and damping factors for horizontal and rocking modes

Poisson's ratio	Function, f
0.25	$f_{11,1}=7.25(V_s/V_c)^2+0.38(V_s/V_c)-0.0013$ $f_{11,2}=17(V_s/V_c)^2+0.915(V_s/V_c)-0.0032$ $f_{7,1}=-55(V_s/V_c)^2+9.3(V_s/V_c)+0.1075$ $f_{7,2}=-38.75(V_s/V_c)^2+6.55(V_s/V_c)+0.0734$ $f_{9,1}=-1.81(V_s/V_c)$ $f_{9,2}=0.375(V_s/V_c)^2-2.67(V_s/V_c)+0.0005$
0.4	$f_{11,1}=7.875(V_s/V_c)^2+0.43(V_s/V_c)-0.0015$ $f_{11,2}=18.75(V_s/V_c)^2+1.02(V_s/V_c)-0.0037$ $f_{7,1}=-57.5(V_s/V_c)^2+9.65(V_s/V_c)+0.1113$ $f_{7,2}=-41.25(V_s/V_c)^2+6.85(V_s/V_c)+0.0746$ $f_{9,1}=-1.94(V_s/V_c)$ $f_{9,2}=0.75(V_s/V_c)^2-2.87(V_s/V_c)+0.0006$

Where, α_L = a displacement factor for lateral motion defined in similar way to α_A and is given by

$$\alpha_{Lf}=0.6\rho_c\left[E_pG_c\right]^{\frac{1}{7}}\left(\frac{r_o}{S}\right)\left(1+\cos^2\beta_p\right) \tag{2.27}$$

$$\alpha_{LH}=0.4\rho_c\left[E_pG_c\right]^{\frac{1}{7}}\left(\frac{r_o}{S}\right)\left(1+\cos^2\beta_p\right) \tag{2.28}$$

$$\alpha_{\theta H}=\alpha_{LH}^2 \tag{2.29}$$

$$\alpha_{\theta M}=\alpha_{LH}^3 \tag{2.30}$$

Here $\rho_c = G_z / G_{av}$ where, G_z = Shear modulus at depth $l_c/4$

$l_c = 2r_o \left[E_p / G_c \right]^{\frac{2}{7}}$ and is known as the critical length of the pile where,

G_c = Average shear modulus over the critical length of the pile.

α_{Lf} = The horizontal interaction factor for fixed headed piles (no head rotation),

α_{LH} = The horizontal interaction factor due to horizontal force (rotation allowed),

$\alpha_{\theta H}$ = Interaction factor due to horizontal force for rotation.

$\alpha_{\theta M}$ = Interaction factor due to moment for rotation.

β_p = Angle subtended by a pile in pile group with respect to the reference pile.

When the calculated interaction factor α exceeds 1/3, its value needs to be replaced by $\alpha' = 1 - 2/\sqrt{27\alpha}$, a correction made to avoid α approaching infinity as S tends to zero. Alternatively, Poulos's interaction curve for static load case under horizontal load may also be used (Chowdhury et al 2009).

The stiffness and damping characteristics of the pile cap are expressed as

$$k_x^f = G_s h \bar{S}_{u1} \text{ and} \tag{2.31}$$

$$c_x^f = h r_o \bar{S}_{u2} \sqrt{G_s \gamma_s / g} \tag{2.32}$$

Where: G_s is shear modulus of the soil, and h is the embedment depth for the pile cap.

The factors $\bar{S}_{u1}$ and $\bar{S}_{u2}$ are given in Table 2.5.

Table 2.5: Values of $\bar{S}_{u1}$ and $\bar{S}_{u2}$ for various Poisson's ratios (Novak et al 1983)

Poisson's ratio	$\bar{S}_{u1}$	$\bar{S}_{u2}$
0.0	3.6	8.2
0.25	4	9.1
0.4	4.1	10.6

iii) Rocking vibration

The expressions for spring stiffness and damping for single piles in rocking motion are

$$k_{\psi 1} = \left(\frac{E_p I_p}{r_o} \right) f_{7,1} \tag{2.33}$$

$$c_{\psi 1} = \left(\frac{E_p I_p}{V_s} \right) f_{7,2} \tag{2.34}$$

Where: I_p is the moment of inertia of the pile cross section about the axis of rotation,

$f_{7,1}$ and $f_{7,2}$ are factors for rotational direction for fixed head piles, as furnished in Table 2.4.

iv) Torsional vibration

For a pile group, the group torsional stiffness is expressed as

$$k_{\psi}^{g} = \sum_{1}^{N} \left[k_{\psi 1} + k_{z1} X_r^2 + k_{x1} Z_c^2 - 2 Z_c k_{x\psi 1} \right] + k_{\psi f} \tag{2.35}$$

Where: X_r is half of the distance between the centerlines of adjacent piles,

Z_c is the distance from the underside of the pile cap to the center of gravity of the pile cap,

k_{z1} and k_{x1} are respectively the vertical and translational stiffness constants of single piles as defined earlier. In addition,

$$k_{x\psi 1} = \left(\frac{E_p I_p}{r_o^2} \right) f_{9,1} \quad \text{and} \tag{2.36}$$

$$k_{\psi f} = G r_o h \bar{S}_{\psi 1} + G_s r_o^2 h \left[\left(\frac{\delta^2}{3} \right) + \left(\frac{Z_c}{r_o} \right)^2 - \delta \left(\frac{Z_c}{r_o} \right) \right] \bar{S}_{u1} \tag{2.37}$$

Where: $\delta = h/r_o$,

h = embedment depth of pile cap,

$\bar{S}_{\psi 1}$ is a factor as given in Table 2.6.

The damping matrix for the pile group is expressed by

$$c_{\psi}^{g} = \sum_{1}^{N} \left[c_{\psi 1} + c_{z1} X_r^2 + c_{x1} Z_c^2 - 2 Z_c c_{x\psi 1} \right] + c_{\psi f} \tag{2.38}$$

Where: c_{z1} and c_{x1} are damping constants of single piles as described earlier.

$$c_{x\psi 1} = \left(\frac{E_p I_p}{r_o V_s} \right) f_{9,2} \tag{2.39}$$

$$c_{\psi}^{f} = \delta r_o^4 \sqrt{G_s \gamma_s / g} \left\{ \bar{S}_{\psi 2} + \left[\left(\frac{\delta^2}{3} \right) + \left(\frac{Z_c}{r_o} \right)^2 - \delta \left(\frac{Z_c}{r_o} \right) \right] \bar{S}_{u2} \right\} \tag{2.40}$$

Table 2.6: Values of $\bar{S}_{\psi 1}$ and $\bar{S}_{\psi 2}$ for various Poisson's ratios (Novak et al 1983).

Poisson's ratio	$\bar{S}_{\psi 1}$	$\bar{S}_{\psi 2}$
0.00	2.5	1.8
0.25	2.5	1.8
0.40	2.5	1.8

3. ANALYSES OF THE BUILDING MODELS STUDIED

3.1. The Building models studied

As indicated in section 1.5, five idealized building models having different heights and lateral force resisting systems but having similar plan geometries are adopted from previous studies of Gashaye (2005) and Solomon (2007). The models are regular both in geometry and stiffness to avoid unnecessary complications due to torsion. These are (G+5) – framed building and (G+10), (G+15), (G+20), and (G+30) – dual frame-shear wall structures, as shown in Figure 3.1. The building elements are all assumed to be made of cast in situ reinforced concrete, and a uniform storey height of 3m is used while the footing columns are assumed to be 1m high.

All the building models are assumed to be located at Addis Ababa and intended to function as classified under category A according to EBCS 1, 1995, Section 2.6.3.1.1 which assigns characteristic imposed loads for different building categories. Details of the building models studied and the loading conditions are given subsequently.

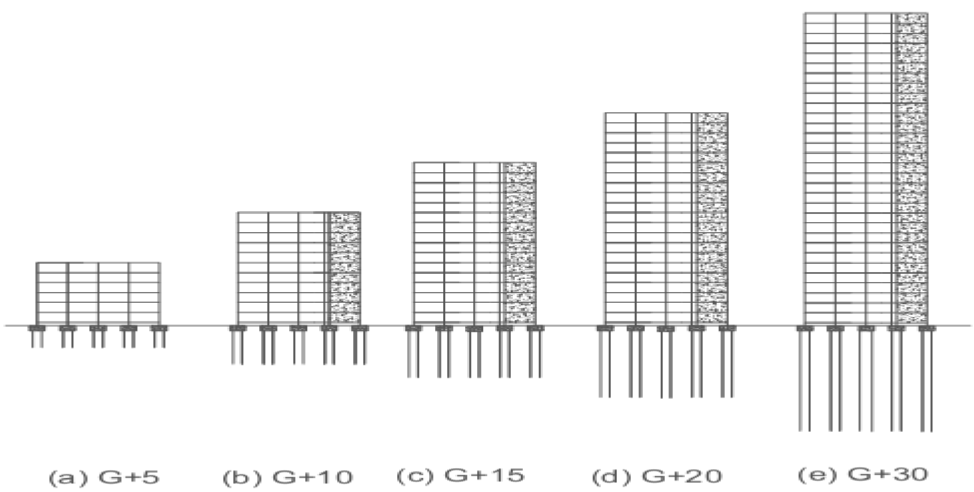


Figure 3.1: The building models studied

1. The first building model is a five-storey (G+5) reinforced concrete building of ordinary moment resisting frames supported on isolated footing or floating pile foundations as the case may be. Details of the model and the loading conditions are given in Table 3.1, and its geometry is shown in Figure 3.2.

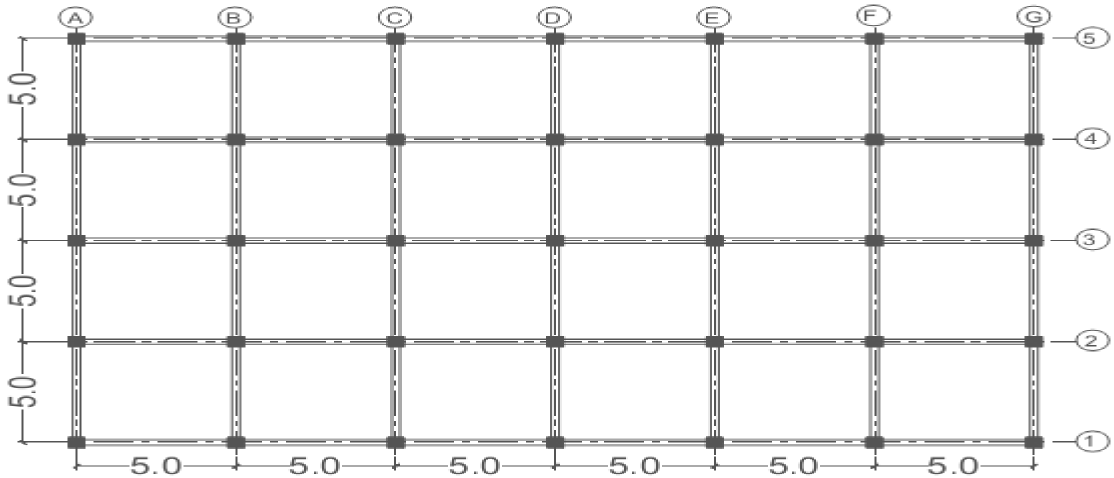


Figure 3.2: Typical structural layout of building Model 1 (G+5)

Table 3.1: Structural details and loading conditions of building model 1 (G+5)

Type of Structural System	Ordinary moment resisting framed structure
Beam Size	250 mm X 400 mm
Column Size	500 mm X 500 mm
Slab thickness	160 mm †
Shear Wall thickness	No shear wall
• Partition wall load	• 1.50 kN/m^2 is applied at all slabs except at roof terrace §
Static gravity loads on slabs • Live load • Floor finishing load	 • 3 kN/m^2 • 1.73 kN/m^2 at all floor slabs.

† The self weight of the slab is calculated and added by the program.

§ It is customary to represent partition weight with a uniform load of 1kN/m^2 or a uniform load computed from the actual or anticipated weights of the partitions placed in any probable position (ASCE 7-98; MacGregor 2005). Adding uniformly distributed load of 0.50 kN/m^2 to this load, a total uniformly distributed load of 1.50 kN/m^2 is applied on all slabs except at roof terrace slab in lieu of partition loads on slabs and wall loads on beams.

2. The second building model is a ten-storey (G+10) reinforced concrete building of dual frame-shear wall system supported on mat or floating pile foundations as the case may be. Details of the model and the loading conditions are given in Table 3.2, and its geometry is shown in Figure 3.3.

