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SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

Assessment of Adjacent RC Buildings Against Free Pounding Distance in Consideration of Soil Structure Interaction

A Thesis in Structural Engineering

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A Thesis

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

The undersigned have examined the thesis entitled '**Assessment of Adjacent RC Buildings Against Free Pounding Distance in Consideration of Soil Structure Interaction**' presented by **ABAYNEH ASMARE,** a candidate for the degree of **Master of Science** and hereby certify that it is worthy of acceptance.

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ABSTRACT

The pounding action mainly depends on the main character which is the peak lateral displacement of the structural systems due seismic action, the influence of soil structure interaction at seismic event in respect to avoid pounding between two adjacently constructed structures is investigated. The modeled structure buildings runs from 6 (G+5) to 26 (G+25) stories with realistic lateral force resisting system and they are studied with different ideal soil class units and with moderate peak ground acceleration value. The result of the structural model systems peak lateral displacement with and without consideration of the effect of soil structure interaction has been presented. Observations of peak lateral displacement regarding the consideration of different class soil type are discussed. The result of the study indicates providing minimum free pounding distance is something elegant to avoid the pounding action in any circumstance and consideration of the soil structure interaction significantly influence the pounding free distance in affirmative way by avoiding pounding that comes from additional peak lateral displacement of the systems.

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A_o	Foundation area
B	Width of foundation
d	Height of effective sidewall contact
D	Depth from surface to bottom of the foundation
G	Shear modulus
h	Depth to centroid of effective
$\overline{h}$	Effective height of the building
h	Height of the building
K_y	Lateral stiffness
K_θ	Rocking stiffness
$K_{x\ sur}$	Equivalent translational spring stiffness along X-axis
$K_{y\ sur}$	Equivalent translational spring stiffness along Y-axis
$K_{z\ sur}$	Equivalent translational spring stiffness along Z-axis
$K_{xx\ sur}$	Equivalent rotational spring stiffness along X-axis
$K_{yy\ sur}$	Equivalent rotational spring stiffness along Y-axis
$K_{zz\ sur}$	Equivalent rotational spring stiffness along Z-axis
L	Length of the foundation
r	Equivalent translational foundation radius
r_o	Equivalent rotational foundation radius
Vs	Shear wave velocity
β_0	Foundation damping factor
β	Damping ratio
β_x	Correction factor for embedment along translational X-axis
β_y	Correction factor for embedment along translational Y-axis
β_z	Correction factor for embedment along translational Z-axis
β_{xx}	Correction factor for embedment along Rocking X-axis
β_{yy}	Correction factor for embedment along Rocking Y-axis
β_{zz}	Correction factor for embedment along Rocking Z-axis
v	Poisson’s ratio
γ	Unit weight of soil

CHAPTER 1 INTRODUCTION

1.1 Background

The first mention of structural pounding in the literature may have been as early as 1926 (Ford, 1926) in which the pounding of non-structural components against the structural elements of a building was discussed and the provision of a sufficient separation gap and proper detailing were recommended.

In past, major earthquake activity, like Mexico City earthquake of 1985, have induced severe structural pounding damage ranges from minor local damage, to total collapse of structure. Cases of structural pounding have been also reported in more recent earthquakes such as the 1994 Northridge earthquake (Tsai, 1997), 1995 Hyogo-ken-Nanbu (Kobe) earthquake (Park et al., 1995) and the 1998 Colombia earthquake.

Since then, the increase in urban development and the associated increase in real-estate values have compelled developers and designers to maximize land usage. Owners want to build their structure aligned with their property line ignoring adjacent structure that lead to pounding at the time of seismic event. In most of the cases checking the minimum pounding free distance, for future earthquake problem that will be applied according to the need, will solve pounding problems. Before design and construction of any structure it is necessary to step out from the routine process and check the surrounding space of the structure to avoid future structural pounding problems.

1.2 Objective

The overall aim of this paper is assessment of a closely constructed structure, reinforced concrete, for maximum lateral displacement to avoid pounding at time of seismic event in consideration of soil-structure interaction (SSI). Also, pointing out the difference in recommended pounding free separation distance between structures with and without consideration of SSI, finding out peak lateral displacement effect for different ideal soil type and height of a structure.

1.3 Scope of the Thesis

In this paper, how to avoid the effect of pounding in the time of seismic action and achieving the exact minimum pounding free gap distance between two closely spaced structures are presented. In addition, it discusses the effect of SSI in the free pounding distance of a structure in three soil classes.

1.4 Methodology

This paper follow different stages (steps) to achieve the aim of the research work. The initial step is literature survey, it studies starting with the cause of earthquake and its background perspective and current view on the topic of both structural pounding and soil structure interaction effect.

As of the second stage code provision insight, the new Ethiopian Building Code (ES EN 1998-1:2015), Euro-Codes and International Building Codes (IBC) is discussed based on how they approach each effect and their recommendation in the analysis and design process. Relating to the topic International relevant books and scientific journals served as important documents to develop the basic formulation view relevant to modeling and analysis of the structure to be considered.

Modeling the sample structures, which have ideally acceptable load resisting system, and collecting of the design data of those sample design model of different height of structure is done in the third step.

In the fourth modeling and analysis, on ETABS, of these sample structures considering major assumption of rigid base situation and finding out related joint displacement is done.

In fifth reanalyzing the whole system to find the related new joint displacement with consideration of soil structure interaction is done by using appropriate code provision.

Finally compare and contrast work is done, depending on the result value of lateral displacement of the structures, between the analysis of the structure with and without consideration of SSI effect and its influence on different types of soil class and with different height of a building.

1.5 Thesis Outline

A brief explanation of each chapter is presented as follows:

Chapter 1 introduce structural pounding and it defines the objectives, methodology and scope of the paper.

Chapter 2 literature survey on pounding and soil structure interaction.

Chapter 3 study code provision for structural pounding and soil structure interaction and analysis approach for the structural models.

Chapter 4 Present modeling and analysis of the structures and its results on fundamental period and lateral displacement at critical floor level with their changes in consideration of soil structure interaction.

Chapter 5 Discusses the outcomes of the analysis and the effect of considering soil structure interaction on the result of lateral displacement and period lengthening effect on the model structures.

Chapter 6 concludes on the effects of soil structural interaction in the dynamic properties of the model structure like fundamental period, lateral displacement and minimum free gap distance that must be achieved between two structures to be constructed adjacently. And it gives recommendation on significance of considering soil structure interaction and further study on its effects.

CHAPTER 2 LITRATURE SURVEY ON POUNDING AND SOIL STRUCTURE INTERACTION

2.1 Earthquake

Lately Scientists have formulated several theories to explain the causes of earthquakes. The theory of plate tectonics seems to explain much of the earthquake activity the world experiences. The continental drift theory, which later developed to model of plate tectonics, was set forth in 1912 by Alfred Wegner. The lithosphere, which is the rigid outer most shell of the earth is broken up in to tectonic plates. The earth's lithosphere is composed of seven or eight plates (depending on how they are defined) and many minor plates. Where the plates meet, their relative motion determines the type of boundary: convergent, divergent or transform as shown in Figure 2-1. The zone where the plates separate from each other is called zone of divergence and where it collides is called zone of convergence and on transform zone the plates slip past each other (USGS).

There is also intra-Plate activity that is well away from the plate boundaries even if they are less understood other activity like fracturing, dam induced seismic action can also cause earthquake.

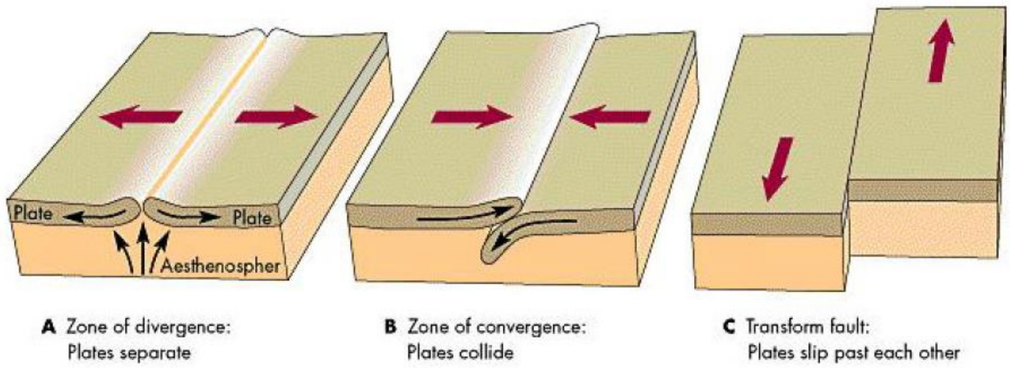


Figure 2-1: Different tectonic movement (USGS)

Generally, earthquake causes many disaster depending on its amplitude and duration in form of ground shaking, landslide, liquification and tsunami. Among this devastating effect of earthquake, structural pounding is one of them not to be overstated in the field of studying these natural phenomena.

2.2 Structural Pounding

Structural pounding is mainly attributed to the difference in the dynamic properties of adjacent structures. The disparities in mass, stiffness, and/or strength result in out-of-phase lateral displacements under external excitations. Impacts will occur if these out-of-phase displacements exceed the available separation gap between the structures. The magnitude of the impact force and the location of impact along the height of the structures depends on the magnitude of the existing separation gap, the extent of the disparity between the dynamic properties of the impacting structures, and the characteristics of the excitation. It is therefore apparent that, under certain conditions, the properties of the supporting soil must also be taken into consideration due to its influence on the above aspects.

The first mention of structural pounding in the literature may have been as early as 1926 (Ford, 1926) in which the pounding of non-structural components against the structural elements of a building was discussed and the provision of a sufficient separation gap and proper detailing were recommended. Since then, the increase in urban development and the associated increase in real-estate values have compelled developers and designers to maximize land usage.

Although the Mexico City earthquake of 1985 is often cited as the most important single event in which extensive pounding damage was reported (Rosenblueth and Meli, 1986), the actual severity of the damage attributed directly to pounding may have been overstated (Anagnostopoulos, 1996), it counts about 15% of the failure case (U.S. Dept. of commerce 1990). Nonetheless, the potential structural and non-structural damage due to pounding should be assessed, be it during the design stage or in the seismic assessment of structures. Sufficient provisions should be implemented to minimize the potential threat to human life (caused by falling debris, e.g. glass or concrete, loss of a structural element, e.g. failure of a column due to sustained pounding at its mid-height and to the worst condition of total collapse of the structure) and to limit the resulting financial losses which may be incurred by the owner(s).

Cases of structural pounding have been reported in more recent earthquakes such as the 1994 Northridge earthquake (Tsai, 1997) (pounding of base-isolated buildings against their stops), the 1995 Hyogo-ken-Nanbu (Kobe) earthquake of 1995 (Park et al., 1995) (collision of pedestrian bridges between buildings) and the 1998 Colombia earthquake.

As population of a country increase land become the scarcest resource, because of the land cost wise utilization of the space becomes not a choice rather an obligation. Owners want to build their structure aligned with their property line ignoring adjacent structure that lead to pounding. In most of the cases checking the minimum pounding free distance, for future earthquake problem that will be applied according to the need, will solve pounding problems. Before design and construction of any structure it is necessary to step out and check the surrounding space of the structure to avoid future pounding problem.

There is also a research paper that is done on seismic pounding case with the topic of "Assessment of Adjacent RC Building in Addis Ababa against pounding" (Kifle, 2013). As the paper writer explains in his scope of the work this paper presents the effect of pounding between two adjacent buildings. This work is limited only to investigate the minimum required separation distance between adjacent buildings to avoid pounding. In addition to this it provides the method of mitigation for buildings which are vulnerable to pounding due to insufficient gap between adjacent buildings. The paper tries to asses this effect of pounding by taking three adjacent RC buildings (six single buildings which have been constructed and are under construction in Addis Ababa). The overall goal of the paper was to investigate the vulnerability of adjacent high rise buildings, which are constructed with insufficient separation distance in Addis Ababa, to pounding failure. And also, to determine the minimum required separation distance between adjacent buildings. Moreover, to provide the method of mitigation of the adjacent buildings which have no sufficient separation distance in between (Kifle, 2013).

2.3 Soil Structure Interaction Effect

In the process of analyzing the seismic response of a structure it is common practice to assume the base of the structure to be fixed, which is a gross assumption since in most situations the foundation soil is flexible. This assumption is realistic only when the structure is founded on solid rock or when the relative stiffness of the foundation soil compared to the superstructure is high. In all other cases, compliance of the soil can induce two distinct effects on the response of the structure, first, modification of the free field motion at the base of the structure, and second, the introduction of deformation from dynamic response of the structure into the supporting soil. The former is referred to as kinematic interaction, while the latter is known as inertial interaction and the whole process is commonly referred to as soil-structure interaction (Arefi, 2008).

2.3.1 Kinematic Interaction

When the earthquake ground motion in the free-field is varying over the area corresponding to that of the rigid foundation, then it can be constrained and modified by the rigid foundation. This deviation from free field motion is called kinematic interaction between the soil and foundation. Moreover, stiffness of the foundation can cause variation of ground motion with depth and scattering of waves at the corners of the foundation (Fig 2.2). If the foundation dimensions are small compared to the wave length of interested frequency range, kinematic interaction has negligible effects on the response (Clough & Penzin, 2003), but if the foundation dimensions are in the same order of the wave length a base slab averaging effect will result (Arefi, 2008).

The output from an analysis accounting for the kinematic interaction is an effective input motion, which is denoted as foundation input motion. The mathematical transformation from the free field motion to the foundation input motion could be performed by a frequency dependent transfer function which is a site-specific curve. Johnson (Johnson, 2003) has shown several experimental tests on different sites and building types. It has been shown that kinematic interaction is important for structures supported on large and stiff foundations (Arefi, 2008).

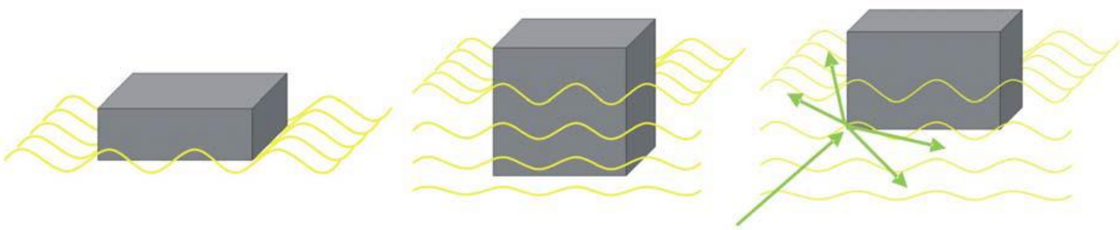


Figure 2-2: Averaging effect (Left), Decreasing motion amplitude with depth (center), Wave scattering at the corners (right). (Arefi, 2008)

Veletsos (Veletsos & Prasad, 1989) and Veletsos (Veletsos et al., 1997) developed several transfer functions between translational and torsional foundation motions and the free field ground motion (Fig. 2.3) which were calibrated later by Kim (Kim & Stewart, 2003) against observed foundation and free field behavior. The transfer function amplitudes computed by Veletsos and his co-workers presented in Figure 2.3 are for circular and rectangular foundations subject to vertically incident incoherent *SH* waves. Similar curves are available for non-vertically incident coherent waves in the references. The transfer functions in Figure 2.3 are prepared such that the foundations dimension *2a* is measured parallel to the direction of *SH* wave polarization, and *2b* is the perpendicular dimension.

The foundation input motion (FIM) can be evaluated using free-field time histories compatible with a design-based acceleration spectrum, through the following procedure:

- Evaluate the Fourier transform of the time history
- Multiplying the acceleration amplitude at each frequency by the corresponding value of transfer function
- Evaluate the inverse Fourier transform of the product.

As experiments, have shown, kinematic effects are more pronounced in pile and very stiff foundations (Kramer & Stewart, 2004), therefore kinematic effects are ignored in this dissertation and only inertial interaction is considered (Arefi, 2008).

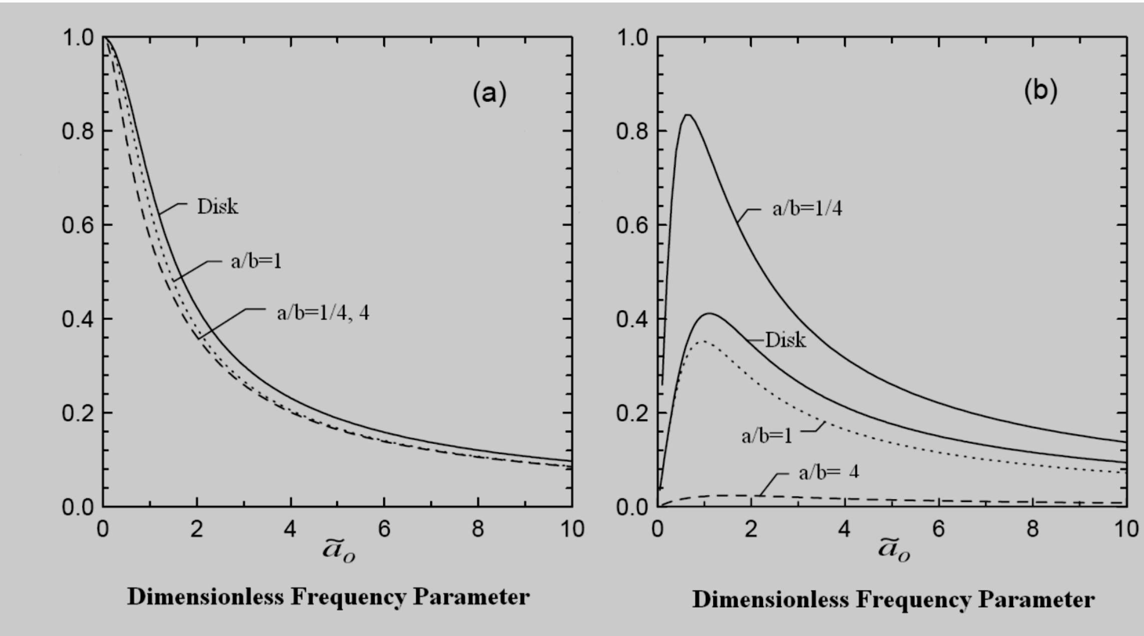


Figure 2-3: Amplitude of transfer function between free-field motion and foundation input motion for different foundation shapes. (a) Translational motion (b) Torsional motion (Veletsos & Prasad, 1989)

2.3.2 Inertial Interaction

The effect due to the existence of soft soil under the foundation of the structure is referred to as inertial interaction. Inertial forces induced in the structure by the foundation motion during the earthquake can cause the compliant soil to deform which in turn affects the super-structure inertial forces. This deformation propagates away from the structure into the soil in form of waves. In other words, the dynamic response of the superstructure decreases. This removal of energy from the system is referred to as radiation damping in the literature. Wolf (1994) used a viscous damper to take into account the radiation damping. The coefficient of the viscous damper is proportional to the wave velocity in the soil and the foundation area. This increase in effective damping is significant for a soil site approaching a homogeneous elastic half space sections (Wolf, 1994; Arefi, 2008).

The compliance of soil foundation is correspondent to a stiffness value. This can be combined with radiation damping properties of foundation in a complex impedance function as it is denoted in soil-structure interaction (SSI) problems. Semi-infinite continuum can be replaced by a simple single degree of freedom (SDOF) mechanical massless model supported by a spring and a dashpot of coefficients, *K* and *C*, respectively, as shown in figure 2-4. Evaluating the spring and viscous damper properties and using the foundation motion as input motion, a more realistic dynamic analysis of the system can be carried out (Arefi, 2008).

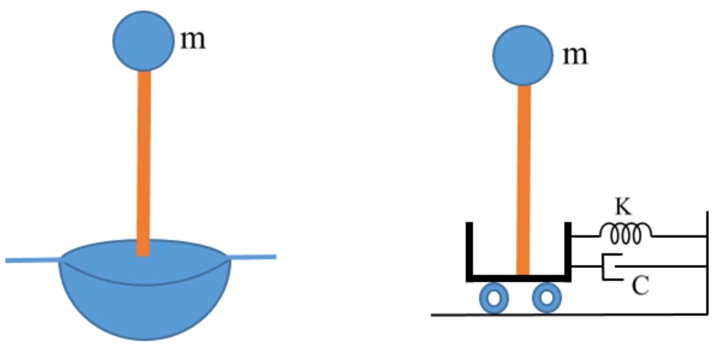


Figure 2-4: Simplified homogeneous elastic half space soil impedance model with spring and dashpot

The effect of SSI can be assessed by comparing the system responses with and without springs and dashpots. This effect is usually accompanied with period lengthening due to compliant soil added to the system and mostly increasing damping due to the radiation effects, (Figure 2.5). The new properties of the system can be evaluated in closed-form for a single degree of freedom structures. Figure 2.5 shows schematically the result of period lengthening and increase of damping for the response of a SDOF structure. It is obvious that when using a general acceleration response spectrum, consideration of SSI effects will reduce the response of the SDOF system (Arefi, 2008), but sometimes considering SSI increases the response of the system by increasing the total displacement and secondary effect P-Δ.

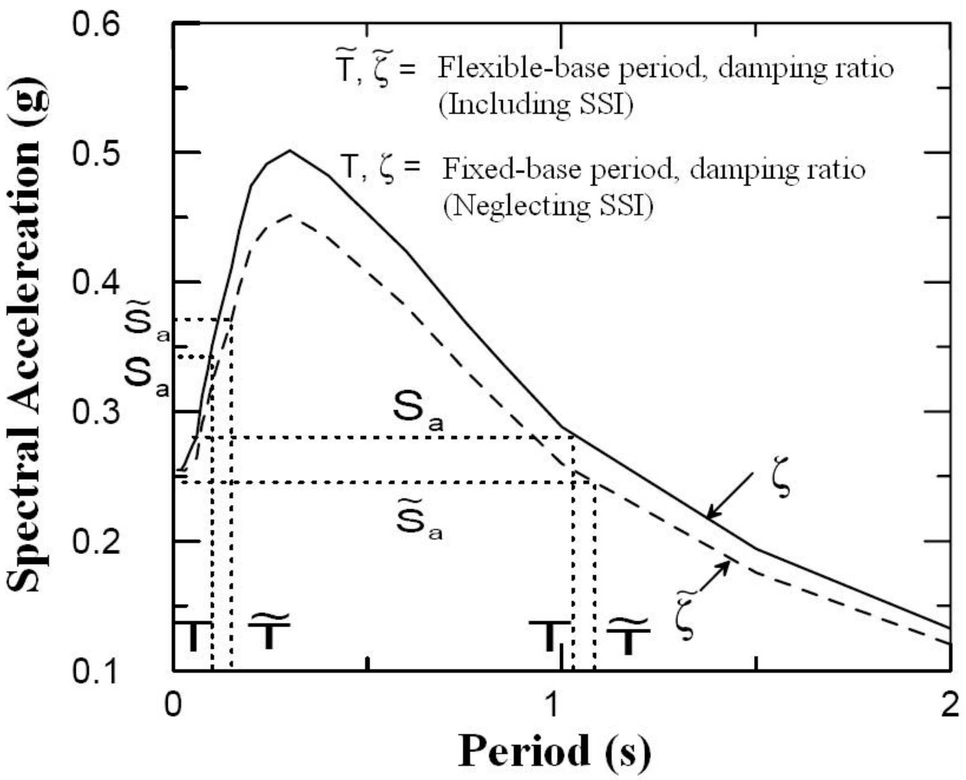


Figure 2-5: Schematic representation of period lengthening and damping increase as a result of considering SSI effect in dynamic response analysis of a SDOF structure (Stewart et al, 2003).

In general, it is not possible to determine a priori whether the inertial interaction effects will decrease or increase the response of the system. It has been shown in most of the cases ignoring the interaction effect is conservative, in some cases it can be detrimental.

Gazetas & Mylonakis, (2001) showed that in certain seismic and soil environments, an increase in the fundamental period of a moderately flexible structure due to SSI may result in an increase in the seismic demand.

CHAPTER 3 CODE PROVISION FOR STRUCTURE POUNDING AND SOIL STRUCTURE INTERACTION

3.1 Background

Design codes are tools that guide a designer to establish the principles and requirements for safety and serviceability of a structure to be constructed. There are specific sections in each code approved by any country which can address different problems and procedures. In this paper our main consideration is seismic induced action on structures covered mainly in ES EN 1998-1:2015 that will give us brief explanation and design procedure.

ES EN 1998-1:2015 (Ethiopian building code of standard: design of structure for earthquake resistance) applies to the design and construction of a structure in seismic region and it has the purpose to ensure the protection of human life, structural damage limit and to give operational chances to important structure like hospital, firefighting building and specific chemical (radiation) related structure at the time of earthquake or seismic excitation.

3.2 Ground Condition

Based on the provision of ES EN 1998-1:2015, the ground conditions are classified under five types (A, B, C, D &E) and additional two unique sub-classes (S_1&S_2) (Table 3-1) according to the soil property and its influence in response to seismic action.

Respective of its hazard condition to seismic action and other effects the code put a precondition before classifying any construction site in to the ground conditions expressed in Table 3-1, that the site must be

- Free from ground rupture
- Slope instability
- Permanent settlement that caused by liquefaction and densification at the time of seismic excitation

Table 3-1: Site soil class according to ES EN 1998-1:2015

Ground type	Description of stratigraphic profile	Parameters		
		$v_{s,30}$ (m/s)	N_{SPT} (blows/30cm)	c_u (kPa)
A	Rock or other rock-like geological formation, including at most 5 m of weaker material at the surface.	> 800	_	_
B	Deposits of very dense sand, gravel, or very stiff clay, at least several tens of meters in thickness, characterized by a gradual increase of mechanical properties with depth.	360 – 800	> 50	> 250
C	Deep deposits of dense or medium-dense sand, gravel or stiff clay with thickness from several tens to many hundreds of meters.	180 – 360	15 - 50	70 - 250
D	Deposits of loose-to-medium cohesionless soil (with or without some soft cohesive layers), or of predominantly soft-to-firm cohesive soil.	< 180	< 15	< 70
E	A soil profile consisting of a surface alluvium layer with v_s values of type C or D and thickness varying between about 5 m and 20 m, underlain by stiffer material with v_s > 800 m/s.			
S_1	Deposits consisting, or containing a layer at least 10 m thick, of soft clays/silts with a high plasticity index (PI > 40) and high water content	< 100 (indicative)	_	10 - 20
S_2	Deposits of liquefiable soils, of sensitive clays, or any other soil profile not included in types A – E or S_1			

3.3 Seismic Zone

Based on ES EN 1998-1:2015, the country is divided into different seismic hazard zone with assumption that in the same zone seismic hazard action activity remains fairly the same. The classification of this seismic hazard zone is done based on a parameter α_0 called bed rock acceleration ratio, which is a ratio of design bed rock acceleration a_{gR} to gravitational acceleration g.

The code gives the seismic hazard map of Ethiopia (Figure 2-6) based on peak ground acceleration value (PGA). According to the hazard map the earthquake sensitive area goes with the rift valley alignment.