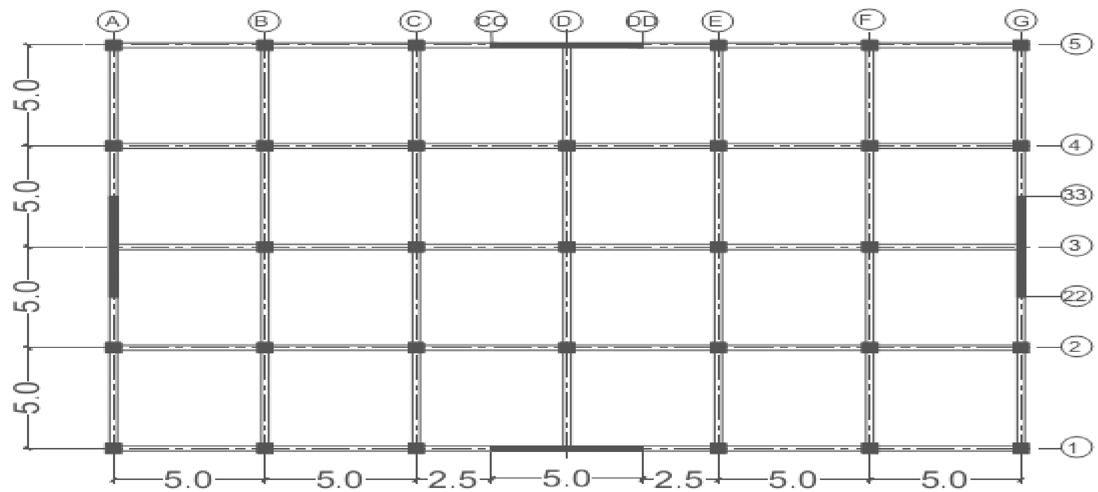


Figure 3.3: Typical structural layout of building Model 2 (G+10)

Table 3.2: Structural details and loading conditions of building model 2 (G+10)

Type of Structural System	Dual frame with shear wall
Beam Size	250 mm X 400 mm
Column Size	500 mm X 500 mm
Slab thickness	160 mm
Shear wall thickness	200 mm
• Partition wall load	• 1.50 kN/m^2 is applied at all slabs except at roof terrace
Static gravity loads on slabs • Live load • Floor finishing load	 • 3 kN/m^2 • 1.73 kN/m^2 at all floor slabs.

3. The third building model is a fifteen-storey (G+15) reinforced concrete building of dual frame-shear wall system supported on mat or end-bearing pile foundations as the case

may be. Details of the model and the loading conditions are given in Table 3.3, and its geometry is shown in Figure 3.4.

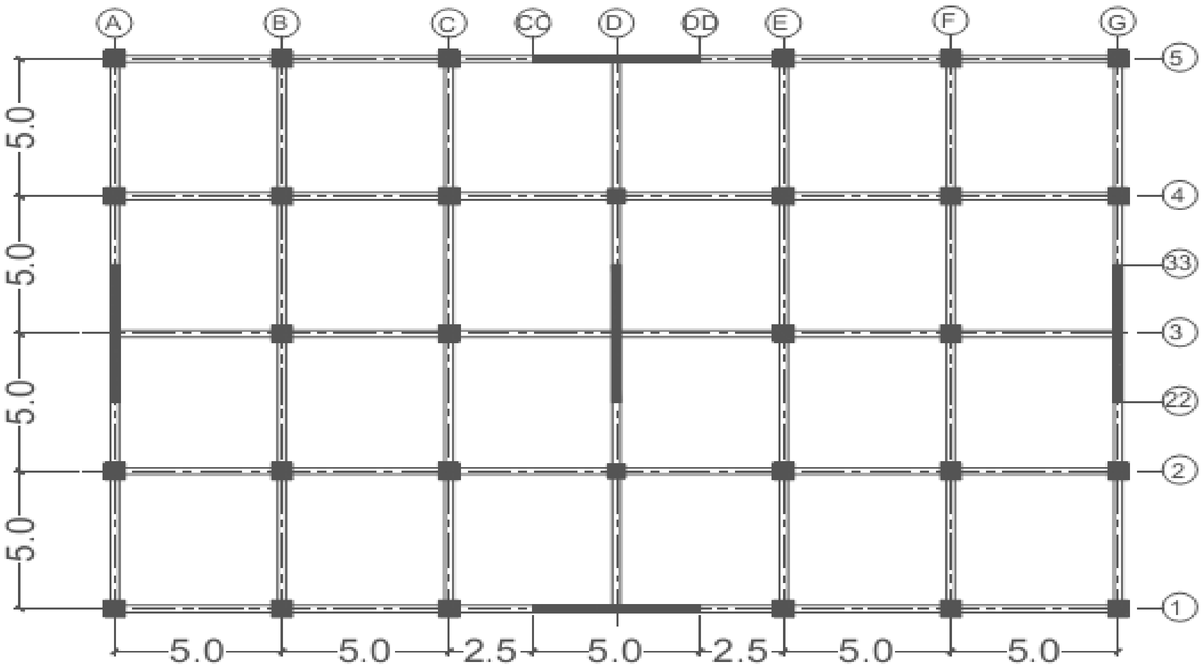


Figure 3.4: Typical structural layout of building Model 3 (G+15)

Table 3.3: Structural details and loading conditions of building model 3 (G+15)

Type of Structural System	Dual (frame with shear wall)
Beam Size	250 mm X 400 mm
Column Size	• 600 mm X 600 mm
Slab thickness	160 mm
Shear Wall thickness	200 mm
• Partition wall load	• 1.50 kN/m^2 is applied at all slabs except at roof terrace
Static gravity loads on slabs • Live load • Floor finishing load	 • 3 kN/m^2 • 1.73 kN/m^2 at all floor slabs.

4. The fourth building model is a twenty-storey (G+20) reinforced concrete building of dual frame-shear wall system supported on mat or end-bearing pile foundations as the case

may be. Details of the model and the loading conditions are given in Table 3.4, and its geometry is shown in Figure 3.5.

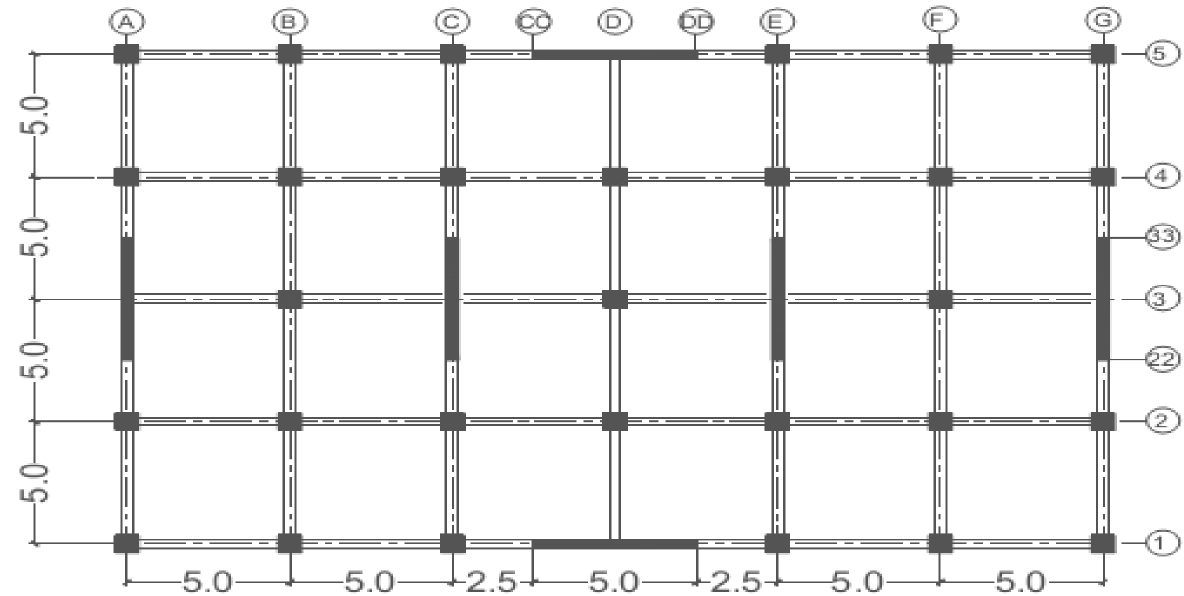


Figure 3.5: Typical structural layout of building Model 4 (G+20)

Table 3.4: Structural details and loading conditions of building model 4 (G+20)

Type of Structural System	Dual (frame with shear wall)
Beam Size	250 mm X 400 mm
Column Size	• 700 mm X 700
Slab thickness	160 mm
Shear Wall thickness	250 mm
• Partition wall load	• 1.50 kN/m^2 is applied at all slabs except at roof terrace
Static gravity loads on slabs • Live load • Floor finishing load	 • 3 kN/m^2 • 1.73 kN/m^2 at all floor slabs.

5. The fifth building model is a thirty-storey (G+30) reinforced concrete building of dual frame-shear wall system supported on mat or end-bearing pile foundations as the case

may be. Details of the model and the loading conditions are given in Table 3.5, and its geometry is shown in Figure 3.6.

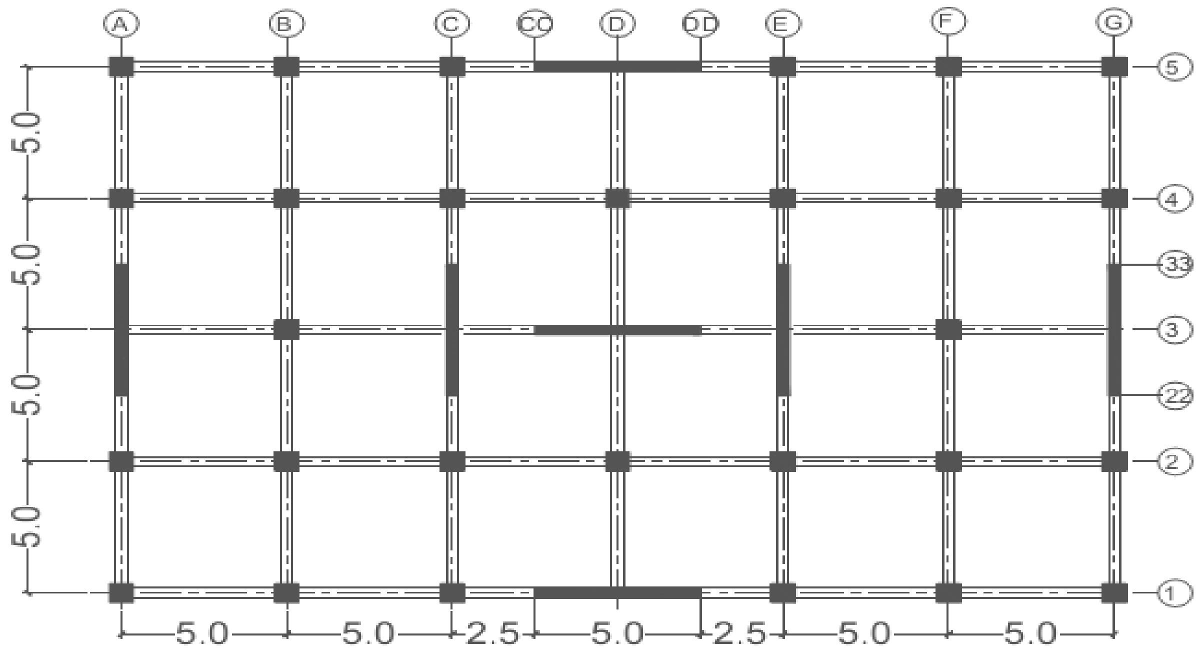


Figure 3.6: Typical structural layout of building Model 5 (G+30)

Table 3.5: Structural details and loading conditions of building model 5 (G+30)

Type of Structural System	Dual (frame with shear wall)
Beam Size	250 mm X 400 mm
Column Size	• 700 mm X 700
Slab thickness	160 mm
Shear Wall thickness	250 mm
• Partition wall load	• 1.50 kN/m^2 is applied at all slabs except at roof terrace
Static gravity loads on slabs • Live load • Floor finishing load	 • 3 kN/m^2 • 1.73 kN/m^2 at all floor slabs.

It is worth mentioning here that, in the dual frame-shear wall systems columns are laterally braced by concrete shear walls proportioned as per Section 10.10.1 of the ACI 318-08 (2008)

Code. It states that “it shall be permitted to consider compression members braced against side sway when bracing elements in a story have a total lateral stiffness of at least 12 times the gross lateral stiffness of the columns within the story”.

The lateral stiffness of a dual frame-shear wall system is the sum of the stiffnesses of columns and shear walls in the storey. For rigid beams with infinite flexural rigidity $E_b I_b = \infty$, where E_b and I_b are the elastic modulus and moment of inertia of the beams, the lateral story stiffness of columns, Q is given by the following equation (Chopra 1995).

$$Q = \sum_{columns} \frac{12 E_c I_c}{h^3} \tag{3.1}$$

Where: E_c = modulus of elasticity of the columns

I_c = the moment of inertia of columns in the direction of bending

h = storey height

And the lateral story stiffness for shear walls, J is given by (Yoshimura & Kikuchi)

$$J = \sum_{SW} \frac{12 E_{SW} I_{SW}}{h^3} \tag{3.2}$$

Where: E_{SW} = modulus of elasticity of the shear walls

I_{SW} = the moment of inertia of shear walls in the direction of bending

h = storey height

The story stiffnesses calculated for the building models is given Table 3.6 below.

Table 3.6: Story stiffness of the building models studied

Model	Total story stiffness (kN/m)	Ratio of story stiffness of shear walls to story stiffness of columns $\left(J/Q\right)$
Model 1 (G + 5)	1.86E+09	Framed building
Model 2 (G + 10)	4.43E+10	25.85
Model 3 (G + 15)	6.75E+10	19.31
Model 4 (G + 20)	9.08E+10	15.44
Model 5 (G + 30)	1.38E+11	13.10

3.2. Foundations provided for the building models

In Table 3.7, the foundations sizes provided for each model is presented. It is worth mentioning here that 2X2 square pile geometries are provided to support columns with different sizes at the internal and external columns depending on the support reactions obtained from analyses of fixed-base models. However, wall foundations are provided for shear walls. It is because of the reason that dynamic impedance functions are not readily available for rectangular pile geometries as the case may require for the shear walls. If a 5X5m square pile geometries were provided for the shear walls, there will be interactions among piles of adjacent pile groups which can't be easily handled.