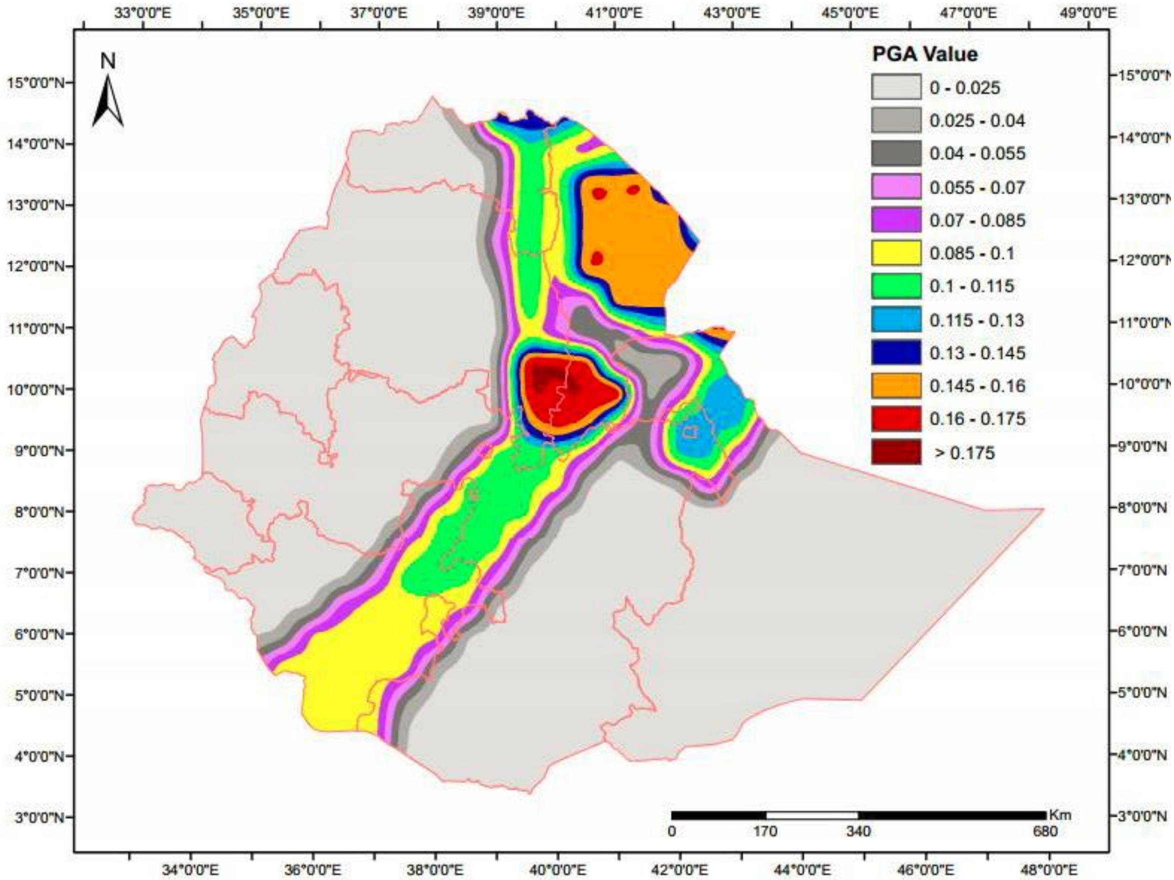


Figure 3-6: Seismic hazard map of Ethiopia according to ES EN 1998-1:2015

For design purpose the seismic hazard map is divided in to 5 zones relative to their sensitivity to earthquake as shown in Table 3-2. Major cities like Hawassa, Jijiga, Nazret, Debre birhan, Dessie, Mekelle and Woldiya are in critical seismic zone which is nominated as zone 5 and zone 4 and the capital Addis Ababa is happen to be in zone 3.

Table 3-2: Bed rock acceleration ratio (ES EN 1998-1:2015)

Zone	5	4	3	2	1	0
α_0	0.20	0.15	0.1	0.07	0.03	0

3.4 Code provision for Structural Pounding

I. ES EN 1998-1:2015 (Euro-code 8 EN 1998-1)

Buildings shall be protected from earthquake-induced pounding from adjacent structures or between structurally independent units of the same building. This principle is considered to be satisfied:

a) if the distance from the property line to the potential point of impact is not less than the maximum horizontal displacement of the building, at the corresponding level for buildings or structurally independent units that do not belong to the same property.

b) if the distance between the two buildings or units at the corresponding level is not less than the square root of the sum- of the squares (SRSS) of the maximum horizontal displacement for buildings or structurally independent units belonging to the same property. In both condition the horizontal displacement is analyzed in accordance with expression of Euro-code 8 EN 1998-1 section 4.23.

A specification is made: If the floor elevations of the building or independent unit under design are the same as those of the adjacent building or unit, the above referred minimum distance may be reduced by a factor of 0.7. The "damage limitation requirement" is considered to have been satisfied, if, under a seismic action having a larger probability of occurrence than the design seismic action corresponding to the "no-collapse requirement".

In this paper, the sample structure models are designed and analyzed in respect to ES EN 1998-1:2015 which is similar to Euro-code 8 EN 1998. In section 5 of EN 1998-1:2004 there are specific rules about concrete buildings design concept, structural types and behavior factor for horizontal seismic action. These specific rules, quoted from Eurocode EN 1998-1:2004 section 5.2.2.2 here under, are used through the process of modeling, designing and analysis of the sample model structures.

5.2.2.2. Behavior factors for horizontal seismic actions

(1) P The upper limit value of the behavior factor q, introduced in ***3.2.2.5(3)*** *to account for energy dissipation capacity, shall be derived for each design direction as follows:*

$$q = q_o k_w \geq 1.5 \quad (3.1)$$

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 (see **(2)** of this sub clause);*

k_w *is the factor reflecting the prevailing failure mode in structural systems with walls (see **(11)**P of this sub clause).*

*(2) For buildings that are regular in elevation in accordance with **4.2.3.3**, the basic values of* q_o *for the various structural types are given in Table 3-3 (on the code table 5-1).*

Table 3-3: Basic value of the behavior factor, q_o, for systems regular in elevation

STRUCTURAL TYPE	DCM	DCH
Frame system, dual system, coupled wall system	$3.0\alpha_u/\alpha_1$	$4.5\alpha_u/\alpha_1$
Uncoupled wall system	3.0	$4.0\alpha_u/\alpha_1$
Torsionally flexible system	2.0	3.0
Inverted pendulum system	1.5	2.0

*(3) For buildings, which are not regular in elevation, the value of qo should be reduced by 20% (see **4.2.3.1(7)** and Table 4.1).*

(4) α1 and αu are defined as follows:

α1 is the value by which the horizontal seismic design action is multiplied in order to first reach the flexural resistance in any member in the structure, while all other design actions remain constant;

αu is the value by which the horizontal seismic design action is multiplied, in order to form plastic hinges in a number of sections sufficient for the development of overall structural instability, while all other design actions remain constant. The factor αu may be obtained from a nonlinear static (pushover) global analysis.

(5) When the multiplication factor αu/ α1 has not been evaluated through an explicit calculation, for buildings which are regular in plan the following approximate values of αu/α1 may be used.

a) Frames or frame-equivalent dual systems.

- *One-story buildings: αu/ α1=1,1;*
- *Multistory, one-bay frames: αu/ α1=1,2;*
- *Multistory, multi-bay frames or frame-equivalent dual structures: αu/ α1=1,3.*

b) Wall- or wall-equivalent dual systems.

- *wall systems with only two uncoupled walls per horizontal direction: αu/ α1=1,0;*
- *other uncoupled wall systems: αu/ α1=1,1;*
- *Wall-equivalent dual, or coupled wall systems: αu/ α1=1,2.*

(6) *For buildings, which are not regular in plan (see **4.2.3.2**), the approximate value of αu/ α1 that may be used when calculations are not performed for its evaluation are equal to the average of (a) 1,0 and of (b) the value given in **(5)** of this sub clause.*

(7) *Values of αu/α1 higher than those given in **(5)** and **(6)** of this sub-clause may be used, provided that they are confirmed through a nonlinear static (pushover) global analysis.*

(8) *The maximum value of αu/ α1 that may be used in the design is equal to 1.5, even when the analysis mentioned in **(7)** of this sub clause results in higher values.*

(9) *The value of qo given for inverted pendulum systems may be increased, if it can be shown that correspondingly higher energy dissipation is ensured in the critical region of the structure.*

(10) *If a special and formal Quality System Plan is applied to the design, procurement and construction in addition to normal quality control schemes, increased values of qo may be allowed. The increased values are not allowed to exceed the values given in Table 5.1 by more than 20%.*

(11) *P The factor kw reflecting the prevailing failure mode in structural systems with walls shall be taken as follows:*

$$k_w = \left\{ \begin{array}{l} 1.00,\ \text{for frame and frame-equivalentdual systems} \\ (1+\alpha_0)/3 \le 1,\ \text{but not less than 0.5, for wall-equivalent and torsionally} \\ \text{flexible systems} \end{array} \right\} \quad \ldots \; \ldots .3.2$$

Where α_o is the prevailing aspect ratio of the walls of the structural system.

(12) *If the aspect ratios h_{wi}/l_{Wi} of all walls i of a structural system do not significantly differ, the prevailing aspect ratio αo may be determined from the following expression:*

$$\alpha o = \Sigma h_{wi} / \Sigma l_{wi} \qquad (3.3)$$

where h_{wi} is the height of wall i; and

l_{wi} is the length of the section of wall i.

(13) *Systems of large lightly reinforced walls cannot rely on energy dissipation in plastic hinges and so should be designed as DCM structures*

II. International Building Code (IBC) 2009

IBC also specifies spacing between the adjacent buildings equal to the square root of the sum of squares (SRSS) of the individual building displacements. Following is an excerpt of the relevant specification from IBC 2009.

1613.6.7 (2009) ***Building separations****. All structures shall be separated from adjoining structures. Separations shall allow for the displacement δ_M. Adjacent buildings on the same property shall be separated by at least, δ_{MT},*

Where $$\delta_{MT} = \sqrt{(\delta_{M1})^2 + (\delta_{M2})^2} \quad 3.4$$

And δ_{M1} and δ_{M2} are the displacements of the adjacent buildings. When a structure adjoins a property line not common to a public way, which structure shall also be set back from the property line by at least the displacement, δ_M, of that structure.

Exception*: Smaller separations or property line setbacks shall be permitted when justified by rational analyses based on maximum expected ground motions*

III. Ethiopian Building Code of Standard (Previous EBCS-8)

Specifies about the separation distance of adjacent buildings as follows: - (23)

Seismic Joint Condition (2.4.2.7)

1. Building shall be protected from collisions with adjacent structures induced by earthquake.

2. The requirement of (1) above is deemed to be satisfied if, the distance from the boundary line to the potential point of impact is not less than the maximum horizontal displacement given as:

$$d_s = \frac{d_e}{\gamma_d} \quad 3.5$$

Where d_s displacement of a point of the structural system induced by the design seismic action,

γ_d displacement behavior factor, assumed equal to γ unless otherwise specified in Chapter 3,

d_e displacement of the same point of the structural system, as determined by a linear analysis

Where γ_d displacement behavior factor equal to γ and can be expressed as follow.

$$\gamma = \gamma_o k_D k_R k_W \qquad 3.6$$

γ_o is basic value of the behavior factor, depending on the structural type. The values is given in the table below

Table 3-4: Basic value γ_o of behavioral factors

Structural Type		γ_o
Frame System		0.2
Dual System	frame equivalent	0.2
	wall equivalent with coupled walls	0.2
	wall equivalent with uncoupled walls	0.2
Wall System	with coupled walls	0.2
	with uncoupled walls	0.25
Core System		0.3
Inverted Pendulum System		0.5

k_D is factor reflecting the ductility class and given as: -

$$k_D = \begin{cases} 1.00 \text{ for DC'' H''} \\ 1.50 \text{ for DC'' M''} \\ 2.00 \text{ for DC'' L''} \end{cases}$$

k_R is factors reflecting the structural regularity in elevation and shall be given as follows.

$$k_R = \begin{cases} 1.00 \text{ for regular structures} \\ 1.25 \text{ for non-regular structures} \end{cases}$$

k_W is factors reflecting the prevailing failure made in structural system with walls shall be taken as follows

$$k_W = \begin{cases} 1.00 \text{ for frame, frame equivalent dual system.} \\ (2.5-0.5\alpha o) \text{ for wall, wall equivalent system} \\ \geq 1 \text{ and core system} \end{cases}$$

Therefore, the value of displacement behavior factor can be calculated using equation 3.3 as follows: -

$\gamma_d = \gamma = \gamma_o k_D k_R k_W$ *by substituting the value of the factors we get: -* $\boldsymbol{\gamma_d = 0.50}$

3. If the floor elevations of a building under design are the same as those of the adjacent building, the distance referred in (2) above may be reduced by a factor of 0.7.

4. Alternatively separation distance is not required if, appropriate shear walls are provided on the primate of the building to act as collision walls ("bumpers"). At least two such walls must be placed at each side subject to pounding and must extend over the total height of the building. Then must be perpendicular to the side subject to collisions and they can end on the boundary line. Then the separation distance for the rest of the building can be reduced to 40mm. 3a-d)

3.5 Code Provision for Soil Structure Interaction

The main concept of site response analysis is that the free field motion is dependent on the properties of the soil profile including stiffness of soil layers. The stiffness of the deposit can change the frequency content and amplitude of the ground motion. Likewise, on the path to the structure, wave properties might be changed due to the stiffness of the foundation. In fact, kinematic interaction is the inability of the foundation to conform to the deformations of the free field ground (Kramer, 1996). On the other hand, the inertial forces and moments induced by structure to the foundation can change the ground motion too. These two effects are discussed in more detail in the following sections (Arefi, 2008). According to the code provision of NEHRP there are two different possible approaches to the problem that can be used to assess the effects of soil-structure interaction.

a.) The first involves modifying the stipulated free-field design ground motion, evaluating the response of the given structure to the modified motion of the foundation, and solving simultaneously with additional equations that define the motion of the coupled system, whereas

b.) The second involves modifying the dynamic properties of the structure and evaluating the response of the modified structure to the prescribed free-field ground motion (Jennings and Bielak, 1973; Veletsos, 1977).

When properly implemented, both approaches lead to equivalent results. However, the second approach, involving the use of the free-field ground motion by modifying the dynamic property of the structure, is more convenient for design purposes and provides the basis of the requirements.

There are also other sophisticated views to find the influence of soil structure interaction. The procedures proposed in these complex preceding sections for incorporating the effects of soil-structure interaction provide sufficient flexibility and accuracy for practical applications. Only for unusual structures and only when the requirements indicate that the interaction effects are of definite consequence in design, would the use of more elaborate procedures be justified. Some of the possible refinements, listed in order of more or less increasing complexity, are:

1. Improve the estimates of the static stiffness's of the foundation, K_y and K_θ, and of the foundation damping factor, β_0, by considering in a more precise manner the foundation type involved, the effects of foundation embedment, variations of soil properties with depth, and hysteretic action in the soil (BSSC 2003).

Solutions may be obtained in some cases with analytical or semi-analytical formulations and in others by application of finite difference or finite element techniques. A concise review of available analytical formulations is provided by Gazetas, (1991). It should be noted, however, that these solutions involve approximations of their own that may offset, at least in part, the apparent increase in accuracy (BSSC 2003).

2. Improve the estimates of the average properties of the foundation soils for the stipulated design ground motion. This would require both laboratory tests on undisturbed samples from the site and studies of wave propagation for the site. The laboratory tests are needed to establish the actual variations with shearing strain amplitude of the shear modulus and damping capacity of the soil, whereas the wave propagation studies are needed to establish realistic values for the predominant soil strains induced by the design ground motion (BSSC 2003).