Even if cases of only flexible-base building models supposed to be supported on pile foundations are presented here, analyses of similar flexible-base building models supposed to be supported on shallow foundations are also carried out. However, due to space limitation only results of flexible-base building models supposed to be supported on shallow foundations are discussed and compared with similar flexible-base building models supposed to be supported on pile foundations.

And, hence the following wall foundation sizes are provided for the shear walls,

- For model 2 (G+10): 1.0m X 5m
- For model 3 (G+15): 1.2m X 5m

- For model 4 (G+20): 1.5m X 5m
- For model 5 (G+30): 1.5m X 5m

Table 3.7: Foundation types and sizes provided to the building models

Model No.	Building model story	Pile foundations provided	
		Type	Size
1	G + 5	2 X 2 Floating Piles	• Ø 0.30m , L = 10 m under external columns • Ø 0.40m , L = 14 m under internal columns
2	G + 10	2 X 2 Floating Piles	• Ø 0.40m , L = 14 m under external columns • Ø 0.50m , L = 18 m under internal columns
3	G + 15	2 X 2 End-bearing Piles	• Ø 0.45m , L = 20 m under all columns
4	G + 20	2 X 2 End-bearing Piles	• Ø 0.60m , L = 23 m under all columns
5	G + 30	2 X 2 End-bearing Piles	• Ø 0.60m , L = 25 m under all columns

In this study the following assumptions are made:

- Circular, bored and cast in place concrete, fixed-head piles are used. The pile caps are assumed to be rigid and free-standing, i.e. there is no contact between the soil and the underside of the pile caps.
- Frictional resistance and bearing resistance are not be assumed to act simultaneously. So end-bearing is ignored in floating piles and the frictional resistance is ignored in the end bearing piles as per the recommendation of IBC (2006), section 1808.2.8.4.
- It is assumed that there is no weak stratum or compressible soil layer beneath the tips of the end bearing piles.
- Both the soil and the supported structure and its foundation respond linearly, and they don’t fail in rupture, excessive settlement, soil instability, liquefaction, or other

mechanism related to ultimate limit state of neither the soil nor the foundation and supported structure.

- The soil and piles are perfectly welded / bonded so that no gap is formed during seismic vibration.
- The piles are assumed to be vertical and linearly elastic.
- There is no interaction among piles of adjacent pile groups.
- Kinematic interaction effects are ignored.
- This study is limited to soils of homogeneous stiffness distributions with depth.

3.3 Provisions of SSPSI in Building Codes

To the author's knowledge, the effect of SSI is not yet addressed in EBCS 8 (1995) and many other internationally known building codes except EC 8 (2004). In some codes like NEHRP Provisions (2003), consideration of SSI is permitted but not required despite growing concerns about the effects of SSI in designs over the recent years. Part 5 of EC 8 (2004) states the effects of SSI and gives clear guidelines at what conditions and for what types of structures to consider the effects of dynamic SSI. However, it does not outline methods or procedures on how to consider the effects of SSI.

Part 5 of EC 8, 2004 (on page 27) states that: "The effects of dynamic SSI shall be taken into account in the case of:

- structures where P-δ (2^{nd} order) effects play a significant role;
- structures with massive or deep seated foundations;
- slender tall structures;
- structures supported on very soft soils, with average shear wave velocity less than 100m/s

Besides, Part 5 of EC 8, 2004 (on page 26) states that: "For pile foundations, bending moments developing due to kinematic interaction shall be computed only when all of the following conditions occur simultaneously:

- the ground profile is of type D, S_1 or S_2, and contains consecutive layers of sharply differing stiffness;

- the zone is of moderate or high seismicity, i.e. the product $a_g S_{factor}$ exceeds 0,10 g , (i.e. exceeds 0.98 m/s^2), and the supported structure is of importance class III or IV." Where: a_g is the design ground acceleration on type A ground, and S_{factor} is soil factor as defined in EN 1998-1:2004.

Note that implicitly for "normal" soil profiles and ordinary buildings kinematic interaction need not be computed.

3.4. Some key parameters of EC 8 (2004) response spectrum in relation to EBCS 8 (1995)

To use the response spectrum (RS) of EC 8 for analyses of the structures using SAP structural analysis software, it is necessary to relate the ground types classified in EC 8 with the EBCS 8 soil classification system. EC 8 is chosen due to its closest similarity with EBCS 8 code than the others. In doing so, the soil types defined in the two codes and some other important parameters defined in the RS function of EC 8 are discussed in the sequel.

3.4.1. Ground types

Section 3.1.2 of EC 8, Part 1, classifies ground types into seven, viz. A, B, C, D, E, S_1 and S_2. The classification is done based on the average shear wave velocity, $V_{s,30}$, Standard Penetration Test blow-count, N-value, and Undrained Shear Strength of soil (S_u), where $V_{s,30}$ = average value of propagation velocity of S waves in the upper 30 m of the soil profile at shear strain of 10^{-5} or less. EBCS 8, section 1.3.2 classifies soils in to three, viz. types A, B and C, based on similar parameters with EC 8. To reconcile the two different classification approaches the three Soil Types (ST) as classified in EBCS is related with the seven types of soils as classified in EC 8 soil as summarized below in Table 3.7 (Ameyu 2011).

Table 3.8: Relation of ground types of EC 8 (2004) with EBCS 8 (1995) (Ameyu 2011)

Code	Ground Types						
EC 8 (2004)	A	B	C	D	E	S_1	S_2
EBCS 8 (1995)	A		B	C			

3.4.2. Design ground acceleration, a_g

The seismic hazard map of Ethiopia, as given in EBCS 8, 1995, is developed based on a 100-years return period. The map divides the country in to five seismic zones viz. zones 0, 1, 2, 3, and 4, where zone 0 is considered seismic free. Accordingly, each of the Zones 1, 2, 3 and 4 is assigned a constant bedrock acceleration ratio, α_o, of 0.03, 0.05, 0.07 or 0.1 respectively, where, α_o is the peak ground acceleration (PGA) at bedrock level normalized with respect to the gravitational acceleration g. This current seismic hazard map of Ethiopia is not only incompatible with the 475-years return period accepted worldwide but also inadequate to ensure safety against loss of life and property (Worku 2011, Worku 2014b).

For Addis Ababa, different PGA values have been proposed in different reference materials. Some of the proposed PGA values for Addis Ababa are 0.05g (EBCS-8:1995), 0.13g – 0.50g (RADIUS project) and 0.3g (UBC - 1997 and IBC - 2000). Reconciling the different suggestions Kassegne (2006) proposes a PGA of 0.1g – 0.12g, as a temporary solution for Addis Ababa based on a design return period of 100 years. So, to be conservative a PGA of 0.12g is adopted for Addis Ababa which is the concern of this study, i.e. for Importance Class II, Ordinary buildings, for which the Importance factor is 1.0.

3.4.3 Damping correction factor, η

The damping correction factor η is used to account for the effect of SSI in modifying the basic response spectra provided for fixed-base conditions. In EN 1998-1 the spectral amplification (from peak ground acceleration to the acceleration at the constant acceleration branch) is fixed at 2.5 and is consistent with 5% viscous damping. It is however anticipated that the spectral shape may be adjusted for other damping values with the correction factor η given by EC 8 as:

$$\eta = \sqrt{10/(5+\xi)} \geq 0.55 \qquad (3.3)$$

Where: ξ is the effective system damping in percentile that accounts for both structural and foundation damping.

Studies reveal that using higher values of foundation damping (i.e. > 5%) results in reduction of base shear by up to the upper limit allowed 30% (Worku 2014a). For reinforced concrete structures, a damping ratio of 5% is used, so that the corresponding damping correction factor η is 1. In this work 5% damping is used as given in EBCS 8, 1995, for reinforced-concrete building structures.

3.4.4 Response Spectra Types

In EC-8, the effect of ground-motion intensity is treated by providing two types of Response Spectra (RS). Type-1 RS is provided for areas where earthquakes are expected to have a magnitude M_s greater than 5.5 (that is, high and moderate seismicity regions); and Type-2 RS for regions of low seismicity Ms ≤ 5.5. The site amplification factors provided for Type-2 RS are greater than those provided for Type-1. In this work both Types 1 and 2 response spectra are used as part of the sensitivity study. However, due to space limitation only results obtained using Type 1 spectrum are presented while discussing and comparing the results obtained using Type 2 spectrum.

3.4.5 Behavior factor, q

To avoid explicit inelastic structural analysis in design, the capacity of the structure to dissipate energy, through mainly ductile behavior of its elements and/or other mechanisms, is taken into account by performing an elastic analysis based on a response spectrum reduced with respect to the elastic one, henceforth called a "design spectrum". This reduction is accomplished by introducing the behavior factor, q.

The behavior factor, q is an approximation of the ratio of the seismic forces that the structure would experience if its response was completely elastic with 5% viscous damping, to the seismic forces that may be used in the design, with a conventional elastic analysis model, still ensuring a satisfactory response of the structure. The values of the behavior factor q, which also account for the influence of the viscous damping being different from 5%, are given for various materials and structural systems according to the relevant ductility classes in the various Parts of EN 1998.

The value of the behavior factor q may be different in different horizontal directions of the structure, although the ductility classification shall be the same in all directions (EC 8 2004).

Hence EN 1998-1, besides the elastic response spectra discussed above, presents the so called design spectra for elastic analysis. In most of the period range, the ratio between the elastic spectrum and the corresponding design spectrum is simply the value of the behavior factor q as indicated above.

As per Section 5.2.2., of EC-8, the behavior factor, q is defined as

$$q = q_o K_w \geq 1.5 \tag{3.4}$$

Where,

q_o is the basic value of the behavior factor, dependent on the type of the structural system and on its regularity in elevation;

K_w is the factor reflecting the prevailing failure mode in structural systems with walls.

So, assuming the structures are of medium ductility the behavior factor calculated as per EC 8 becomes

$q = q_o K_w$ = 3.0*1.3*1 = 3.9 > 1.5, for all models in both x and y directions.

3.4.6 Seismic weight

The seismic weight used in calculation of the base shear includes dead loads of structural and non structural elements of the structure, and 25% of the live load (EC 8: 2004; Naeim 2008).

3.4.7 Modal combination

Using the RS method for multi degree of freedom (MDOF) systems, the maximum modal response is obtained for each mode of a set of modes, which are used to represent the seismic response of the structure. The following options are available in SAP software for modal combinations: Complete Quadratic Combination (CQC) method, Square Root of the Sum of the Squares (SRSS) method, Absolute (ABS) method, General Modal Combination (GMC) method.

Of these methods, the CQC method is recommended for 3D analyses because it takes into account the statistical coupling between closely spaced modes caused by modal damping (Wilson 2002; Naeim 2008). So the CQC method is used in this study for modal combination.

3.4.8 Directional and orthogonal effects

The principal direction of an earthquake motion is not known and for most geographical locations it cannot be estimated. Therefore, the only rational earthquake design criterion is that the structure must resist an earthquake of a given magnitude from any possible direction and in addition a probability exists that motions normal to the principal direction will occur simultaneously. Thus, horizontal seismic actions can be described by two orthogonal components assumed as being independent and represented by similar response spectra (EC 8 2004; Wilson 2003).

For three-dimensional response spectra analyses, it has been shown that the commonly used percentage combination rules i.e. design of elements for 100 percent of the prescribed seismic forces in one direction plus 30 or 40 percent of the prescribed forces applied in the perpendicular direction are empirical and can underestimate the design forces in certain members and produce a member design that is relatively weak in one direction. However, an SRSS combination of two 100 percent spectra analyses with respect to any user-defined orthogonal axes will produce design forces that are not a function of the reference system and the resulting structural design has equal resistance to seismic motions from all directions (Wilson 2003).

4. SENSITIVITY STUDY

4.1 General

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

- Soil types – Types C and D are used in the cases of floating piles and shallow foundations; and Soil Type A is used in the cases of end bearing piles,
- Foundation types :
  - Pile foundations
    - floating piles are used for Models 1 and 2,
    - end bearing piles are used for models 3, 4 and 5.
  - Shallow foundations
    - isolated footings are used for Model 1,
    - mat foundations are used for models 3, 4 and 5.
- Response spectra types – Type 1 and Type 2 EC 8, 2004 response spectra are used,

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

- Fundamental natural periods of vibrations,
- Base reactions (i.e. base shears and overturning moments),
- Bending moment variations along lengths of beams,
- Shear force variations along lengths of beams,
- Axial forces in columns,
- Bending moment variations in shear walls.

The following material properties for characterizing foundation soils and concrete are used.

- Poisson's ratio of soils for shallow foundations: 0.333,

- Poisson's ratio of soils for pile foundations: 0.4‡,
- Mass density of soils: 1600 kg/m^3,
- Young's modulus of elasticity of concrete: 2.5E+10 Pa,
- Typical Shear wave velocity for type A soil: 800 m/s,
- Typical Shear wave velocity for type C soil: 250 m/s,
- Typical Shear wave velocity for type D soil: 100 m/s.

‡ Since rotational impedance functions for pile foundations given by Kaynia and Kausel (1982) are based on Poisson's ratio of 0.4.

4.2 Discussion of results

Due to space limitation only the results of the (G+5) and (G+10) models supposed to be supported on pile foundations and subjected to Type 1 RS are presented. For the other models and analyses cases only the results are summarized, compared and discussed.