3. Incorporate the effects of interaction for the higher modes of vibration of the structure, either approximately by application of the procedures recommended in Bielak (1976), Roesset et al. (1973), and Tsai (1974) or by more precise analyses of the structure-soil system. The latter analyses may be implemented either in the time domain or by application of the impulse response functions presented in Veletsos and Verbic (1973). However, the frequency domain analysis is limited to systems that respond within the elastic range while the approach involving the use of the impulse response functions is limited, at present, to soil deposits that can adequately be represented as a uniform elastic half space. The effects of yielding in the structure and/or supporting medium can be

considered only approximately in this approach by representing the supporting medium by a series of springs and dashpots whose properties are independent of the frequency of the motion and by integrating numerically the governing equations of motion (Parmelee et al., 1969; BSSC 2003).

4. Analyze the structure-soil system by finite element method (for example, Lysmer et al., 1981; Borja et al., 1992), taking due account of the nonlinear effects in both the structure and the supporting medium (BSSC 2003).

3.6 Analysis Approach

The new Ethiopian code, ES EN 1998-1:2015, gives two types of linear-elastic analysis approach depending on the structural characteristics of the building;

- Static (lateral force method of analysis)
- Dynamic (modal response spectrum analysis)

The model response spectrum analysis is applicable to all types of buildings, but there is precondition (ES EN 1998-1:2015 section 4.3.3.2) set by the code to use static analysis approach which is

(1) This type of analysis can be applied to buildings whose response is not significantly affected by contributions from higher modes of vibration.
(2) Must be regular in both plan and elevation
(3) Have fundamental period of vibration $T_1 \leq 2$ sec in the two-main direction

If one of the above three conditions aren't met, the code requires the use of dynamic analysis as the preferred approach.

In this work, response spectrum analysis (RSA) approach is used with the help of structural analysis software program for the system with and without consideration of foundation flexibility (SSI effect) because some of the structures fundamental period is above 2 sec.

To analyze the effect of SSI, the paper follows two approaches, which are the code method and spring model analysis. In code method, the analysis is done by modifying the dynamic property of the structure to account for the flexibility of the soil and in spring model analysis, we use spring with a value of stiffness to account for the SSI effect instead of fixed base situation. Both approaches are followed using the structural analysis software, ETABS.

3.6.1 Fundamental Period Determination

Generally, each structure has a unique natural or fundamental period of vibration a time that is required to complete one cycle in the first mode of the vibration. Height, mass and stiffness of the structure are major input in finding the fundamental period of a structure.

The determination of the fundamental period and mode shapes can be performed using modal analysis available in the structural software program according to the selected pertinent code. As a result, we take the significant modes of the system for the possible maximum lateral deflection at the time.

3.6.2 Fundamental Period Lengthening Determination in Consideration of SSI

According to NEHRP, the effect of SSI takes into consideration by modifying the structure fundamental period T to modified effective period $\overline{T}$. The modified effective period is dependent on the fundamental period and the soil flexibility translational and rocking stiffness property. using the provision of NEHRP.

$$\overline{T} = T\sqrt{1 + \frac{\overline{K}}{K_y} + \frac{\overline{K}\overline{h}^2}{K_\theta}} \qquad 3.7$$

The portions in the radical format represent translational (K_y) and rocking (K_θ) stiffness of the flexible foundation system and effective stiffness ($\overline{K} = 4\pi^2\ (\overline{w}/gT^2)$) and height ($\overline{h}$) of the fixed base structure. For building foundation systems having lateral continuity, such as mats or footings interconnected with grade beams, stiffnesses K_y and K_θ can often be approximated as

$$K_y = 8\left(\frac{GR_h}{2-v}\right) \qquad 3.9$$

$$K_\theta = \left(\frac{8GR_\theta{}^3}{3(1-v)}\right) \qquad 3.10$$

where: R_h = an equivalent foundation radius that matches the area of the foundation, A_0 (i.e., $R_h = \sqrt{(A_0/\pi)}$); R_θ = an equivalent foundation radius that matches the moment of inertia of the foundation in the direction of shaking. (i.e., R_θ = $\sqrt[4]{(4I_o/\pi)}$; G = the strain-dependent shear modulus, as defined in the standard; v = the soil Poisson's ratio.

The expression above is static stiffness to get the dynamic foundation stiffness we introduce dynamic modifiers, α_h and α_θ. The dynamic foundation stiffnesses are dependent on the soil type, foundation shape, foundation embedment and structural properties. The coefficients α_h and α_θ are generally frequency-dependent dynamic modifiers applied on the respective static stiffness given by Equation (3-9&3-10) above for the horizontal and rocking motion, respectively.

$$K_y = 8\left(\frac{GR_h}{2-v}\right)\alpha_h \qquad 3.11$$

$$K_\theta = \left(\frac{8GR_\theta{}^3}{3(1-v)}\right)\alpha_\theta \qquad 3.12$$

The modifier α_h may be taken as unity for all practical purposes, the modifier α_θ must be established depending on the ratio R_m/v_sT (BSSC 2004,2010: Worku,2014) which is dimensionless coefficient that depends on the period of excitation, the dimensions of the foundation, and the properties of the supporting medium (Luco, 1974; Veletsos and Verbic, 1973; Veletsos and Wei, 1971).

Foundation embedment has the effect of increasing the stiffnesses K_y and K_θ. For embedded foundations for which there is positive contact between the side walls and the surrounding soil, K_y and K_θ may be determined from the following approximate formulas (BSSC 2004,2010).

$$K_y = K_y\left[1+\frac{2}{3}\frac{d}{R_h}\right] \qquad 3.13$$

$$K_\theta = K_\theta\left[1+2\frac{d}{R_\theta}\right] \qquad 3.14$$

3.6.3 Effective damping modification due to SSI

The effective flexible-base damping $\bar{\beta}$ is contributed from both the structural viscous damping β and the foundation damping β_o consisting generally of radiation and material damping components. Veletsos & Nair (1975) established the following relationship for the system damping based on equivalence of maximum deformations of the two oscillators. (Worku, 2014)

$$\bar{\beta} = \beta_O + \frac{\beta}{\left(\frac{\bar{T}}{T}\right)^3} \qquad 3.15$$

The plots of Equation (3-15) against the period ratio are given in Figure 3-7 for the commonly assumed fixed-base structural damping (FBSD) of 5% and a number of foundation damping (FD) values ranging from 3% to 20%. Such ranges of foundation damping ratios have been reported in the past (Stewart *et al* 2003; Worku, 2014).

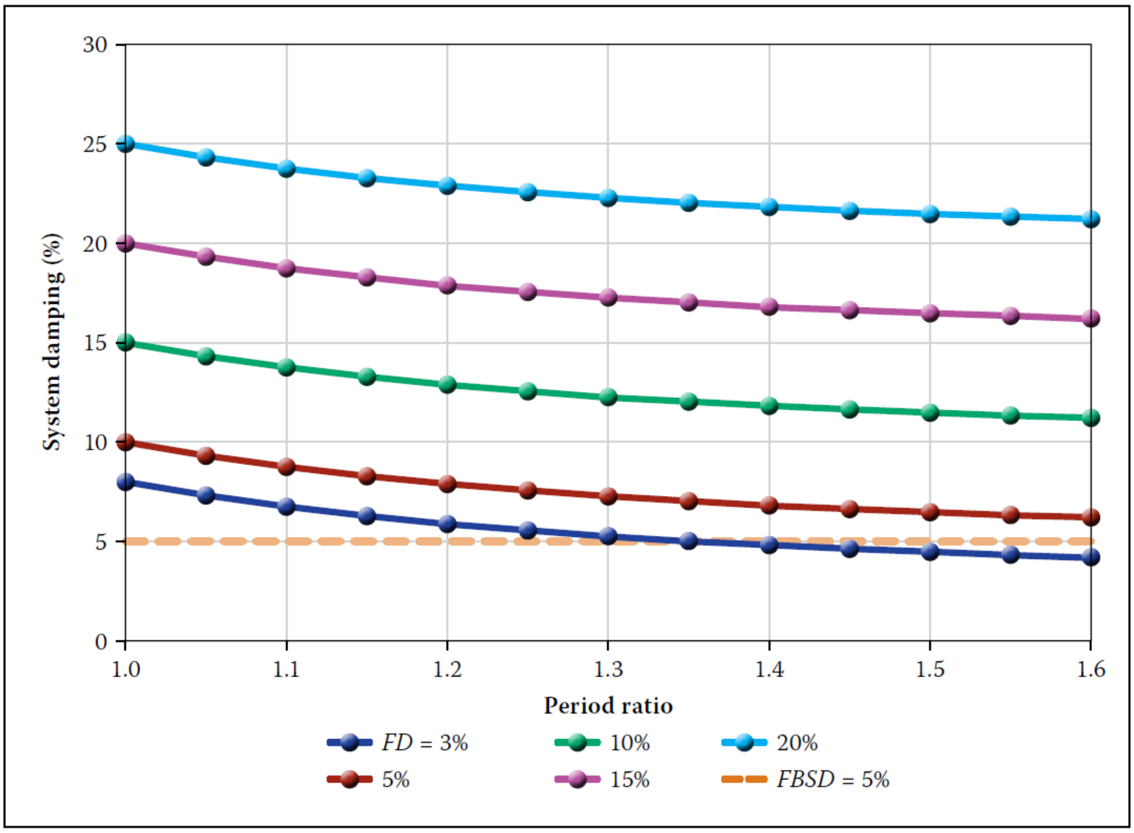


Figure 3-7: Variation of the system damping with $\bar{T}/T$ for different foundation damping (Worku, 2014).

The plots show that the overall effective damping of the flexible-base system is larger than the fixed-base damping (5%) with the exception of the rare case of the foundation damping itself being very low (smaller than 5%), and the period ratio being large. For any given foundation damping, the system damping gradually decreases with increasing period ratio due to the decreasing contribution of the structural damping with increasing period ratio. It should, however, be noted that the effective damping may not generally be taken less than the structural damping of 5% (BSSC 2004, BSSC 2010; Worku, 2014).

Foundation damping factor β_o represents the damping contributions from foundation-soil interaction (with hysteretic and radiation components) (Stewart et al, 1999). For the simple case of a circular foundation with radius r on a uniform half-space with soil hysteretic damping ratio β, the relationships between the fixed- and flexible-base oscillator properties depend on $\bar{h}/r$ and the design spectral response acceleration parameter at short period S_{DS}.

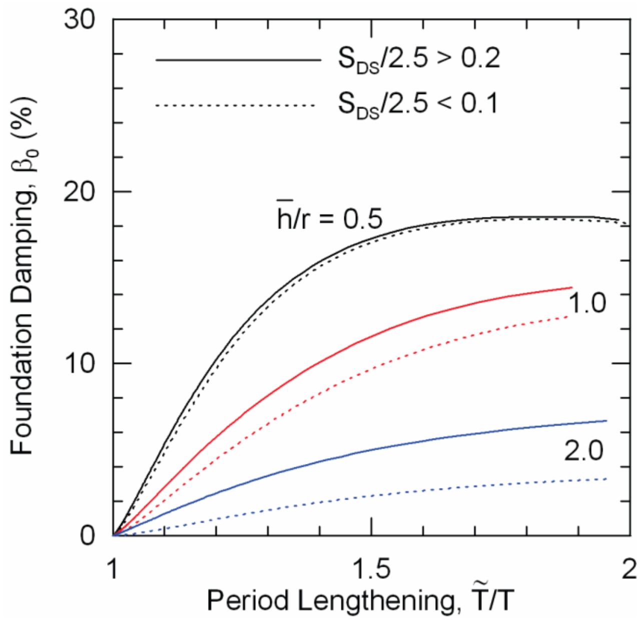
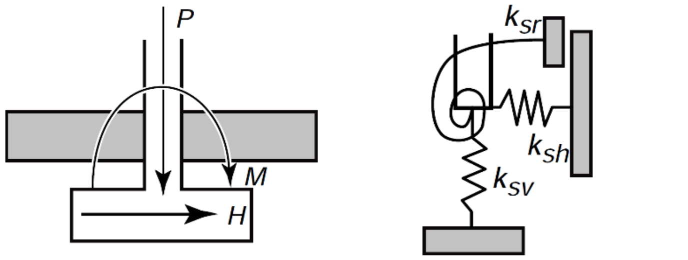


Figure 3-8: Foundation Damping Factor (NEHRP, FEMA P-750 2009)

3.6.4 Spring model analysis

This spring model analysis system is presented here as alternative approach to the code based method of analysis. The structure will be modeled as it stands on spring, with coined stiffness value in consideration of foundation size, embedment and soil property of the site, with six degree of freedom.



Foundation load **Uncoupled spring model**

Figure 3-9: Uncoupled spring model representation (NEHRP, FEMA P-750, 2009)

Researchers have developed spring stiffness solutions that are applicable to any solid base mat shape on the surface of, or partially or fully embedded in, a homogeneous half space (Gazetas, 1991). Rectangular foundations are most common in buildings. Therefore, the general spring stiffness solutions were adapted to the general rectangular foundation problem, which includes rectangular strip footings. Using Table 3-5 & 3-6, a two-step calculation process is required. First, the stiffness terms are calculated for a foundation at the surface. Then, an embedment correction factor is calculated for each stiffness term.

$$K_{eff(i)} = \beta_{(i)} * K_{sur} \qquad 3.16$$

Here below are the code provisions for the equivalent spring stiffness values with amplification correction depending on embedment depth, foundation size and shape factor.

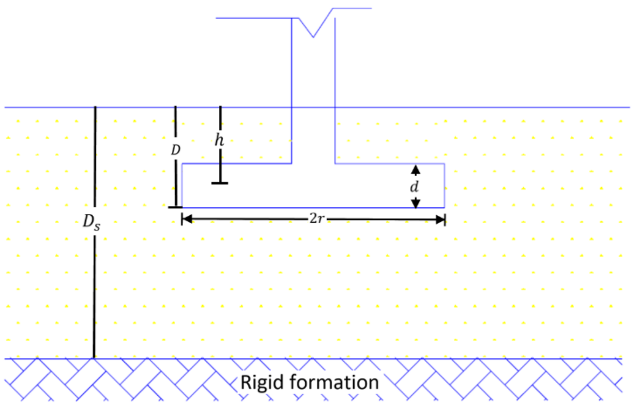
Table 3-5: Spring stiffness at the surface (Umal,2015; Pais and Kausel 1988)

	Translation stiffness	Rocking Stiffness
Along X-axis	$\boldsymbol{K_x = \frac{GB}{2-\mu}\left[3.4\left(\frac{L}{B}\right)^{0.65} + 1.2\right]}$	$\boldsymbol{K_{xx} = \frac{GB^3}{1-\mu}\left[0.4\left(\frac{L}{B}\right) + 0.1\right]}$
Along Y-axis	$\boldsymbol{K_y = \frac{GB}{2-\mu}\left[3.4\left(\frac{L}{B}\right)^{0.65} + 0.4\frac{L}{B} + 0.8\right]}$	$\boldsymbol{K_{yy} = \frac{GB^3}{1-\mu}\left[0.47\left(\frac{L}{B}\right)^{2.4} + 0.034\right]}$
Along Z-axis	$\boldsymbol{K_z = \frac{GB}{1-\mu}\left[1.55\left(\frac{L}{B}\right)^{0.75} + 0.8\right]}$	$\boldsymbol{K_{yy} = GB^3\left[0.53\left(\frac{L}{B}\right)^{2.45} + 0.51\right]}$

	Embedment correction factor (Translation)	Embedment correction factor (Rocking)
Along X-axis	$\boldsymbol{\beta_x = \left(1 + 0.21\sqrt{\frac{D}{B}}\right).\left[1 + 1.6\left(\frac{hd(B+L)}{BL^2}\right)^{0.4}\right]}$	$\boldsymbol{\beta_{xx} = 1 + 2.5\frac{d}{B}\left[1 + \frac{2d}{B}\left(\frac{d}{D}\right)^{-0.2}\sqrt{\frac{B}{L}}\right]}$
Along Y-axis	$\boldsymbol{\beta_y = \left(1 + 0.21\sqrt{\frac{D}{B}}\right).\left[1 + 1.6\left(\frac{hd(B+L)}{BL^2}\right)^{0.4}\right]}$	$\boldsymbol{\beta_{yy} = 1 + 1.4\left(\frac{d}{L}\right)^{0.6}\left[1.5 + 3.7\left(\frac{d}{L}\right)^{1.9}\left(\frac{d}{D}\right)^{-0.6}\right]}$
Along Z-axis	$\boldsymbol{\beta_z = \left[1 + \frac{D}{21B}\left(2 + 2.6\frac{B}{L}\right)\right]\left[1 + 0.32\left(\frac{d(B+L)}{BL}\right)^{2/3}\right]}$	$\boldsymbol{\beta_{zz} = 1 + 2.6\left(1 + \frac{B}{L}\right)\left(\frac{d}{B}\right)^{0.9}}$

Table 3-6: Correction factor for embedment (Umal,2015; Pais and Kausel 1988)

Figure 3-10: Vertical soil stratum and soil Dimension



CHAPTER 4 STRUCTURAL MODELING AND ANALYSIS

Structural pounding comes from seismic induced lateral displacement and taking a consideration of flexible base system (SSI) will give a different modified effective period, damping ratio and as a result new lateral displacement.

This paper follows the following process of analysis

- Finding fundamental period (T) from the modal analysis generated from the software program in consideration of fixed base foundation system.
- Analyzing period lengthening ratio value to find the modified fundamental period of the structure and corresponding effective damping ratio value for the Code method SSI effect
- Analyzing Six-degree of freedom equivalent stiffness value to replace the soil flexibility for the alternative spring model analysis.
- Analyzing critical floor lateral displacement for fixed base model, spring model and Code method (modified dynamic property T and β) for each structure for the three different ideal soil types B, C and D.

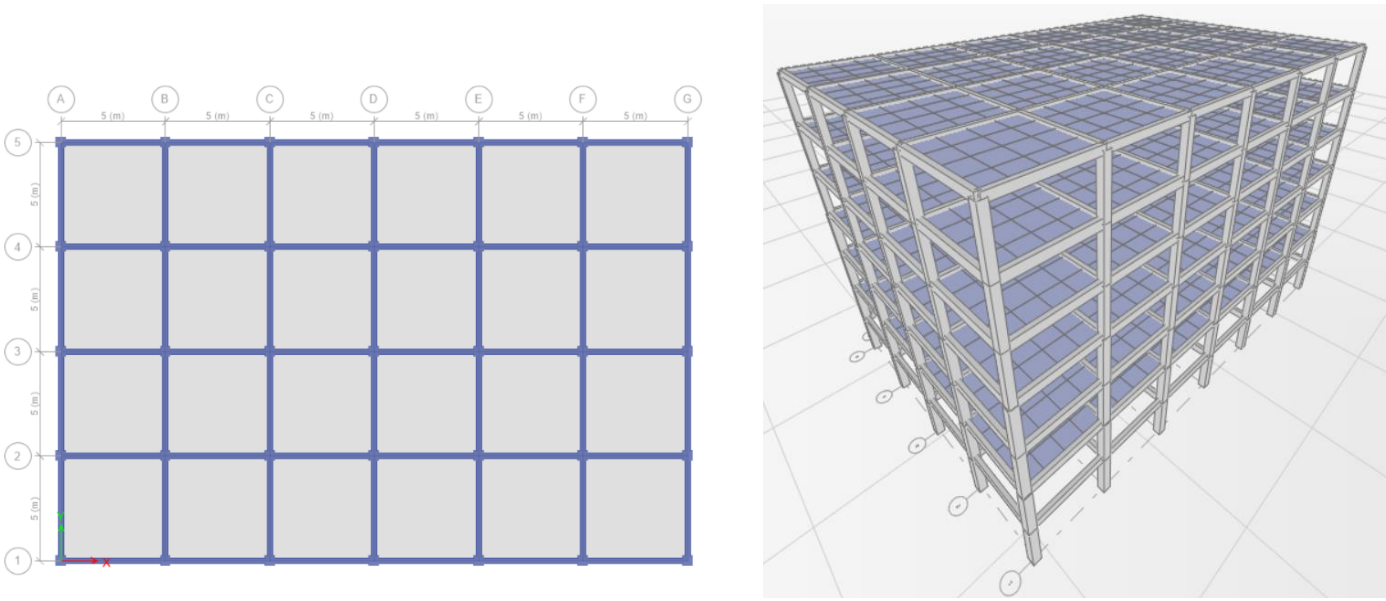
The structural systems are modeled to present more realistic structural and architectural appearance, to represent the common building that are constructed in our country, with progressive floor level to see each lateral displacement effect with incremental height of a building. The study contains 5 structural models, a G+5 structure with frame system and structures with dual system (frame and shear wall) with different incremental floor level G+10, G+15, G+20 and G+25.

4.1 Model Structural Systems

The structural systems have a uniform floor height of 3m with concrete roof slab. Reasonable live and dead load on the slabs are taken according to code requirements. Concrete material of grade C-25 is taken with Shear wall thickness of 30-75 cm is provided depending on floor height starting form G+10 structure. Incremental column size from 40cmX40cm to 100cmX100cm is taken as per requirement of the service and ultimate limit state capacity. Floor beam size taken in the system are 25cmX50cm for all floor systems except in the ground floor, where the beams are 25cmX40cm.Floor slab with a uniform thickness of 16 cm are assigned. Each structural system is described as per their shear wall arrangement and a 3D perspective of the system.

Structural System-I

The first structural system is a regular reinforced concrete building with 6 floor (G+5) on isolated footings. The beams are 25cmX50cm for all floors except in the ground floor, Where the beams are 25cmX40cm.The columns are incremental in size depending on the floor system requirement of ultimate and service limit state.

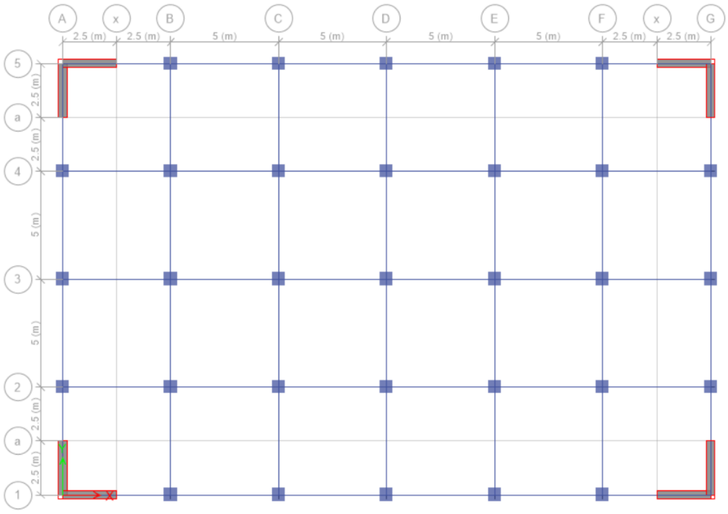


a). typical floor plan b) 3D model of the structure

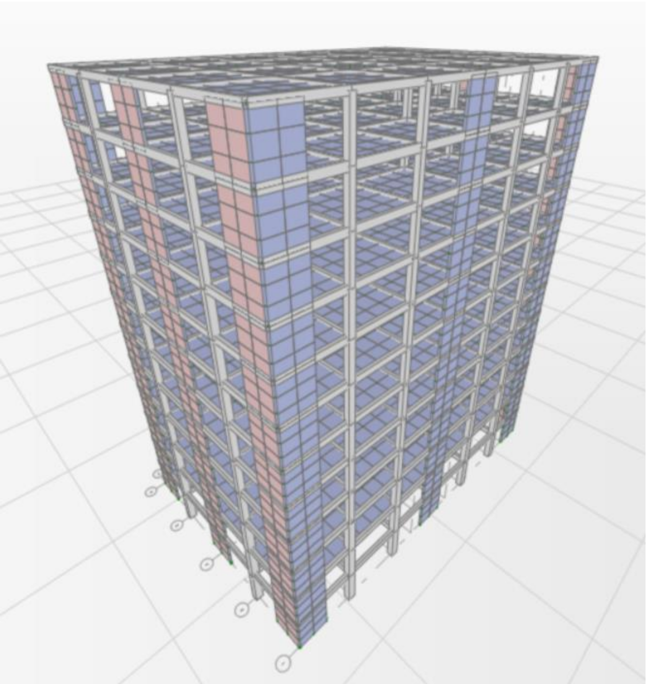
Figure 4-11: Floor Plan and 3D model Configuration for structural system I

Structural system-II

A structural system, regular reinforced concrete structure, with dual (both frame and shear wall) lateral force resisting system of 11 floors building (G+10) on a mat foundation.



a). typical floor plan

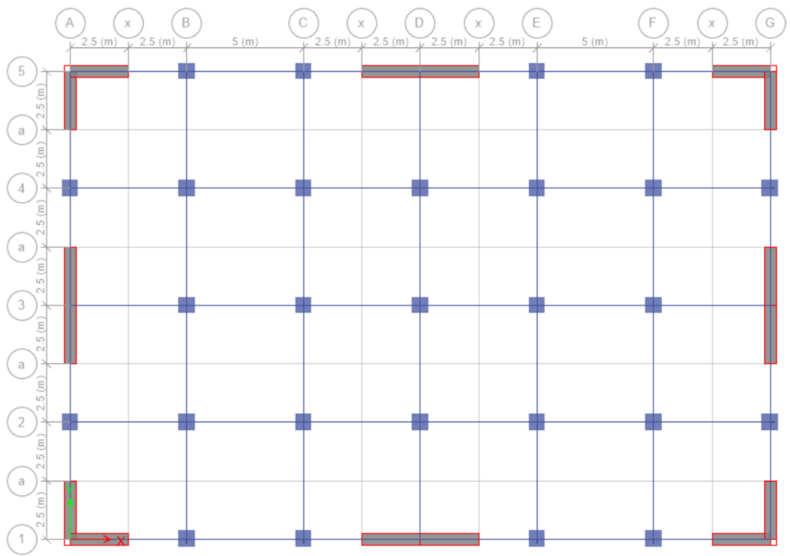


b) 3D model of the structure

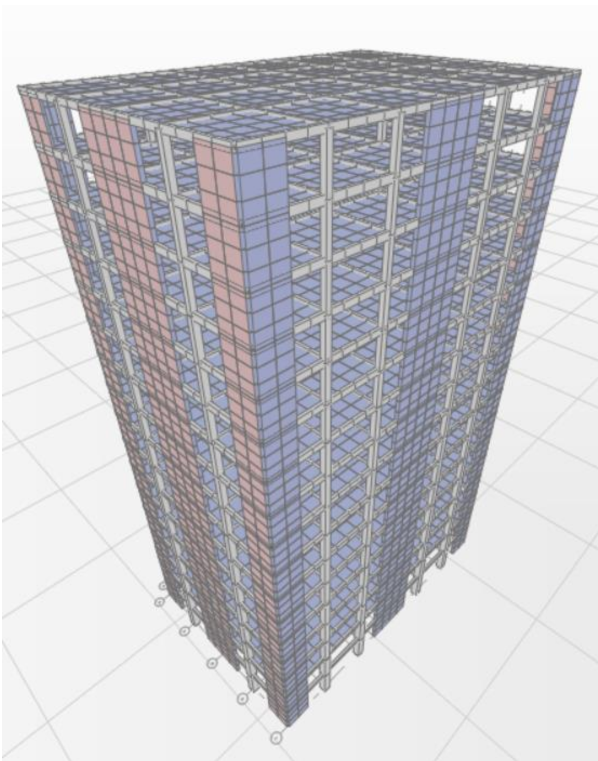
Figure 4-12: Floor Plan Configuration with shear wall arrangement for structural system III

Structural System-III

A structural system, regular reinforced concrete structure, with dual (both frame and shear wall) lateral force resisting system of 16 floors (G+15) on a mat foundation.