4.2.1 Dynamic stiffness and damping values

The dynamic stiffnesses and associated dampings are calculated for each case of analyses employing equation 2.11 as discussed in Chapter 2. The results for models 1 and 2 are presented in Tables 4.1 and 4.2 respectively. In these tables, $\bar{K}_x$ and $\bar{K}_y$ are respectively the lateral horizontal stiffnesses in x and y directions, $\bar{K}_z$ is the vertical stiffness, $\bar{K}_{rx}$ and $\bar{K}_{ry}$ are respectively the rocking stiffnesses about x and y axes, $\bar{K}_{xry}$ and $\bar{K}_{yrx}$ are respectively the cross swaying-rocking stiffnesses and $\bar{K}_t$ is the torsional stiffness about z axis. Similarly, C_x, C_y are respectively the lateral horizontal dampings in x and y directions, C_z is the vertical damping, C_{rx} and C_{ry} are respectively the rocking dampings about x and y axes, C_{xry} and C_{yrx} are respectively the cross swaying-rocking dampings and C_t is the torsional damping about z axis.

Table 4.1: Dynamic stiffness and damping values for Model 1 (G+5)

Stiffness and Damping values	Foundation Type - Floating Piles			
	Soil type			
	Type C		Type D	
	Piles under external columns	Piles under internal columns	Piles under external columns	Piles under internal columns
$\bar{K}_x$ (kN/m)	3.4E+05	4.6E+05	8.1E+04	1.1E+05
$\bar{K}_y$ (kN/m)	3.4E+05	4.6E+05	8.1E+04	1.1E+05
$\bar{K}_z$ (kN/m)	2.8E+05	3.6E+05	1.6E+05	2.3E+05
$\bar{K}_{rx}$ (kNm/rad)	4.6E+05	1.1E+06	2.8E+05	6.7E+05
$\bar{K}_{ry}$ (kNm/rad)	4.6E+05	1.1E+06	2.8E+05	6.7E+05
$\bar{K}_{xry}$ (kNm/rad)	2.1E+05	3.7E+05	8.4E+04	1.5E+05
$\bar{K}_{yrx}$ (kNm/rad)	2.1E+05	3.7E+05	8.4E+04	1.5E+05
$\bar{K}_t$ (kNm/rad)	2.4E+05	5.8E+05	5.7E+04	1.4E+05
C_x (kN/m)	2.4E+05	3.2E+05	5.7E+04	1.1E+05
C_y (kN/m)	2.4E+05	3.2E+05	5.7E+04	1.1E+05
C_z (kN/m)	2.5E+05	3.2E+05	1.5E+05	2.0E+05
C_{rx} (kNm/rad)	4.5E+03	1.1E+04	2.8E+03	6.7E+03
C_{ry} (kNm/rad)	4.5E+03	1.1E+04	2.8E+03	6.7E+03
C_{xry} (kNs/rad)	1.4E+04	2.4E+04	5.4E+03	9.7E+03
C_{yrx} (kNs/rad)	1.4E+04	2.4E+04	5.4E+03	9.7E+03
C_t (kNm/rad)	2.4E+05	5.8E+05	5.7E+04	1.4E+05

Table 4.2: Dynamic stiffness and damping values for Model 2 (G+10)

Stiffness and Damping values	Soil type					
	Type C			Type D		
	Wall[‡] footing for shear walls	Pile foundation		Wall[‡] footing for shear walls	Pile foundation	
		Piles under external columns	Piles under internal columns		Piles under external columns	Piles under internal columns
$\bar{K}_x$ (kN/m)	2.0E+06	4.6E+05	5.7E+05	2.0E+06	1.1E+05	1.3E+05
$\bar{K}_y$ (kN/m)	2.0E+06	4.6E+05	5.7E+05	2.0E+06	1.1E+05	1.3E+05
$\bar{K}_z$ (kN/m)	3.7E+06	3.6E+05	4.4E+05	3.7E+06	2.3E+05	2.9E+05
$\bar{K}_{rx}$ (kNm/rad)	1.7E+07	1.1E+06	2.0E+06	1.7E+07	6.7E+05	1.3E+06
$\bar{K}_{ry}$ (kNm/rad)	2.7E+07	1.1E+06	2.0E+06	2.6E+07	6.7E+05	1.3E+06
$\bar{K}_{xry}$ (kNm/rad)	3.3E+06	3.7E+05	5.8E+05	3.3E+06	1.5E+05	2.3E+05
$\bar{K}_{yrx}$ (kNm/rad)	3.3E+06	3.7E+05	5.8E+05	3.3E+06	1.5E+05	2.3E+05
$\bar{K}_t$ (kNm/rad)	3.7E+07	5.8E+05	1.1E+06	3.7E+07	1.4E+05	2.6E+05
C_x (kN/m)	1.3E+07	3.2E+05	4.1E+05	5.1E+06	7.6E+04	9.5E+04
C_y (kN/m)	1.8E+07	3.2E+05	4.1E+05	7.2E+06	7.6E+04	9.5E+04
C_z (kN/m)	1.5E+07	3.2E+05	4.0E+05	6.0E+06	2.0E+05	2.6E+05
C_{rx} (kNm/rad)	1.2E+07	1.1E+04	2.1E+04	6.0E+06	6.7E+03	1.3E+04
C_{ry} (kNm/rad)	1.8E+07	1.1E+04	2.1E+04	9.2E+06	6.7E+03	1.3E+04
C_{xry} (kNs/rad)	8.7E+06	2.4E+04	3.8E+04	3.4E+06	9.7E+03	1.5E+04
C_{yrx} (kNs/rad)	1.2E+07	2.4E+04	3.8E+04	4.8E+06	9.7E+03	1.5E+04
C_t (kNm/rad)	4.5E+06	5.8E+05	1.1E+06	2.9E+06	1.4E+05	2.6E+05

‡ - refer section 3.2 for the reason why wall footings are proposed as foundations of shear walls.

From Tables 4.1 and 4.2, it can be inferred that both dynamic stiffness and damping values increase as foundation sizes increase for similar soil types. Besides, it is observed that both dynamic stiffness and damping increase as shear wave velocity of soil increases i.e. higher for soil type C than soil type D. The above conclusion holds true also for end bearing piles and shallow foundation types. The results also reveal that for all the building models studied both dynamic stiffness and damping values obtained for shallow foundations are higher than for floating and end bearing pile foundations for all the building models.

4.2.2 Comparison of natural periods

The natural periods of vibrations of the models are obtained from SAP structural analyses program. The results obtained for the first two modes of vibrations for both the fixed-base and flexible-base models are presented in Table 4.3. The natural periods of vibrations obtained for the building models are somehow smaller than values expected for common reinforced concrete structures except for the five-story model. It is likely due to the higher stiffnesses of the shear walls provided in orthogonal directions. The natural periods of vibrations for flexible-base models are found to be higher than their fixed-base counterpart models; and the period elongation becomes higher for the higher modes of vibrations. The following observations are additionally made with regard to the period elongation.

4.2.2.1. Models supported on floating pile foundations

The fundamental natural periods of the flexible-base models are observed to be elongated when compared with the fixed-base models. The period elongations are more pronounced for models supported on pile foundations than similar models supported on shallow foundations, for both soil Types-C and D. The period elongations depend on the type of the supporting soil, as the natural periods of vibrations of models get more elongated when supported on Type D soils than Type C soils as expected.

4.2.2.2. Models supported on end-bearing pile foundations

The fundamental natural periods of vibrations of the flexible-base models supported on end-bearing piles are also higher than the fixed-base counterpart models. The maximum observed elongation is for the fifteen-story model, of which the natural period of vibration for the flexible-base model supported on end-bearing pile increases 1.28 times its fixed-base model. Similarly, the natural periods of the twenty and thirty-story flexible-base models supported on end-bearing piles increase respectively 1.18 and 1.22 times their counterpart fixed-base models.

In addition, the following remarks can be made regarding the fundamental periods of vibrations of the models:

- The fundamental natural periods of pile supported models are more elongated than models supported on shallow foundations. The natural periods of the models studied are more than 0.5s which is in the descending branch of the response spectrum, so that period elongation due to soil deformability is expected to result in a decrease of the seismic demand.

- Elongation of the fundamental natural periods of vibrations of models supported on floating pile foundations are observed to be higher than models supported on end-bearing piles when soil flexibility is considered.

- For flexible-base models supported on shallow foundations, soil type has no influence on the degree by which the natural periods of the flexible-base models get elongated. However, for flexible-base models supported on floating pile foundations Type-D soil results in more elongation of the natural periods than Type-C soil.

- Gashaye (2005) and Solomon (2007) have also obtained similar results on effects of soil flexibility on the fundamental natural periods of vibrations of models supported on shallow foundations.

Table 4.3: Natural periods of vibrations for models 1 (G+5) and 2 (G+10)

	Mode	Analyses cases			
		Fixed-base model on ST C	Fixed-base model on ST D	Flexible-base model on ST C	Flexible-base model on ST D
		a) Model 1 (G+5)			
Period t (sec)	1	0.5435	0.5435	0.6097	0.7047
	2	0.5321	0.5321	0.5925	0.6960
		b) Model 2 (G+10)			
	1	0.5105	0.5105	0.7734	0.8585
	2	0.5064	0.5064	0.7468	0.8013

4.2.3 Effect of SSPSI on base reactions

The base reactions of the model structures are obtained from structural analyses results of SAP software and comparisons are made between the analyses results obtained for fixed-base and flexible-base models. The base shears and overturning moments are observed to get reduced when soil flexibility is considered. The degree by which the base reactions get reduced depends on the foundation type, soil type, type of response spectra, and also on the height of the model building structures.

4.2.3.1. Models supported on floating pile foundations

As shown in Tables 4.4 and 4.5, the base shears (FX, FY and FZ) and overturning moments (MX, MY and MZ), about X, Y and Z axes respectively are observed to get reduced when soil flexibility is considered. For both the five and ten-story models, soil type is observed to influence the reduction in base reactions, and it is observed to be more reduced when the models are supported in soil type D than soil type C.

Higher reductions of base reactions are observed for the ten-story model than the five-story model in both soils types C and D. This is attributed to the higher degree of elongation of the natural periods of vibrations of the ten-story model due to the effects of SSPSI.

4.2.3.2. Models supported on end-bearing pile foundations

For the fifteen, twenty and thirty storey models also, both base shears and overturning moments are observed to get reduced when SSPSI is considered. Besides, soil Type C results in greater reductions of base shears and overturning moment than soil Type D similar to models supported on floating pile foundations.

In addition, flexible-base models supported on end-bearing piles show greater reductions in both base shear and overturning moment than models supported on floating pile foundations contrary to the higher degree of period elongation for the later models. This is most likely due to the higher damping in the rocking vibrations for the flexible-base models supported on end-bearing piles than the flexible-base models supported on floating pile foundations.

The following observations are generally made in the comparison of base shears and overturning moments between the fixed-base and flexible-base models.

It is also observed that:

- Higher reduction of base reactions are observed for all models assumed to be supported on pile foundations than their counterpart models supported on shallow foundations in both soil types C and D.
- Other parameters kept constant, higher reductions of base reactions are obtained for Type-2 response spectrum than Type-1.
- Higher reductions of base reactions are obtained as the building height increase owing to the accompanying period lengthening.

Table 4.4: Base reactions for model 1 (G+5)

Analyses cases	Global FX	Global FY	Global FZ	Global MX	Global MY	Global MZ
	kN	kN	kN	kN-m	kN-m	kN-m
Fixed-base model on ST C	2.7E+03	2.7E+03	5.3E+04	5.6E+05	7.6E+05	4.9E+04
Fixed-base model on ST D	3.2E+03	3.2E+03	5.3E+04	5.7E+05	7.5E+05	5.7E+04
Flexible-base model when supported on pile foundation on ST C	1.6E+03	1.6E+03	5.3E+04	5.5E+05	7.7E+05	2.9E+04
Flexible-base model when supported on pile foundation on ST D	2.0E+03	1.9E+03	5.3E+04	5.5E+05	7.7E+05	3.5E+04

Table 4.5: Base reactions for model 2 (G+10)

Analyses cases	Global FX	Global FY	Global FZ	Global MX	Global MY	Global MZ
	kN	kN	kN	kN-m	kN-m	kN-m
Fixed-base model on ST C	4.7E+03	4.7E+03	9.7E+04	1.1E+06	1.3E+06	8.4E+04
Fixed-base model on ST D	5.5E+03	5.5E+03	9.7E+04	1.1E+06	1.3E+06	9.9E+04
Flexible-base model when supported on pile foundation on ST C	2.4E+03	2.3E+03	9.7E+04	1.0E+06	1.4E+06	4.3E+04
Flexible-base model when supported on pile foundation on ST D	3.5E+03	3.1E+03	9.7E+04	1.0E+06	1.4E+06	5.9E+04

4.2.4. Effect of SSPSI on bending moments in beams

The effect of SSPSI on bending moment variations along beams is assessed through systematic comparisons of results obtained for the fixed-base and flexible-base building models on selected beams. In reference to Figure 4.1, the comparisons are made for end bay beams of "frame on axis D" for the 1st and 2nd models, and interior bay beams of "frame C" for the 3rd model and "frame on axis B" for the remaining 4th and 5th models. These "axes and bays" are selected as considerable differences in bending moments (BM) and shear forces (SF) between the fixed-base and flexible-base models are observed along these frames. The differences in BM and SF in beams become increasing while going down the storey levels from top to bottom. So, the comparisons are made for selected beams of the lowest storey.

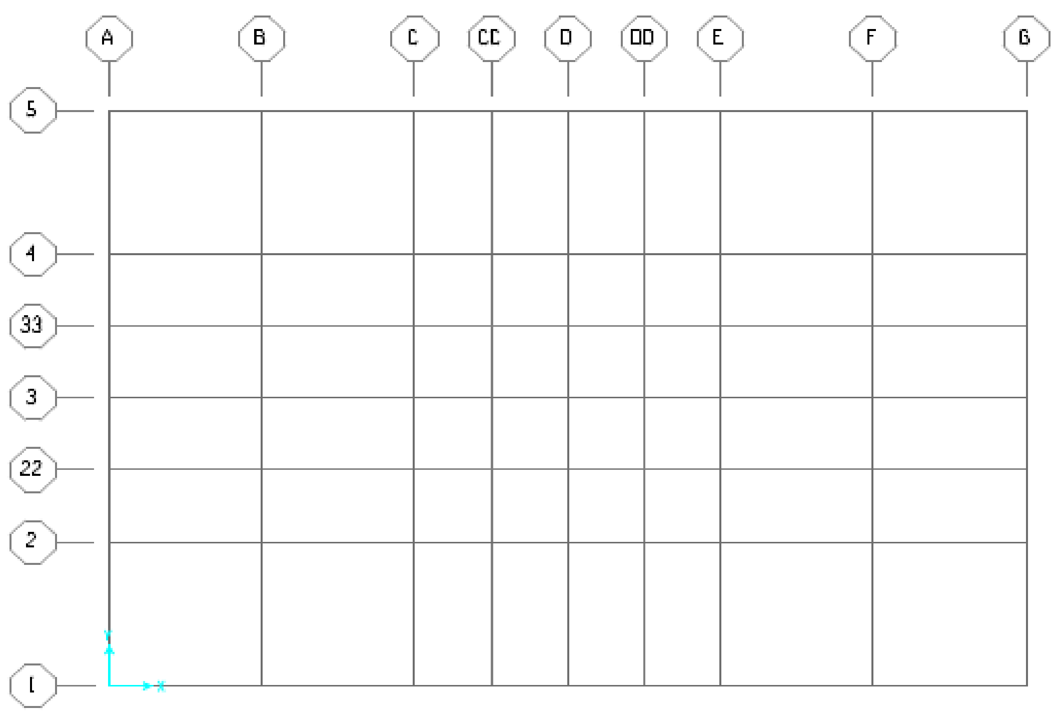


Figure 4.1: Frame layout

4.2.4.1. Models supported on floating pile foundations

i) Model 1 (G+5)

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

As shown in Figure 4.2 BMs along beams of flexible-base models supported on pile foundations are observed to vary from that obtained for the fixed-base models. The BMs along beams are observed to be higher when soil flexibility is considered. The differences in BMs between the flexible-base and fixed-base models are observed to be higher at the supports; and are observed to be lower towards the span diminishing almost to zero at the mid span. BMs along beams of flexible-base models supported on pile foundations are observed to be higher than similar flexible-base models supported on shallow foundations.

For flexible-base models supported on both pile and shallow foundations, the BMs along beams are observed to be higher in soil type D than soil type C.

ii) Model 2 (G+10)

The BMDs of the flexible-base and fixed-base models of the (G+10) model obtained from SAP for different cases are plotted together as shown in Figure 4.3 as done above for model 1. In Figure 4.3 BMs along beams of flexible-base models are observed to be lower than that obtained for the fixed-base models. The differences in BMs between the flexible-base and fixed-base models are observed to be higher at the supports and lower towards the span diminishing almost to zero at the mid span.

For flexible-base models the BMs along beams are higher for soil type D than soil type C. This is due to the fact that Type D soil is relatively softer than Type C soil in which the effects of SSI are more pronounced.

The differences in BMs between the fixed-base models and flexible-base models on shallow foundations are observed to be higher towards one support of the beam and lower towards the other support than similar models supported on pile foundations.

From the results obtained for models 1 and 2, it can be observed that consideration of soil flexibility has dual advantageous:

- It can avoid unsafe designs of beams as higher BMs along beams are resulted when it is considered as shown for model 1,
- It may reward economic advantages as lower BMs along beams are resulted when it is considered as shown for model 2.
- Gashaye (2005) has also obtained similar results on effects of soil flexibility on the BM variations along beams of models supported on shallow foundations.

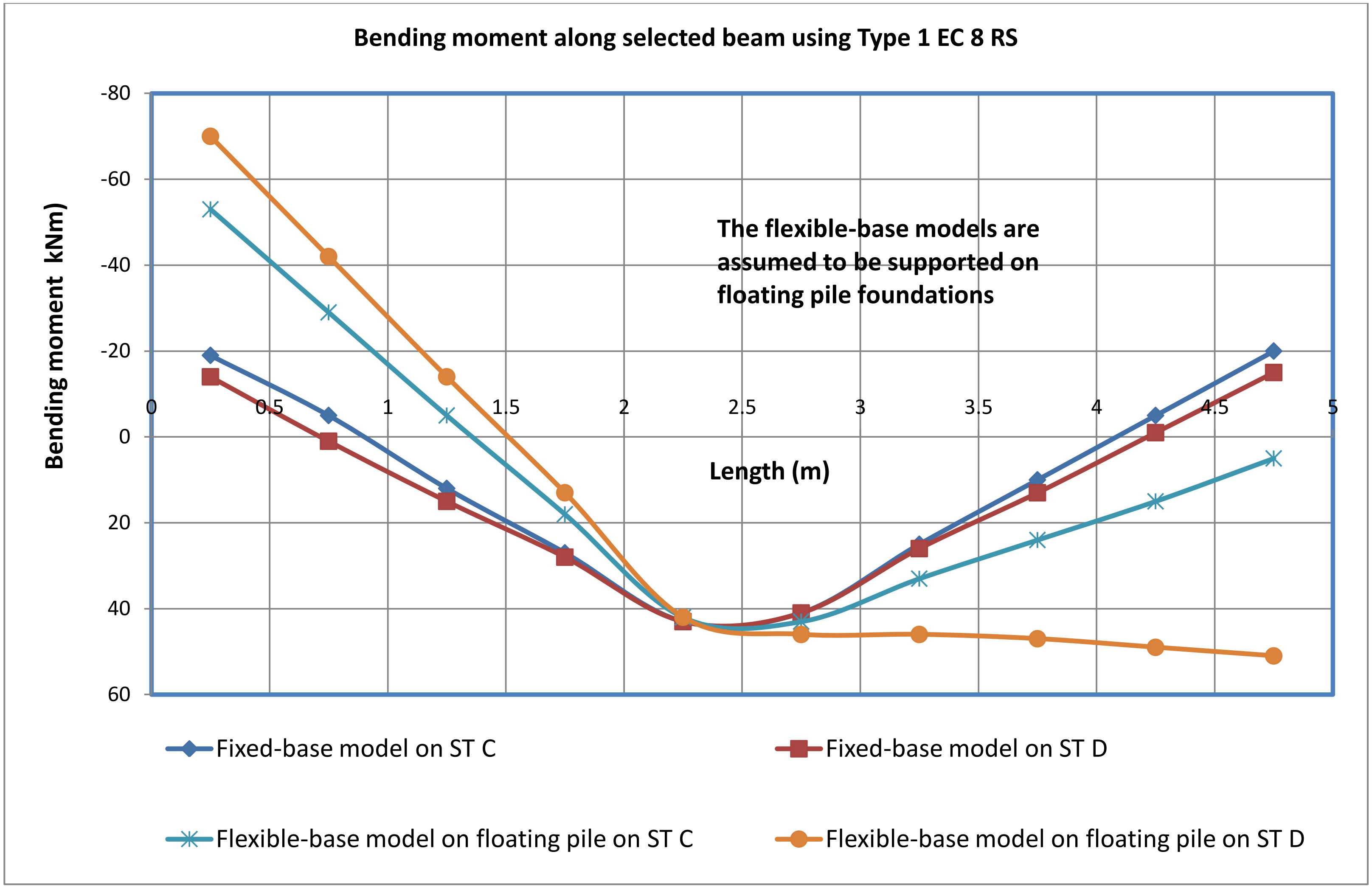


Figure 4.2: Bending moment diagram along selected beam of model 1 (G+5)

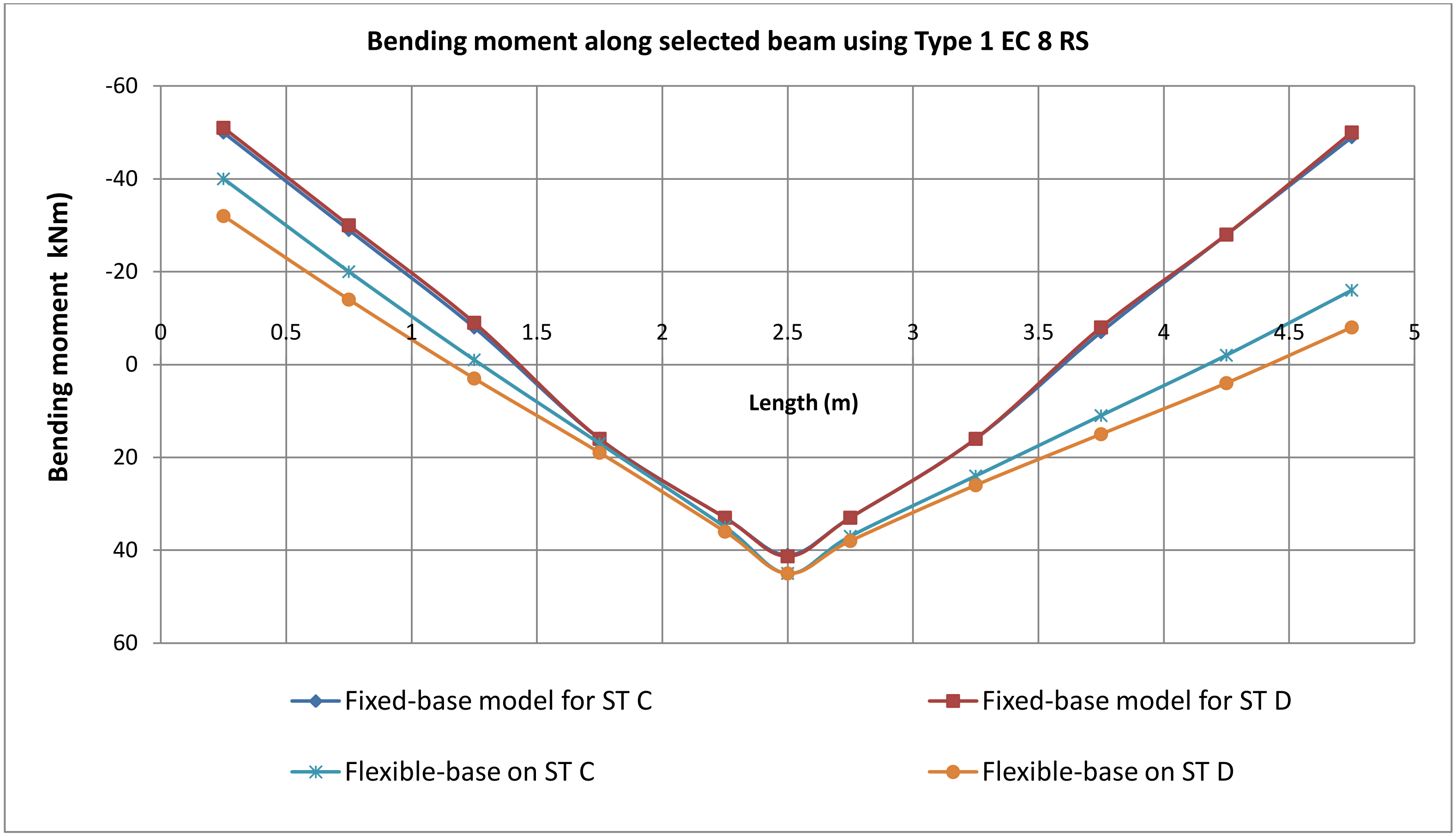


Figure 4.3: Bending moment diagram along selected beam of model 2 (G+10)

4.2.4.2. Models supported on end-bearing pile foundations

The BMs along selected beams of flexible-base models supported on end-bearing pile foundations are also observed to vary from that obtained for their counterpart fixed-base models. The BM variations along beams of flexible-base and fixed-base models for the 3rd (G+15), 4th (G+20) and 5th (G+30) models are plotted in Figures 4.4 – 4.6.

From these Figures, it is observed that for models supported on end-bearing piles, support moments are observed to be higher at one support and lower at the other support when soil flexibility is considered. However, span moments are observed to be almost identical for both fixed-base and flexible-base models.

In addition it is observed that:

- For flexible-base models supported on shallow foundations, the support moments are observed to be higher at both supports; and higher differences in span moments are observed for soil type D than soil type C.
- The BMs along beams obtained using Type-2 RS are lesser as compared with those obtained using Type-1 RS. This might be due to the higher magnitude of Type-1 RS for which higher differences of BMs between the fixed-base and flexible-base models along beams are observed.