a). typical floor plan

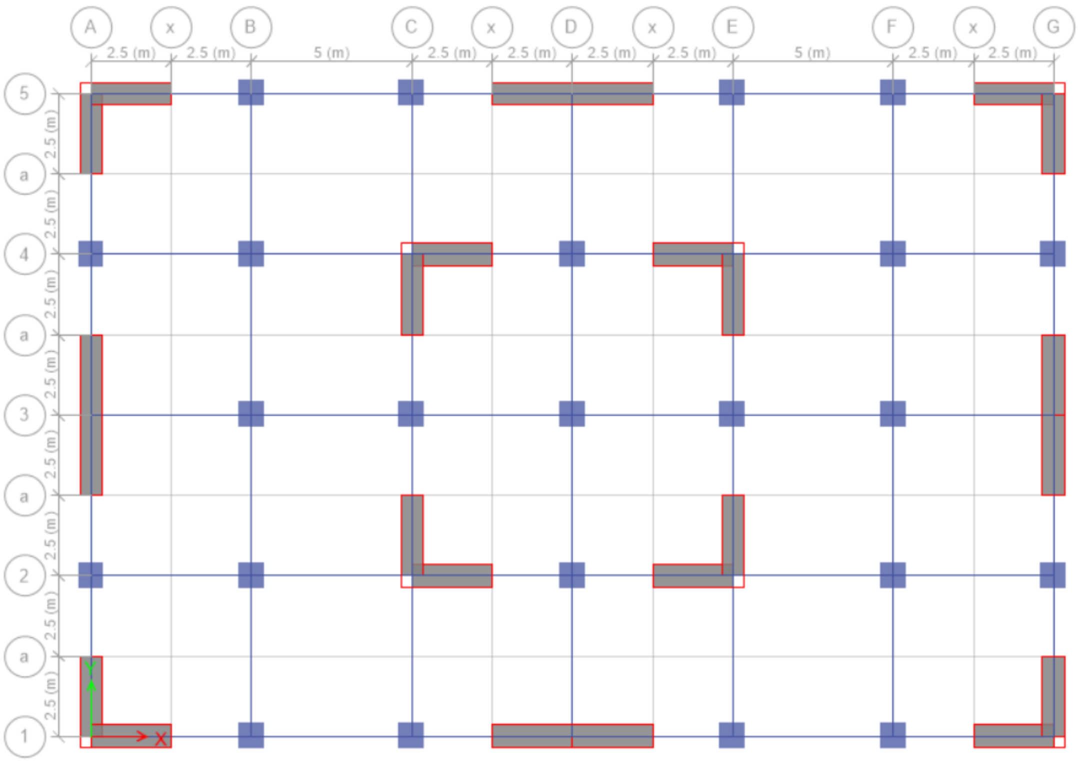


b) 3D model of the structure

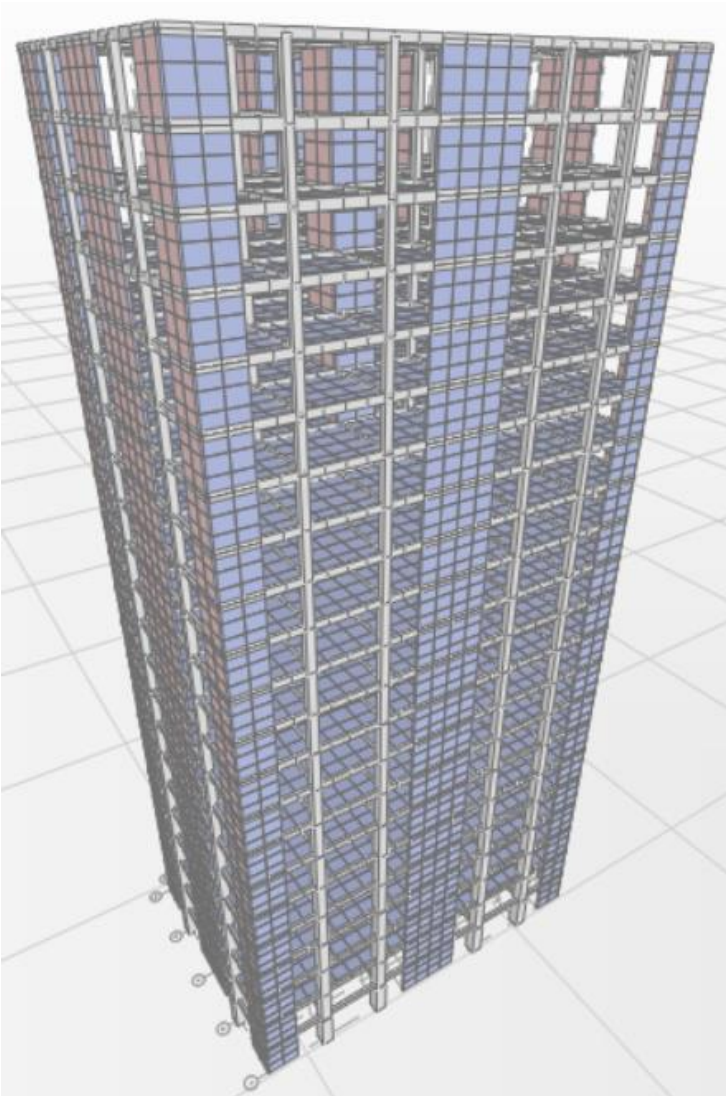
Figure 4-13: Floor Plan Configuration with shear wall arrangement for structural system V

Structural System-IV and V

This two-structural systems (structural system IV and V) have identical shear wall and frame arrangement for lateral force resisting system with regular reinforced concrete structure of 21 floors (G+20) and 26 floors (G+25) respectively both on a Mat foundation



a). Typical floor plan of structural system IV and V



b) 3D model of the structure

Figure 4-14: Floor Plan Configuration with shear wall arrangement for structural system IV&V.

4.2 Result of the model structural systems

4.2.1 Structure system -I (G+5)

The period of each structural model analysis is done by the software program resulting from the response spectrum modal analysis is given below.

Period (T)	Mode-1	Mode-2	Mode-3	Mode -4	Mode-5	Mode-6	Mode-7	Mode-8	Mode -9
	1.054	1.027	0.912	0.348	0.340	0.306	0.219	0.217	0.203

4.2.1.1 Code method analysis

The modified period of vibration is calculated according to the NEHRP as described by equation 3.7 in previous section and its detail.

Table 4-7: Imposed & self-weight load on the system with the modified period

G+5			LOAD (KN)		
	Area (m^2)	No of floor	Imposed	Self-weight	Total
Roof floor	600	1	1800	2250	4050
Floor (1^{st}-5^{th})	600	5	18000	11520	29520
Ground		1		624	624
				Total	34194

	Sub soil class		
	B	C	D
ϒ (KN/m^3)	16	16	16
v	0.4	0.4	0.4
Vs (m/s)	600	350	150
G (KN/m^2)	587155.96	199796.13	36697.25
R_h (m)	13.82	13.82	13.82
R_θ (m)	12.63	12.63	12.63
α_h	1	1	1
R_m/V_sT	0.02	0.034	0.080
α_θ	1	1	0.955
K_y (N/m)	42294317.74	14538292.48	2697202.38
K_θ (N.m)	5923556826	2072314058	373439307.1
$\overline{K}$ (KN/m)	224010	224010	224010
$\overline{T}/T$	1.003	1.008	1.044

The next table shows the result of effective damping coefficient depending on the period lengthening ratio and the soil type.

Table 4-8: Effective damping coefficient of structure system I

	Soil class	$\overline{T}/T$	Bo	β (%)
G+5	B	1.003	0.15	5.107
	C	1.008	0.20	5.077
	D	1.044	0.94	5.330

4.2.1.2 Spring stiffness Analysis

Depending on the equation given in table 3-5 each direction translational (K_x, K_y & K_z) and rocking stiffness (K_{xx}, K_{yy} & K_{zz}) calculated and will be multiplied by embedment correction factor, which is given in table 3-6, per each directional perspective.

Table 4-9: Surface stiffness for structural system I

	Soil class		
	B	C	D
ϒ (KN/m^3)	16	16	16
ν	0.4	0.4	0.4
Vs (m/s)	600	350	150
G (KN/m^2)	587155.9633	199796.1264	36697.24771
B (m)	3.7	4.2	4.75
α_h	1	1	1
R_m/V_sT	0.02	0.034	0.080
α_θ	1	1	0.955
$K_{x(sur)}$(N/m)	6245871.56	2412538.226	501146.789
$K_{y(sur)}$ (N/m)	6245871.56	2412538.226	501146.789
$K_{z(sur)}$ (N/m)	7350214.067	2839102.956	589755.3517
$K_{xx(sur)}$ (N.m)	24784342.51	12335412.84	3129943.138
$K_{yy(sur)}$ (N.m)	24784342.51	12335412.84	3129943.138
$K_{zz(sur)}$ (N.m)	30930859.45	15394595.23	3906169.037

Table 4-10: Correction factor for embedment

Soil class	D	d	H	B	L	β x	β y	β z	βxx	βyy	βzz
B	0.80	0.40	3.40	3.70	3.70	2.25	2.25	1.21	1.74	1.91	2.31
C	1.00	0.50	3.40	4.20	4.20	2.24	2.24	1.23	1.84	1.98	2.43
D	1.20	0.60	3.40	4.75	4.75	2.22	2.22	1.24	1.91	2.03	2.51

And at the end K_{eff} is analyzed for each direction by multiplying each embedment correction value with its corresponding surface stiffness, like for sample translational x-axis corrected formula as $K_{eff(x\ )} = \beta_{(\ x)} * K_{sur}$

Table 4-11: Corrected equivalent spring stiffness

		Soil class		
	Stiffness	B	C	D
Translation stiffness (N/m)	$K_{eff\,(x)}$	14062775.7	5414684.0	1112227.4
	$K_{eff\,(y)}$	14062775.7	5414684.0	1112227.4
	$K_{eff\,(Z)}$	8901144.6	3481596.3	729106.0
Rocking stiffness (N.m)	$K_{eff\,(xx)}$	43224620.8	22721741.5	5976261.648
	$K_{eff\,(yy)}$	47390471.0	24451764.4	6352124.054
	$K_{eff\,(zz)}$	71462554.6	37395840.2	9794514.094

Using the corrected spring constant as a constraint for the structure we have another modal analysis result depending on the ideal soil type

Table 4-12: Modal analysis by the spring method

	Modal analysis of soil type B								
Period (T)	Mode-1	Mode-2	Mode-3	Mode -4	Mode-5	Mode-6	Mode-7	Mode-8	Mode -9
	1.056	1.028	0.913	0.348	0.341	0.306	0.219	0.218	0.203
	Modal analysis of soil type C								
	Mode-1	Mode-2	Mode-3	Mode -4	Mode-5	Mode-6	Mode-7	Mode-8	Mode -9
	1.058	1.030	0.914	0.348	0.341	0.306	0.220	0.218	0.203
	Modal analysis of soil type D								
	Mode-1	Mode-2	Mode-3	Mode -4	Mode-5	Mode-6	Mode-7	Mode-8	Mode -9
	1.065	1.035	0.917	0.349	0.341	0.306	0.220	0.218	0.203

And generally taking the first modal result as fundamental period of the structure the period ratio is tabulated below.

Table 4-13: Period lengthening ratio of spring model

	Sub soil class		
$\bar{T}/T$	B	C	D
	1.002	1.004	1.010

4.2.1.3 Lateral joint displacement

As It has been discussed in the previous section, we analyze the system in three steps to find the possible maximum lateral deflection. The first method is nominal fixed base analysis with fundamental period T and we reanalyze the structures by changing the restraints of fixed base system to spring with equivalent stiffness value as second method. Using the code method, as third step, we change the dynamic property of the system (fundamental period and damping coefficient) and reanalyze again the previous fixed base structure to have the possible maximum lateral deflection. Here below is max possible joint displacement by EQ-Y spectrum in cm unit for both analysis method (code and spring model) in the ideal soil classes including the nominal fixed base situation.

Table 4-14: Lateral joint displacement at roof level of G+5

Soil Class	Fixed base (cm)	Spring Model (cm)	Code method(cm)
B	8.373	25.387	18.957
C	15.478	32.540	26.558
D	22.263	39.552	33.816

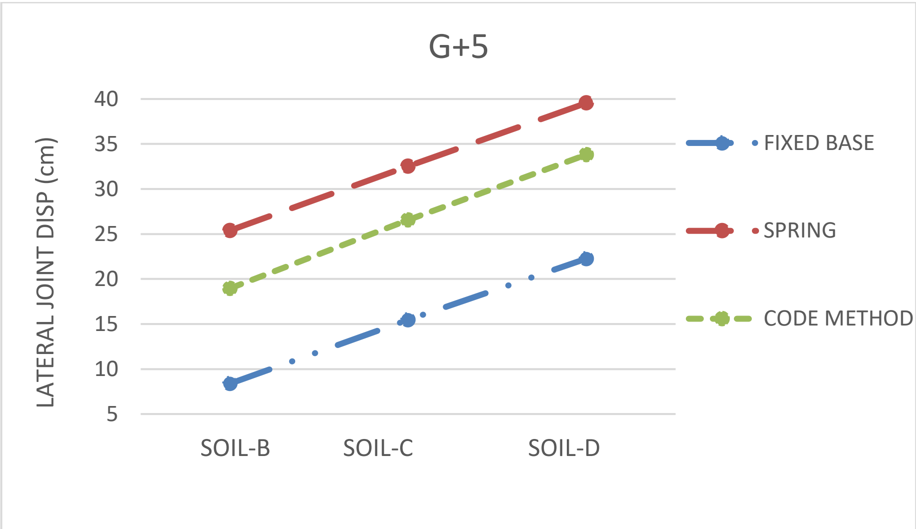


Figure 4-15: Lateral joint displacement at roof level of G+5

The above graph notification gives insight that both the code model analysis and spring model method have significant maximum lateral deflection at roof level in ideal soil class B, C and D.

4.2.2 Structure system -II (G+10)

The period of each structural model analysis is done as per 4.21. here below we only put the results in tabulation.