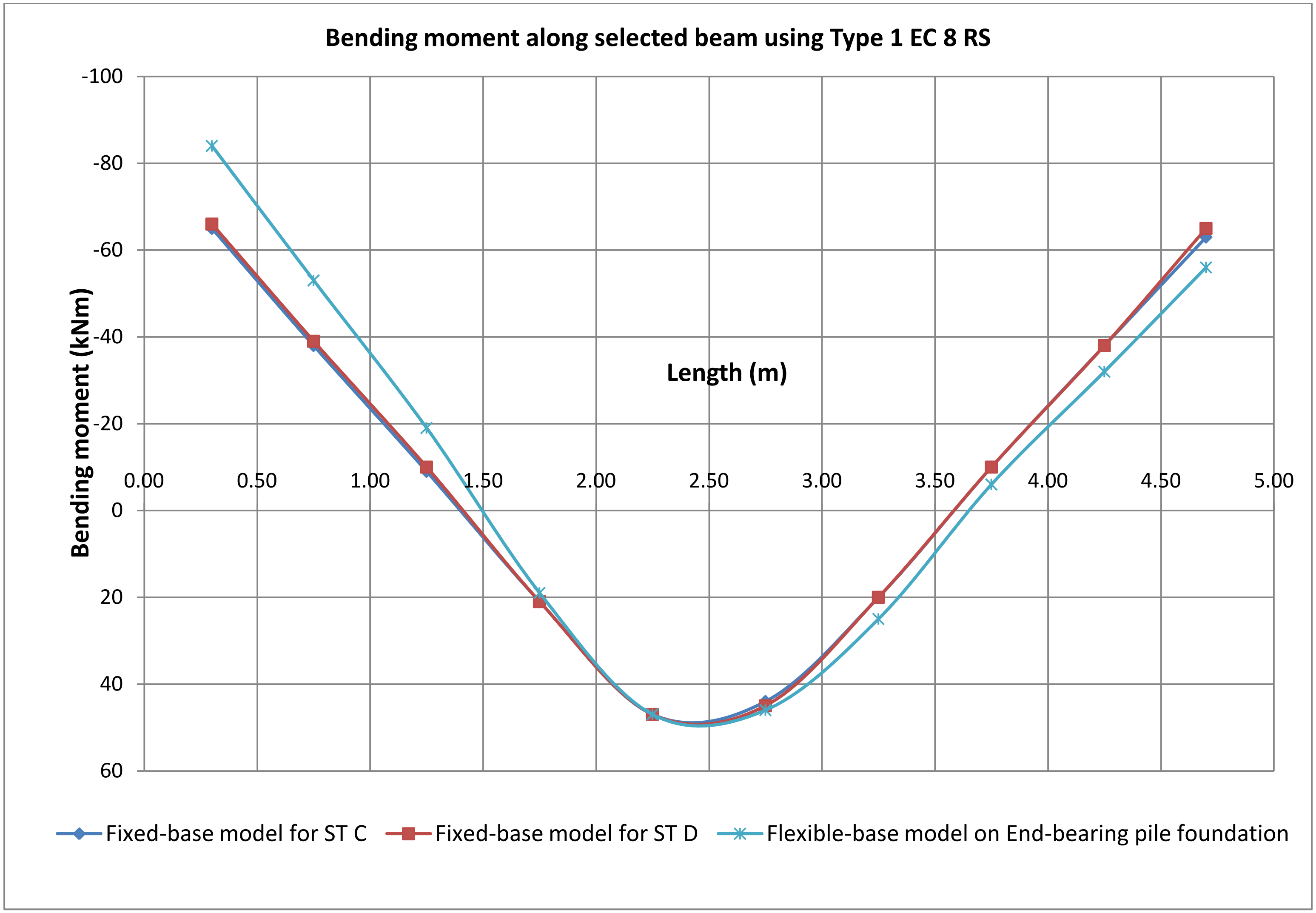


Figure 4.4: Bending moment diagram along selected beam of Model 3 (G+15)

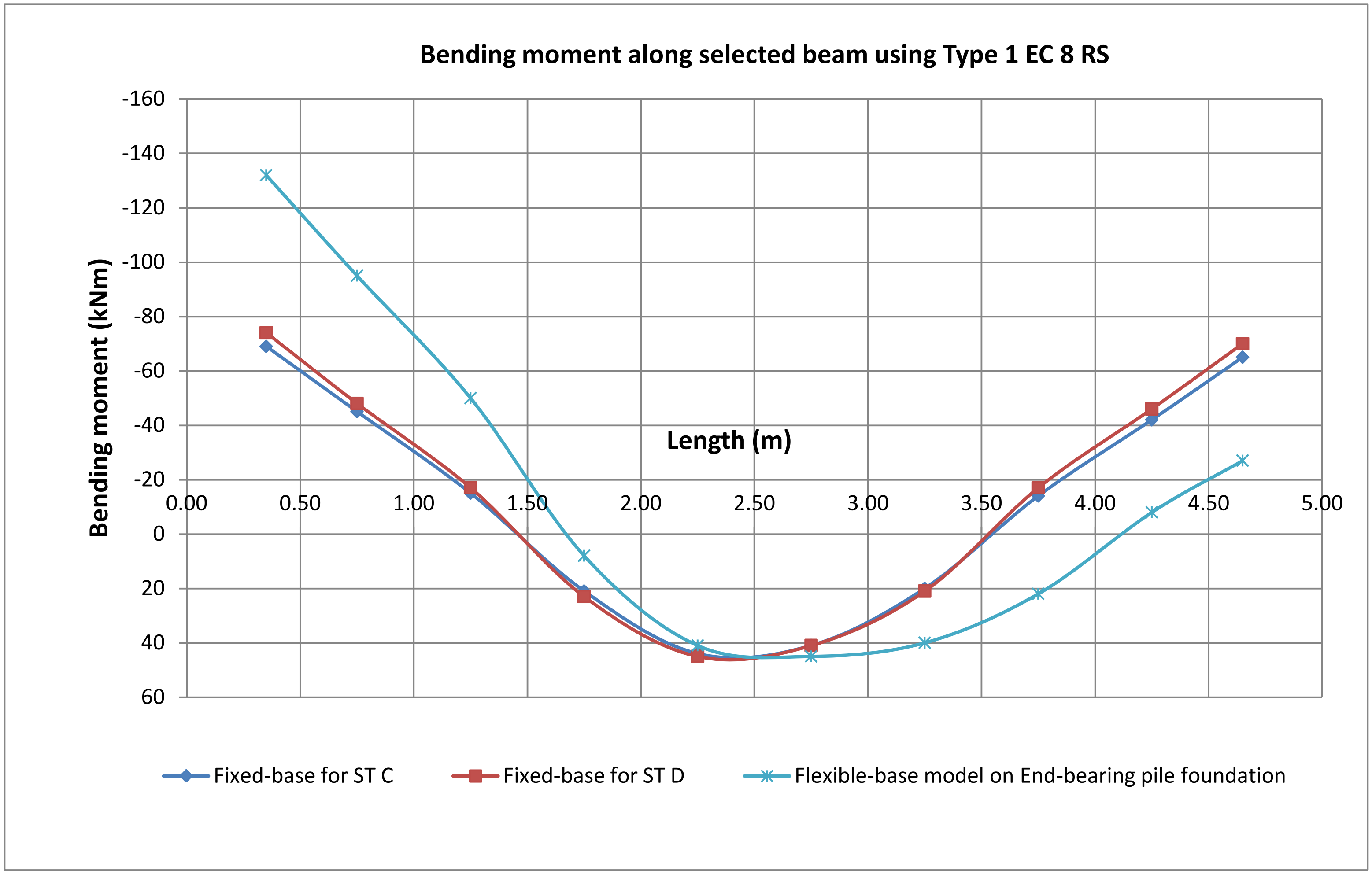


Figure 4.5: Bending moment diagram along selected beam of Model 4 (G+20)

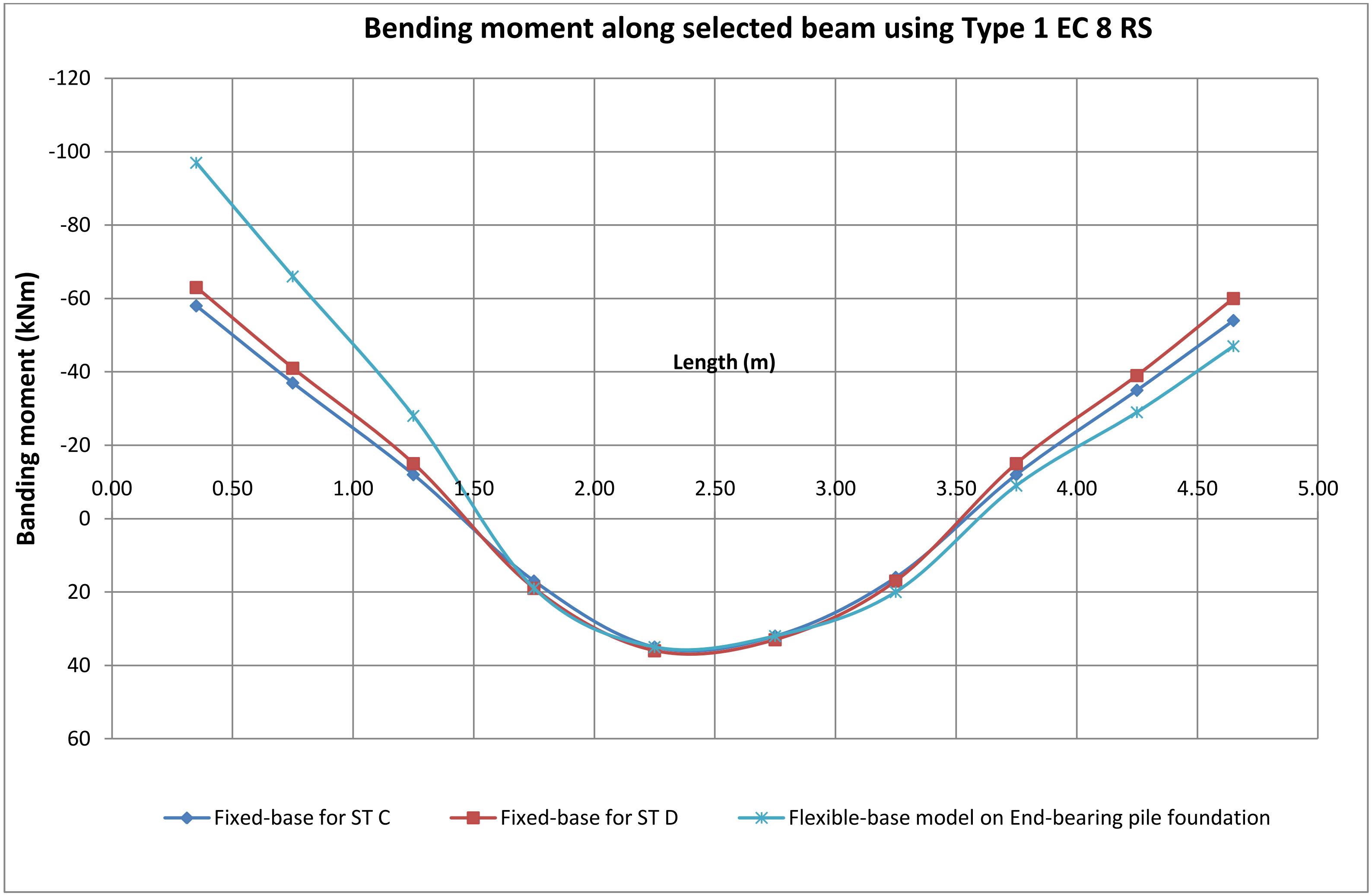


Figure 4.6: Bending moment diagram along selected beam of Model 5 (G+30)

4.2.5. Effect of SSPSI on Shear Forces (SF) in beams

The effect of SSPSI on shear force (SF) variations along length of beams is assessed through comparisons of Shear Force Diagrams (SFD) obtained for the fixed-base and flexible-base models. Similar beam elements selected for plotting BMDs are used for plotting SFDs too for similar reasons.

4.2.5.1. Models supported on floating pile foundations

The SFDs of fixed-base and flexible-base models of Model 1 (G+5) and Model 2 (G+10) are presented in Figures 4.7 and 4.8 respectively. As shown in the Figures SF along beams of flexible-base models vary from that obtained for the fixed-base models. SFs in beams become higher along one half of the beam and lower at the other half when soil flexibility is considered. Higher differences in SFs along beams are observed for Type D soil than Type C soil in model 1, however the reverse is observed in model 2. The differences are higher for flexible-base models supported on floating piles than similar models supported on shallow foundations.

Gashaye (2005) has also obtained similar results on effects of soil flexibility on the SF variations along beams of models supported on shallow foundations.

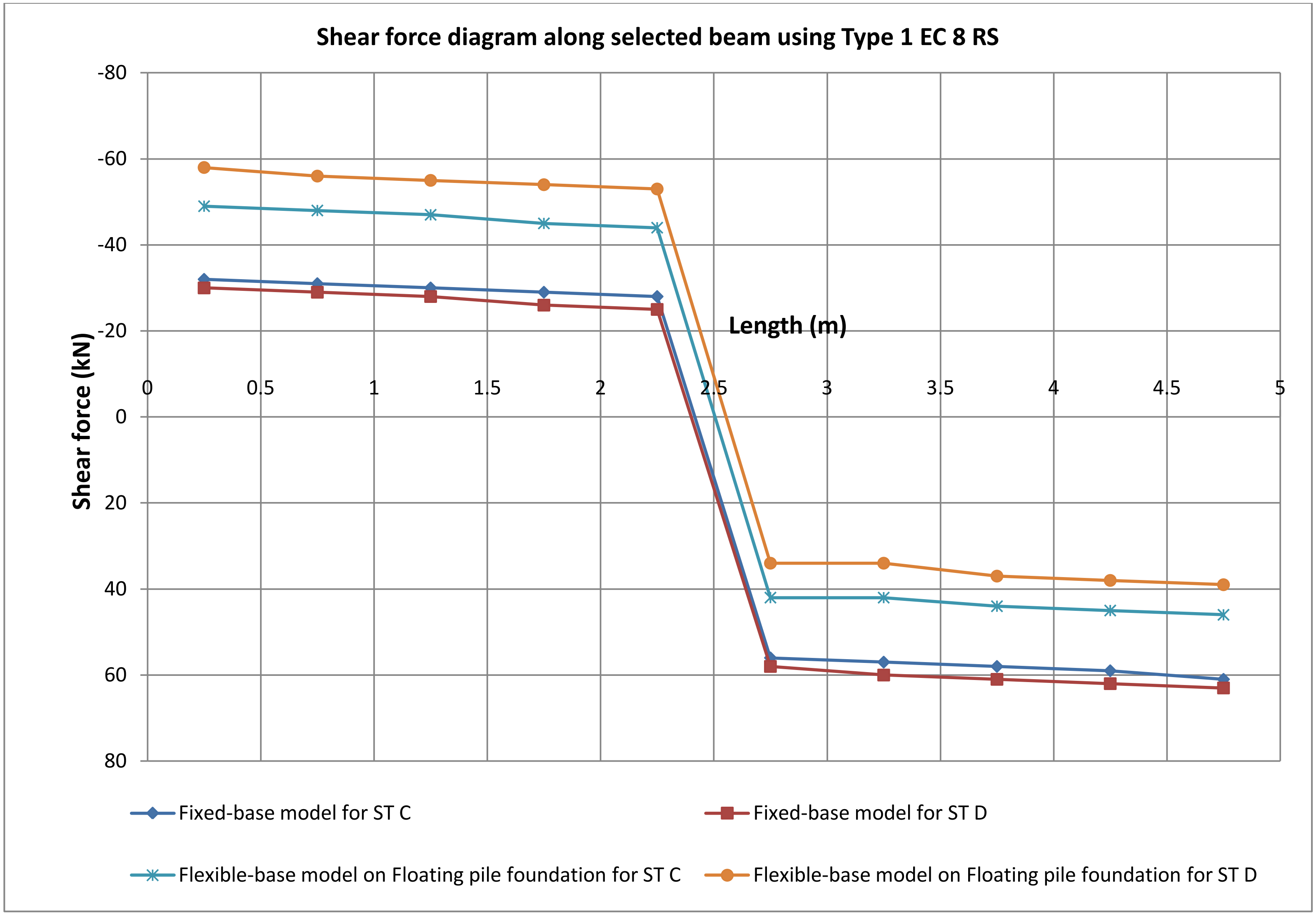


Figure 4.7: Shear force diagram along selected beam of Model 1 (G+5)

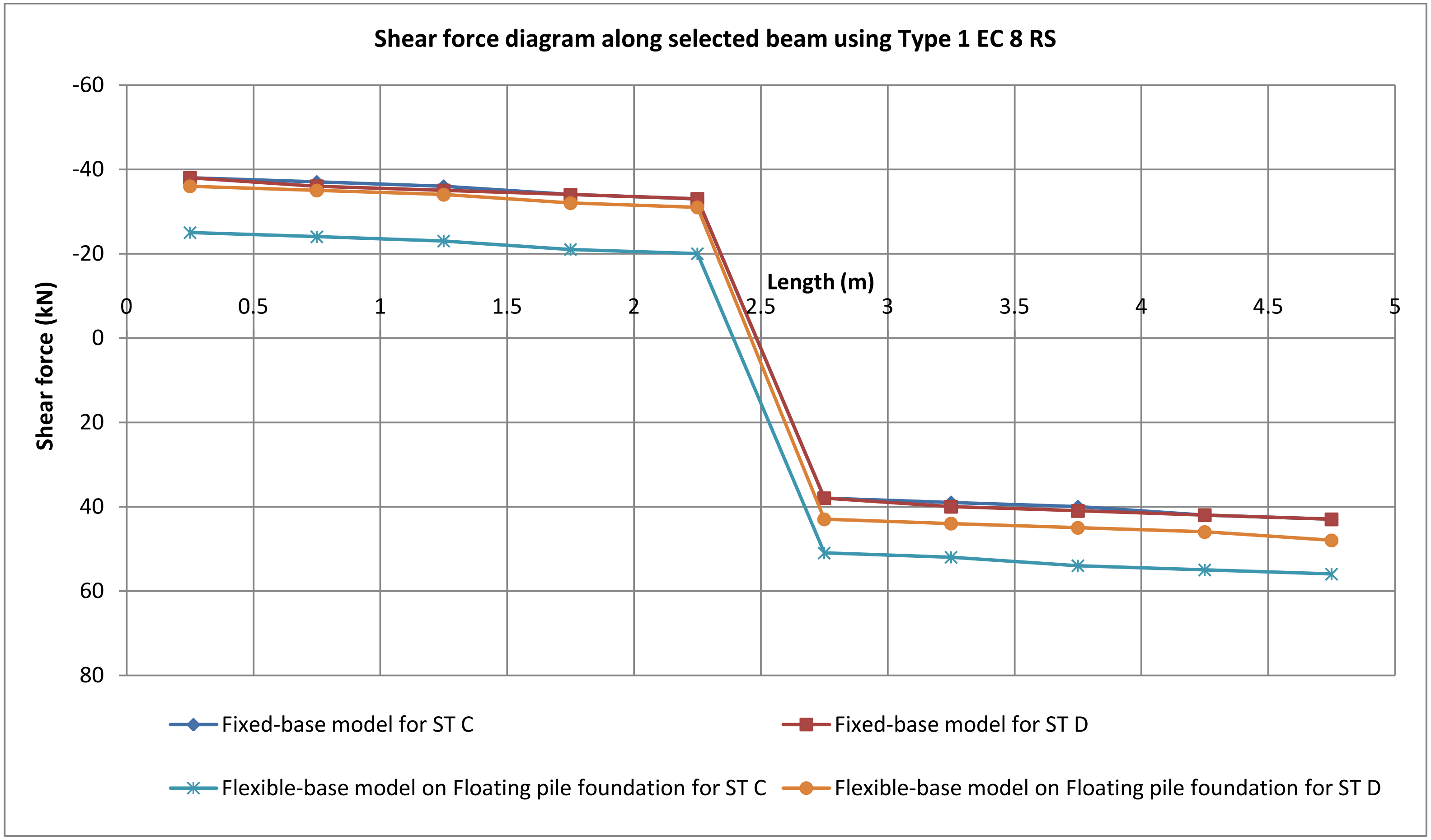


Figure 4.8: Shear force diagram along selected beam of Model 2 (G+10)

4.2.5.2. Models supported on end-bearing pile foundations

The differences in SF variations along beams of models supported on end-bearing piles are observed to increase at one support and decrease at the other support of the beam, diminishing to zero at the mid span as observed for models supported on floating piles. The differences in SFs depend on the heights of the models as it increases with the heights of the models. Besides, the differences in SFs are lesser for flexible-base models supported on shallow foundations than those supported on end-bearing pile foundations.

In addition it is observed that,

- For flexible-base models supported on shallow foundations the SF increases throughout the beam length of Model 1 (G+5); and for the other models the SFs are observed to get increased at one side of the mid span and increase at the other side except for Model 4 (G+20) in soil type C for which the differences in SF get decreased throughout the beam length.
- Type of response spectra has considerable influence on SF differences along beams. SF differences in beams obtained using Type-1 RS are higher than SF differences obtained using Type-2 RS indicating that the effects are considerable as the magnitude of seismic shaking increases.

4.2.6. Effect of SSPSI on Axial forces in columns

The differences in axial forces at columns are observed to increase as the storey level lowers, so that the comparisons are made for columns found at story level 1. The axial forces are compared for columns of axes A, B and C, as shown in Fig. 4.1 in which the differences are higher for the internal axis C columns.

The axial forces in columns get increased for some cases and decreased for other cases when SSPSI is considered. The differences in axial forces in columns for the models are tabulated and presented below.

4.2.6.1. Models supported on floating pile foundations

From Table 4.6 the axial forces in columns of the models supported on floating pile foundations are observed to decrease at some axes and increase at others when SSPSI is considered. The soil types have not significant influence on the axial forces in columns. Flexible-base models supported on floating piles results in more reduced axial forces in columns than similar models supported on shallow foundations.

4.2.6.2. Models supported on end-bearing pile foundations

For models 3 (G+15) and 4 (G+20), the axial forces in columns of the flexible-base models supported on end-bearing piles are observed to significantly increase at the internal axis C when compared with their counterpart fixed-base models for both soil types C and D. However, at the external axes A and B, the axial forces in columns of the flexible-base models supported on end-bearing piles show no significant differences with their counterpart fixed-base models on soil type C; and get reduced when compared with the fixed-base models on soil type D.

In model 5 (G+30) the axial forces in columns of flexible-base models supported on end-bearing piles are observed to be higher than similar fixed-base models at Axis C for both soil types C and D. However, at the external Axis A and B the axial forces in columns of flexible-base models supported on end-bearing piles are observed to be higher than similar fixed-base models on soil type C and lower on soil type D.

The degree by which the axial forces in columns of flexible-base models supported on end-bearing piles gets increased are observed to be higher than similar models supported on shallow foundations when SSI is considered.

It is also observed that the reduction in axial forces in columns due to soil flexibility are more pronounced in flexible-base models supported on floating piles than those models on end-bearing piles. This might be attributed to the lesser stiffness values of floating piles than the end-bearing piles.

For all models, the axial forces in columns obtained using Type 1 response spectrum are higher than that obtained using Type 2 response spectrum.

For the ordinary moment resisting frame system of model 1 the axial forces in the external columns of axis A obtained for flexible-base models are more than the respective fixed-base cases. However, the axial forces are observed to get reduced for the interior columns of axes B, C and D. For the remaining dual frame – shear wall system models the axial forces in the columns become reduced for the medium rise (G+10) model and increased for the rest high rise models.

Table 4.6: Axial forces in columns (kN) of models 1 (G+5) and 2 (G+10)

Analysis case using Type 1 EC-8 RS	Axial force in columns (kN) at Level 1 and along Axis 1 in the transverse direction						
	Model 1(G+5)				Model 2 (G+10)		
	Axis in the longitudinal direction				Axis in the longitudinal direction		
	A	B	C	D	A	B	C
Fixed-base model on ST C	1325	2151	2111	2111	1459	2245	2402
Fixed-base model on ST D	1360	2152	2111	2111	1510	2285	2562
Flexible-base model supported on shallow foundation on ST C	1360	2110	2121	2119	1464	2240	2299
Flexible-base model supported on shallow foundation on ST D	1398	2113	2121	2119	1692	2406	3010
Flexible-base model supported on floating pile foundation on ST C	1318	2043	2093	2099	1567	2021	1468
Flexible-base model supported on floating pile foundation on ST D	1382	1920	1973	1981	1672	2102	1569

Table 4.7: Axial forces in columns (kN) of models 3 (G+15), 4 (G+20) and 5 (G+30)

Analysis case	Axial force in columns (kN) at Level 1 and along Axis 1 in the transverse direction								
	Model 3 (G+15)			Model 4 (G+20)			Model 5 (G+30)		
	Axis in the longitudinal direction			Axis in the longitudinal direction			Axis in the longitudinal direction		
	A	B	C	A	B	C	A	B	C
Fixed-base model on ST C	2867	4312	4143	4400	5300	5866	6487	7408	6642
Fixed-base model on ST D	3048	4418	4787	5133	5782	7505	7696	8401	8225
Flexible-base model supported on shallow foundation on ST C	2838	4249	4106	4176	5065	5512	6343	7225	7115
Flexible-base model supported on shallow foundation on ST D	3136	4466	4851	5152	5786	7096	7737	8384	8808
Flexible-base model supported on end-bearing pile foundation on ST A	2887	4288	4919	4326	5307	6606	6927	8112	10529

4.2.7. Effect of SSPSI on bending moments in shear walls

Bending moments in shear walls for models of dual frame-shear wall system along heights of the shear walls are presented in Figures 4.9 – 4.12. From these figures it is observed that consideration of SSPSI results in reduction of BMs in shear walls at the upper stories. However, at the lower two stories the BMs significantly get increased when SSPSI is considered. The differences in BMs are more pronounced for flexible-base models supported on pile foundations than similar models supported on shallow foundations. The considerably higher moments when soil flexibility is considered at the lower stories might be the reason for the failures at the pile-cap joint observed by Pender (1993).

Flexible-base models supported on shallow foundations show higher differences of BMs in shear walls for Type-D soils than Type-C soils. Type-2 RS results in lesser differences of bending moments in shear walls than Type-1 RS at the upper shear walls. However, at the lower two stories the bending moments obtained using Type-2 RS are considerably higher than those obtained using Type-1 spectrum.