Period (T)	Mode-1	Mode-2	Mode-3	Mode -4	Mode-5	Mode-6	Mode-7	Mode-8	Mode -9
	1.209	1.186	0.896	0.385	0.373	0.336	0.310	0.291	0.267

4.2.2.1 Code method analysis

The results calculation is as described under 4.2.1

Table 4-15: Imposed & self-weight load on the system with the modified period

G+10			LOAD (KN)		
	Area (m^2)	No of floor	Imposed	Self-weight	Total
Roof floor	600	1	1800	2250	4050
Floor (1^{st}-10^{th})	600	10	30000	24000	54000
Shear wall	75	12		6750	6750
Ground		1		624	624
				TOTAL	65424

	Sub soil class		
	B	C	D
ϒ (KN/m^3)	16	16	16
ν	0.4	0.4	0.4
Vs (m/s)	600	350	150
G (KN/m^2)	587155.9633	199796.1264	36697.24771
R_h (m)	13.82	13.82	13.82
R_θ (m)	12.63	12.63	12.63
α_h	1	1	1
R_m/V_sT	0.017	0.03	0.070
α_θ	1	1	0.97
K_y (N/m)	41863857.57	14391816.45	2656846.739
K_θ (N.m)	5757047850	2015654753	364162935.9
$\overline{K}$ (KN/m)	784740.1339	784740.1339	784740.1339
$\overline{T}/T$	1.011	1.031	1.160

Table 4-16: Effective damping coefficient of the structure model

	Soil class	$\overline{T}/T$	βo	β (%)
G+10	B	1.010	0.10	4.940
	C	1.029	0.20	4.759
	D	1.148	2.00	5.204

4.2.2.2 *Spring stiffness Analysis*

Table 4-17: Corrected equivalent spring stiffness

	Soil class		
Stiffness	B	C	D
$K_{eff\,(x)}$ (N/m)	17456679.43	6699768.41	1371258.52
$K_{eff\,(y)}$ (N/m)	17456679.43	6699768.41	1371258.52
$K_{eff\,(Z)}$ (N/m)	12818230.84	4881929.78	1021314.99
$K_{eff\,(xx)}$ (N.m)	83567490.89	41775772	11535509.63
$K_{eff\,(yy)}$ (N.m)	99087669.62	48945509.2	13517948.18
$K_{eff\,(zz)}$ (N.m)	131712909.7	66278543.7	18339137.28

Using the corrected spring constant as a constraint for the structure we have another modal analysis result depending on the ideal soil type

Table 4-18: Modal analysis by the spring method

Period (T)	Modal analysis of soil type B								
	Mode-1	Mode-2	Mode-3	Mode -4	Mode-5	Mode-6	Mode-7	Mode-8	Mode -9
	1.332	1.301	0.923	0.400	0.390	0.339	0.314	0.296	0.273
	Modal analysis of soil type C								
	Mode-1	Mode-2	Mode-3	Mode -4	Mode-5	Mode-6	Mode-7	Mode-8	Mode -9
	1.412	1.371	1.000	0.407	0.397	0.340	0.315	0.298	0.276
	Modal analysis of soil type D								
	Mode-1	Mode-2	Mode-3	Mode -4	Mode-5	Mode-6	Mode-7	Mode-8	Mode -9
	1.635	1.551	1.180	0.424	0.412	0.345	0.319	0.304	0.281

And generally taking the first modal result as fundamental period of the structure the change in period is tabulated below.

Table 4-19: Period lengthening ratio of spring model

	Sub soil class		
$\overline{T}/T$	B	C	D
	1.101	1.167	1.352

As per discussed modeling analysis in the previous sections here are the joint displacement result for the G+10 (frame and shear wall) structural system. Here below is max possible joint displacement by EQ-Y spectrum in cm unit.

Table 4-20: Lateral joint displacement at roof level of G+10

Soil Class	Fixed base (cm)	Spring Model (cm)	Code method(cm)
B	18.705	38.289	30.801
C	20.777	41.627	33.104
D	29.890	60.075	43.239

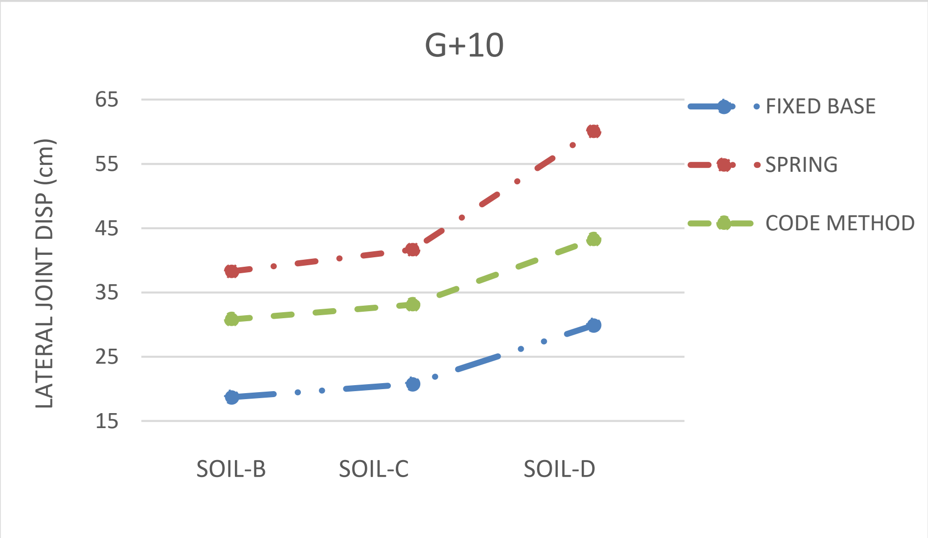


Figure 4-16: Lateral joint displacement at roof level of G+10

4.2.3 Structure system -III (G+15)

The period of each structural model analysis is done as per 4.2.1. here below we only put the results in tabulation.

Period (T)	Mode-1	Mode-2	Mode-3	Mode -4	Mode-5	Mode-6	Mode-7	Mode-8	Mode -9
	1.478	1.394	0.942	0.408	0.399	0.268	0.215	0.215	0.178

4.2.3.1 Code method analysis

Table 4-21: Imposed & self-weight load on the system with the modified period

G+15			LOAD (KN)		
	Area (m^2)	No of floor	Imposed	Self-weight	Total
Roof floor	600	1	1200	2250	3450
Floor (1^{st}-15^{th})	600	15	45000	36000	81000
Shear wall	100	17		12750	12750
Ground		1		624	624
				TOTAL	97824

	Sub soil class		
	B	C	D
ϒ (KN/m^3)	16	16	16
ν	0.4	0.4	0.4
Vs (m/s)	600	350	150
G (KN/m^2)	587155.9633	199796.126	36697.2477
R_h (m)	13.82	13.82	13.82
R_θ (m)	12.63	12.63	12.63
α_h	1	1	1
R_m/V_sT	0.02	0.034	0.057
α_θ	1	1	0.985
K_y (N/m)	41863857.57	14391816.45	2670298.619
K_θ (N.m)	5757047850	2015654753	376822822
$\overline{K}$ (KN/m)	891829.7969	891829.797	891829.797
$\overline{T}/T$	1.013	1.038	1.189

Table 4-22: Effective damping coefficient of the structure model

	Soil class	$\overline{T}/T$	βo	β (%)
G+15	B	1.013	0.07	4.878
	C	1.038	0.10	4.575
	D	1.189	0.75	3.725

4.2.3.2 Spring stiffness Analysis

Table 4-23: Corrected equivalent spring stiffness

	Soil class		
Stiffness	B	C	D
$K_{eff\,(x)}$ (N/m)	16743773.14	6456491.751	1345755.067
$K_{eff\,(y)}$ (N/m)	16743773.14	6456491.751	1345755.067
$K_{eff\,(Z)}$ (N/m)	12103163	4637556.543	978804.4326
$K_{eff\,(xx)}$ (N.m)	68953846.6	35320916.81	10397660.7
$K_{eff\,(yy)}$ (N.m)	81532518.46	41220327.37	12020939.1
$K_{eff\,(zz)}$ (N.m)	108859175.1	56000429.93	16538307.6

Using the corrected spring constant as a constraint for the structure we have another modal analysis result depending on the ideal soil type

Table 4-24: Modal analysis by the spring method

Period (T)	Modal analysis of soil type B								
	Mode-1	Mode-2	Mode-3	Mode -4	Mode-5	Mode-6	Mode-7	Mode-8	Mode -9
	1.713	1.595	1.064	0.470	0.455	0.303	0.240	0.239	0.186
	Modal analysis of soil type C								
	Mode-1	Mode-2	Mode-3	Mode -4	Mode-5	Mode-6	Mode-7	Mode-8	Mode -9
	1.814	1.679	1.128	0.485	0.470	0.337	0.255	0.254	0.192
	Modal analysis of soil type D								
	Mode-1	Mode-2	Mode-3	Mode -4	Mode-5	Mode-6	Mode-7	Mode-8	Mode -9
	2.08	2.044	1.29	0.515	0.504	0.337	0.255	0.254	0.192

And generally taking the first modal result as fundamental period of the structure the the change in period is tabulated below.

Table 4-25: Period lengthening ratio of spring model

	Sub soil class		
$\overline{T}/T$	B	C	D
	1.159	1.227	1.407

As per discussed modeling analysis in the previous sections here are the joint displacement result for the G+15 (frame and shear wall) structural system. Here below is max possible joint displacement by EQ-Y spectrum in cm unit.

Table 4-26: Lateral joint displacement at roof level of G+15

Soil Class	Fixed base (cm)	Spring Model (cm)	Code method(cm)
B	23.106	44.79	35.695
C	25.747	49.933	38.327
D	35.892	55.921	49.913

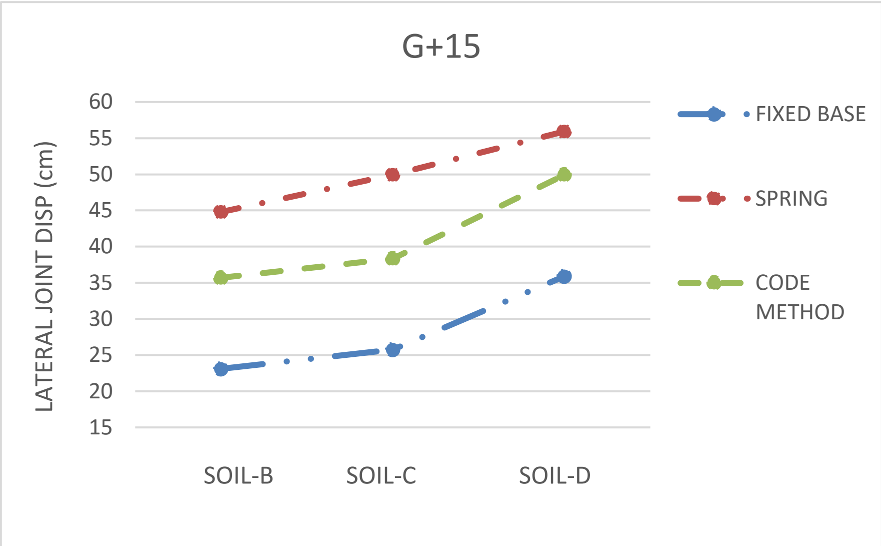


Figure 4-17: Lateral joint displacement at roof level of G+15

4.2.4 Structure system -IV (G+20)

The period of each structural model analysis is done as per 4.2.1. here below we only put the results in tabulation.

Period (T)	Mode-1	Mode-2	Mode-3	Mode -4	Mode-5	Mode-6	Mode-7	Mode-8	Mode -9
	1.881	1.787	1.305	0.514	0.503	0.361	0.252	0.251	0.194

4.2.4.1 Code method analysis

Table 4-27: Imposed & self-weight load on the system with the modified period

G+20			LOAD (KN)		
	Area (m^2)	No of floor	Imposed	Self-weight	Total
Roof floor	600	1	1800	2160	3960
Floor (1^{st}-20^{th})	600	20	60000	46080	106080
Shear wall	125	22		20625	20625
Ground		1		624	624
				TOTAL	131289

	Sub soil class		
	B	C	D
ϒ (KN/m^3)	16	16	16
ν	0.4	0.4	0.4
Vs (m/s)	600	350	150
G (KN/m^2)	587155.963	199796.126	36697.2477
R_h (m)	13.82	13.82	13.82
R_θ (m)	12.63	12.63	12.63
α_h	1	1	1
R_m/V_sT	0.011	0.019	0.045
α_θ	1	1	1
K_y (N/m)	42724777.91	14684768.51	2737558.02
K_θ (N.m)	6090065803	2128973362	406646140.2
$\overline{K}$ (KN/m)	995006.5662	995006.5662	995006.5662
$\overline{T}/T$	1.015	1.043	1.213

Table 4-28: Effective damping coefficient of the structure model

	Soil class	$\overline{T}/T$	βo	β (%)
G+20	B	1.015	0.06	4.840
	C	1.043	0.1	4.504
	D	1.213	0.7	3.501

4.2.4.2 Spring stiffness Analysis

Table 4-29: Corrected equivalent spring stiffness

	Soil class		
Stiffness	B	C	D
$K_{eff\,(x)}$ (N/m)	21725853.33	8063028.58	1676265.23
$K_{eff\,(y)}$ (N/m)	21725853.33	8063028.58	1676265.23
$K_{eff\,(Z)}$ (N/m)	15761289.11	5913526.07	1246692.79
$K_{eff\,(xx)}$ (N.m)	169093694.1	82612436.7	24333931.8
$K_{eff\,(yy)}$ (N.m)	195696259.9	94580770.6	27517281.6
$K_{eff\,(zz)}$ (N.m)	268642559.8	131621336	38836311

Using the corrected spring constant as a constraint for the structure we have another modal analysis result depending on the ideal soil type

Table 4-30: Modal analysis by the spring method

Period (T)	Modal analysis of soil type B								
	Mode-1	Mode-2	Mode-3	Mode -4	Mode-5	Mode-6	Mode-7	Mode-8	Mode -9
	2.055	1.951	1.420	0.548	0.536	0.388	0.264	0.264	0.201
	Modal analysis of soil type C								
	Mode-1	Mode-2	Mode-3	Mode -4	Mode-5	Mode-6	Mode-7	Mode-8	Mode -9
	2.182	2.080	1.497	0.563	0.552	0.401	0.269	0.269	0.205
	Modal analysis of soil type D								
	Mode-1	Mode-2	Mode-3	Mode -4	Mode-5	Mode-6	Mode-7	Mode-8	Mode -9
	2.551	2.434	1.627	0.596	0.570	0.419	0.281	0.276	0.212

And generally taking the first modal result as fundamental period of the structure the change in period is tabulated below.

Table 4-31: Period lengthening ratio of spring model

	Sub soil class		
$\overline{T}/T$	B	C	D
	1.159	1.227	1.407

As per discussed modeling analysis in the previous sections here are the joint displacement result for the G+20 (frame and shear wall) structural system. Here below is max possible joint displacement by EQ-Y spectrum in cm unit.