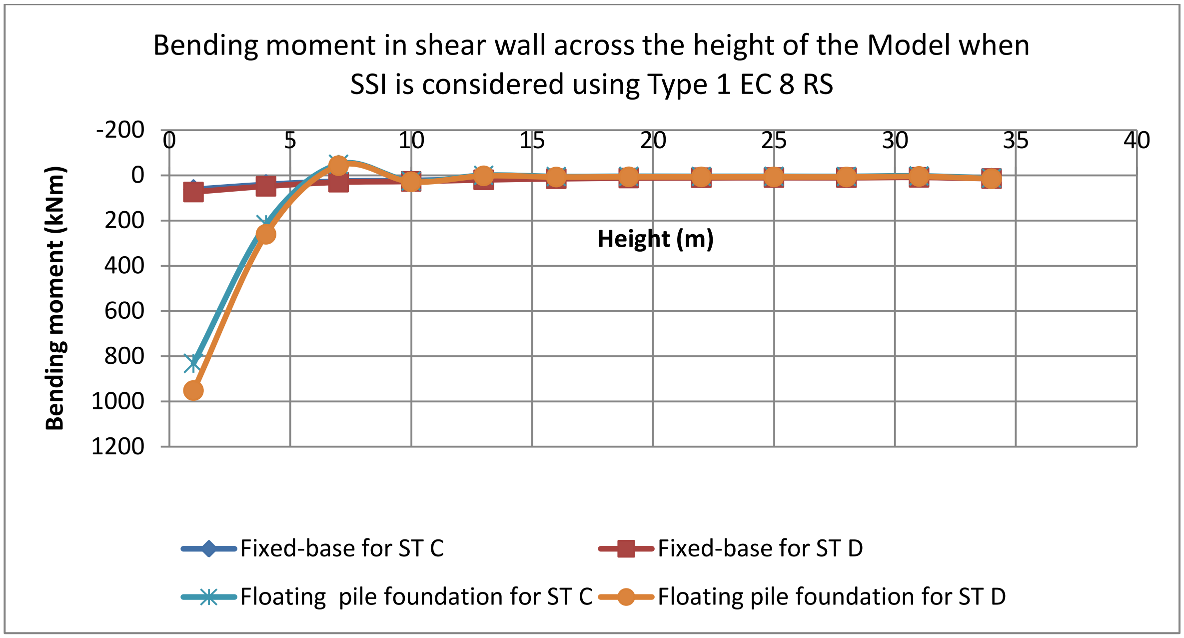


Figure 4.9: Bending moment diagram in shear wall of Model 2 (G+10)

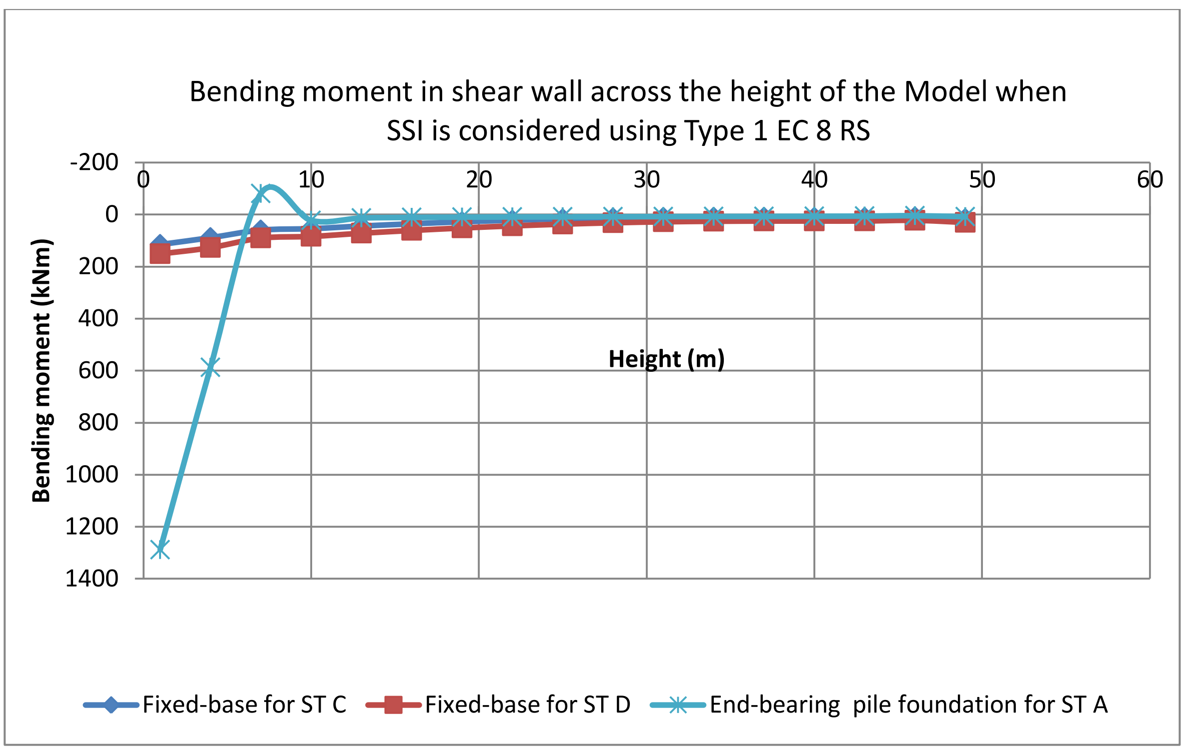


Figure 4.10: Bending moment diagram in shear wall of Model 3 (G+15)

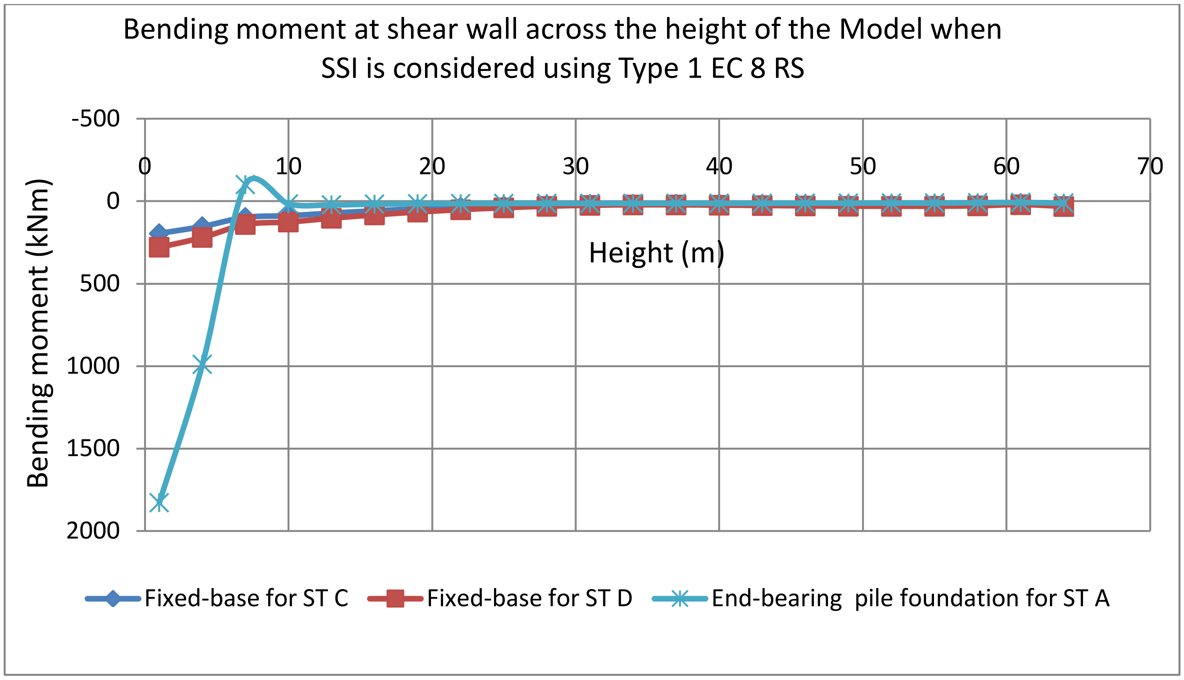


Figure 4.11: Bending moment diagram in shear wall of Model 4 (G+20)

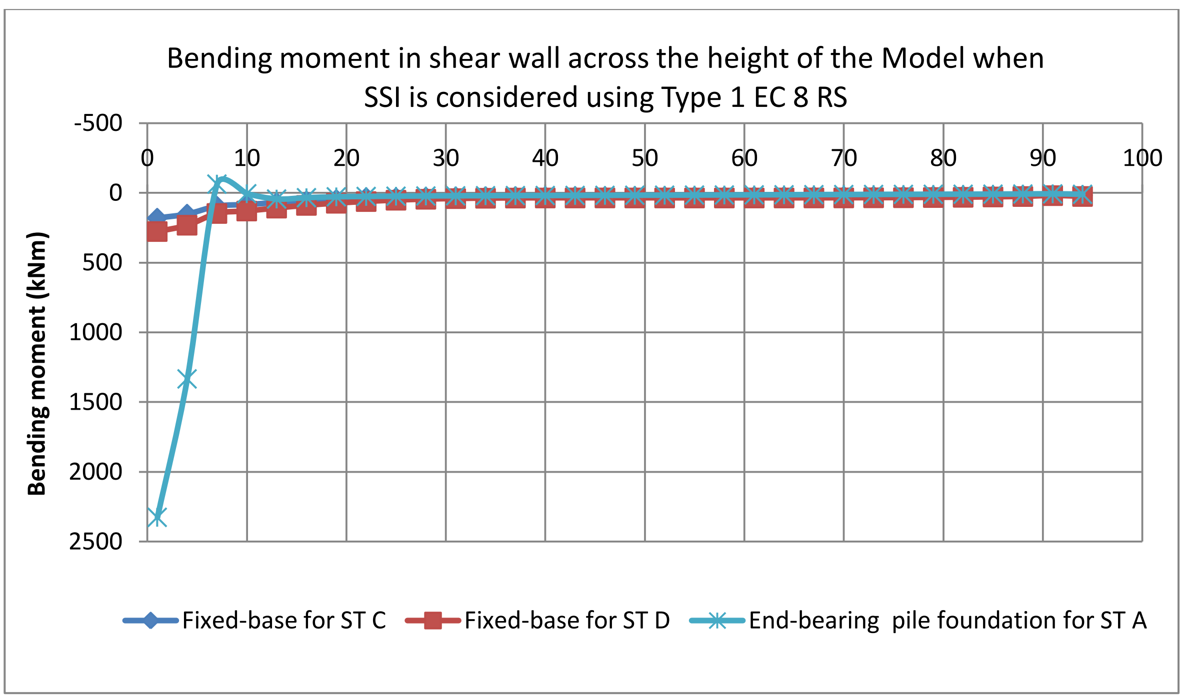


Figure 4.12: Bending moment diagram in shear wall of Model 5 (G+30)

5. CONCLUSIONS AND RECOMMENDATIONS

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

5.1. CONCLUSIONS

- The natural periods of vibrations of flexible-base models supported on pile foundations are elongated more than similar models supported on shallow foundations when soil-foundation-structure interaction effects are considered. The period elongation results in reductions of base shears and overturning moments which are more pronounced for the building models supported on pile foundations than similar building models supported on shallow foundations.
- Bending moments and shear forces in beams are observed to be affected by consideration of the effects of soil flexibility. When soil-foundation-structure interaction effects are considered, both BM and SF are observed to increase with variable magnitudes along beams for all the models supported on shallow foundations except the five-story framed models for which reductions of both BM and SF are observed at some points along beams. However, for models supported on pile foundations, the BM and SF vary along beams showing increments and decrements with variable magnitudes. The effects are more pronounced for models supported on floating pile foundations than the models supported on end-bearing pile foundations. For all models supported on both pile and shallow foundations the differences in BM and SF in beams become zero at the mid span.
- Axial forces in columns and bending moments in shear walls are also observed to be affected by consideration of soil-foundation-structure interaction effects. The axial forces in columns increase at some axes and decrease at other axes when soil

flexibility is considered, the difference being higher for models supported on pile foundations than models supported on shallow foundations. The bending moments in shear walls are observed to get significantly increased at the lower two stories and decrease at the upper stories.

- Soil-foundation-structure interaction effects are more pronounced for pile supported models than models supported on shallow foundations. Types of soils and response spectrum are observed to influence the seismic response of models when soil flexibility is considered.
- For some cases Type-C soil results in higher seismic response than Type-D soil; and for some other cases the converse is observed. Similarly, Type-1 response spectrum results in higher seismic response than Type-2 response spectrum for some cases; and for some other cases the converse is observed, though Type-D soil and Type-1 response spectrum higher differences of seismic responses between the flexible-base and fixed-base models are observed in most of the cases.
- From this study it is observed that consideration of soil-foundation-structure interaction effects results higher or lower seismic responses than the conventional fixed-base analysis approach. By using the flexible-base analysis models economic advantages can be gained when seismic responses are lower and risks due to failures by over loads can be avoided when seismic responses are higher than the conventional fixed-base analysis approach. This emphasizes the importance of considering soil-foundation-structure interaction effects in lieu of the conventional fixed-base analysis approach.
- Using the newer approach of Worku (2013) reduced single and group pile impedances and damping are obtained for the five and ten-story models supported on floating pile foundations as compared with the approach of Gazetas et al (1993) which is followed in this study. Besides, the natural periods for these building models are observed to be more elongated than the natural periods obtained using the approach of Gazetas et al (1993). In addition, the differences in BM and SF

along beams are found to be more than that obtained using the approach of Gazetas et al (1993).

5.2. Recommendations for future studies

In most national seismic codes consideration of soil-foundation-structure interaction effects in usual analysis and designs of buildings is not yet taken as a must to do procedure, except codes like Eurocode 8 (2004) despite the growing concerns on the subject. So, for further investigation, the following ideas are recommended as important extensions of this study.

- The effects of soil flexibility on the inter story displacements of models and the inelastic seismic responses are not covered in this study and can be taken as important extension of this study.
- This study covers only building models supported on pile and shallow foundations. So investigation of the influence of soil-foundation-structure interaction effects for models supported on piled raft foundations is worth studying.

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Declaration

The thesis is my original work, has not been presented for a degree in any other university and that all sources of materials used for the thesis have been duly acknowledged.

Name________________________________

Signature_____________________________