Table 4-32: Lateral joint displacement at roof level of G+20

Soil Class	Fixed base (cm)	Spring Model (cm)	Code method(cm)
B	33.164	54.026	46.88
C	35.853	59.496	49.870
D	47.657	72.381	62.996

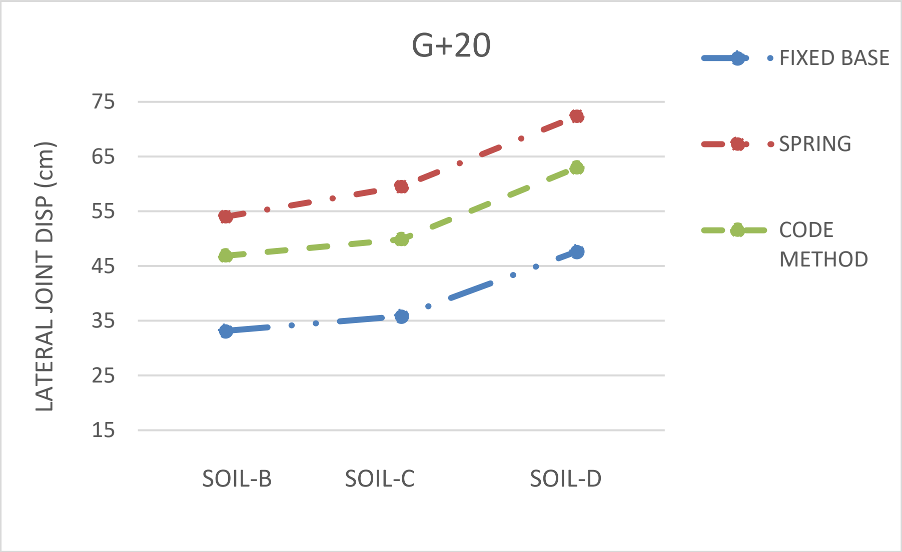


Figure 4-18: Lateral joint displacement at roof level of G+20

4.2.5 Structure system -V (G+25)

The period of each structural model analysis is done as per 4.2.1. here below we only put the results in tabulation.

Period (T)	Mode-1	Mode-2	Mode-3	Mode -4	Mode-5	Mode-6	Mode-7	Mode-8	Mode -9
	2.496	2.354	1.740	0.682	0.663	0.480	0.322	0.320	0.233

4.2.5.1 Code method analysis

Table 4-33: Imposed & self-weight load on the system with the modified period

G+20			LOAD (KN)		
	Area (m^2)	No of floor	Imposed	Self-weight	Total
Roof floor	600	1	1800	2250	4050
Floor (1^{st}-25^{th})	600	25	90000	60000	150000
Shear wall	125	27		25312.5	25312.5
Ground		1		624	624
				TOTAL	179986.5

	Sub soil class		
	B	C	D
ϒ (KN/m^3)	16	16	16
ν	0.4	0.4	0.4
Vs (m/s)	600	350	150
G (KN/m^2)	587155.963	199796.126	36697.2477
R_h (m)	13.82	13.82	13.82
R_θ (m)	12.63	12.63	12.63
α_h	1	1	1
R_m/V_sT	0.008	0.014	0.034
α_θ	1	1	1
K_y (N/m)	43155238.07	14904482.56	2804817.422
K_θ (N.m)	6256574779	2213962319	432663167.7
$\overline{K}$ (KN/m)	1382106.413	1382106.413	1382106.413
$\overline{T}/T$	1.022	1.062	1.291

Table 4-34: Effective damping coefficient of the structure model

	Soil class	$\overline{T}/T$	βo	β (%)
G+25	B	1.022	0.05	4.737
	C	1.062	0.10	4.280
	D	1.291	0.60	2.923

4.2.5.2 Spring stiffness Analysis

Table 4-35: Corrected equivalent spring stiffness

	Soil class		
Stiffness	B	C	D
$K_{eff\,(x)}$ (N/m)	24718366.96	9229306.96	1924794.21
$K_{eff\,(y)}$ (N/m)	24718366.96	9229306.96	1924794.21
$K_{eff\,(Z)}$ (N/m)	18199748.16	6893439.756	1448287.81
$K_{eff\,(xx)}$ (N.m)	271437627.9	138876555	40937474.3
$K_{eff\,(yy)}$ (N.m)	312590286.2	157376015.5	45232455.8
$K_{eff\,(zz)}$ (N.m)	432170247.8	221880142	65190952.2

Using the corrected spring constant as a constraint for the structure we have another modal analysis result depending on the ideal soil type

Table 4-36: Modal analysis by the spring method

Period (T)	Modal analysis of soil type B								
	Mode-1	Mode-2	Mode-3	Mode -4	Mode-5	Mode-6	Mode-7	Mode-8	Mode -9
	2.699	2.522	1.874	0.723	0.703	0.513	0.339	0.337	0.246
	Modal analysis of soil type C								
	Mode-1	Mode-2	Mode-3	Mode -4	Mode-5	Mode-6	Mode-7	Mode-8	Mode -9
	2.843	2.709	1.959	0.739	0.720	0.528	0.345	0.344	0.251
	Modal analysis of soil type D								
	Mode-1	Mode-2	Mode-3	Mode -4	Mode-5	Mode-6	Mode-7	Mode-8	Mode -9
	3.326	3.287	2.181	0.776	0.760	0.588	0.359	0.358	0.263

And generally taking the first modal result as fundamental period of the structure the change in period is tabulated below.

Table 4-37: Period lengthening ratio of spring model

	Sub soil class		
$\bar{T}/T$	B	C	D
	1.081	1.140	1.332

As per discussed modeling analysis in the previous sections here are the joint displacement result for the G+25 (frame and shear wall) structural system. Here below is max possible joint displacement by EQ-Y spectrum in cm unit.

Table 4-38: Lateral joint displacment at roof level of G+25

Soil Class	Fixed base (cm)	Spring Model (cm)	Code method(cm)
B	45.598	65.787	60.707
C	47.284	69.102	60.582
D	54.721	102.345	70.851

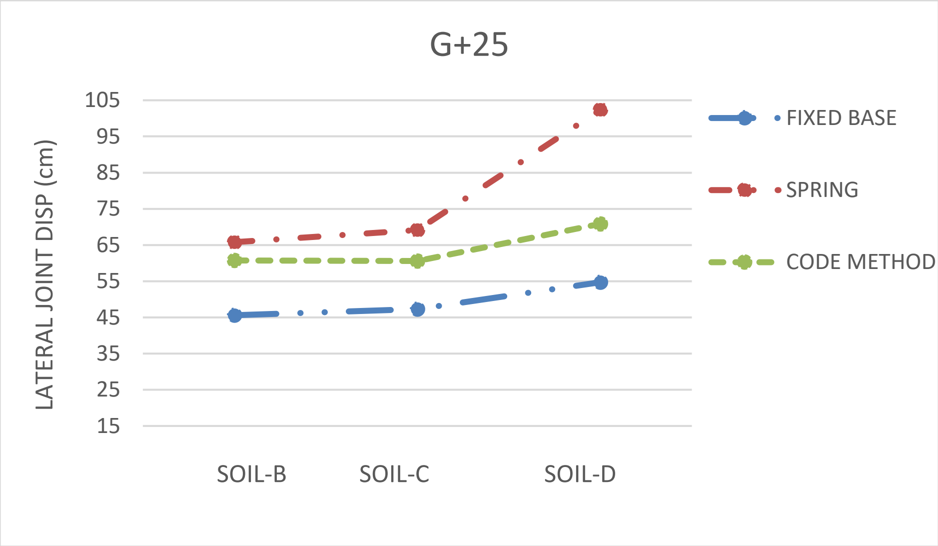


Figure 4-19: Lateral joint displacement at roof level of G+25

CHAPTER 5 DISCUSSION

As mentioned in previous section before, we use the structural analysis software ETABS in modeling and analysis of all the structural samples of this thesis paper. The results are put in perspective of the fundamental period of the structure and their maximum lateral displacement value on the two-approach analysis, which is the spring model and code method analysis.

In the first SSI consideration analysis depending on the modified natural fundamental period and damping ratio, we take the possible maximum displacement on critical roof and floor level and as of second analysis method, on the spring system model we also find the modified fundamental period and critical lateral displacement to see whether taking the consideration of SSI have effect on max lateral deflection of the structure.

5.1 Fundamental period (Code method)

In consideration of the code modification analysis depending on the soil ideal sub class we have different fundamental period based on the structural type of model taken.

Table 5-39: Modified fundamental period value (code method)

structural system	Period (T)	Modified period $\bar{T}$		
		B	C	D
Structural system I (G+5)	1.054	1.057	1.062	1.100
Structural system III (G+10)	1.209	1.222	1.246	1.402
Structural system IV (G+15)	1.478	1.497	1.534	1.757
Structural system V (G+20)	1.881	1.909	1.962	2.282
Structural system VI (G+25)	2.496	2.550	2.650	3.223

The comparison between the results of each model fundamental period and its modification (in consideration of SSI by the code method) depending on the ideal soil type by the reference of the code that is already described for each structure model are shown in detail in figure 5-20.

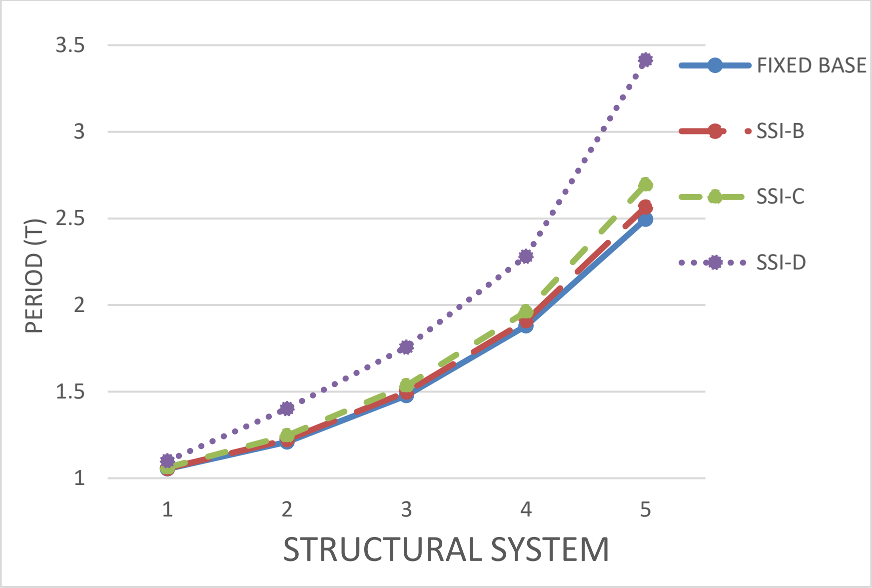


Figure 5-20: Fundamental period for fixed base Vs SSI consideration (code method)

Depending on the ideal sub soil class we have different period lengthening ratio based on the type of our model structures.

Table 5-40: Period lengthening ratio per ideal soil type (code method)

	$\overline{T}/T$		
structural system	B	C	D
Structural system I (G+5)	1.003	1.008	1.044
Structural system III (G+10)	1.011	1.031	1.16
Structural system IV (G+15)	1.013	1.038	1.189
Structural system V (G+20)	1.015	1.043	1.213
Structural system VI (G+25)	1.022	1.062	1.291

5.2 Fundamental period (spring model method)

Here we have the changed fundamental period of the structural models considered based on the ideal sub soil classes by the spring model analysis approach.

Table 5-41: Modified fundamental period value (spring model method)

structural system	Period (T)	Modified period $\bar{T}$		
		B	C	D
Structural system I (G+5)	1.054	1.056	1.058	1.065
Structural system III (G+10)	1.209	1.241	1.274	1.373
Structural system IV (G+15)	1.478	1.713	1.814	2.08
Structural system V (G+20)	1.881	2.055	2.182	2.551
Structural system VI (G+25)	2.496	2.699	2.843	3.326

Here below the figure shows the comparison between the results of each model fundamental period and its modification (in consideration of SSI by the spring model method) depending on the ideal sub soil type.

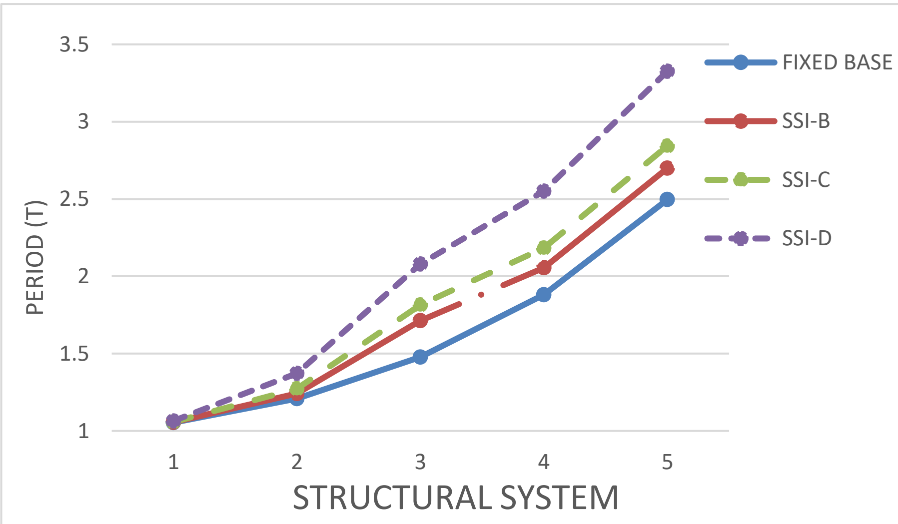


Figure 5-21: Fundamental period for fixed base Vs SSI consideration (spring model method)

Depending on the soil ideal class, we will have different period lengthening ratio based on the structural type of our structure model for the spring model analysis approach.

Table 5-42: Period lengthening ratio per ideal soil type (spring model method)

	$\overline{T}/T$		
structural system	B	C	D
Structural system I (G+5)	1.002	1.004	1.010
Structural system III (G+10)	1.101	1.167	1.352
Structural system IV (G+15)	1.159	1.227	1.407
Structural system V (G+20)	1.093	1.160	1.360
Structural system VI (G+25)	1.081	1.140	1.332

5.3 Minimum free gap distance

As per code provision, all structures shall be separated from adjoining structures, separations shall allow for the displacement δ_M. Adjacent buildings on the same property shall be separated by at least, δ_{MT},

Where $$\delta_{MT} = \sqrt{(\delta_{M1})^2 + (\delta_{M2})^2}$$

according to our new code δ_{M1} and δ_{M2} are the displacements of the adjacent buildings. Taking this code provision, we will analyze the results for minimum gap distance according to the soil profile assumed for each model structure

Sample group is taken from the model structures to show the minimum gap distance that must be achieved in the system to prevent collision between closely constructed structure on fixed base and in consideration SSI, both method, at significant floor level that are exposed to collision.

5.3.1 Group-A (G+5 and G+10)

The gap distance that must be achieved is based on the maximum lateral deflection and critical collision point at roof level of the structural system I (G+5) and the 6th floor of the structural system II (G+10). The tables below show the minimum gap distance that must be achieved based on consideration of the ideal sub soil class B, C and D.

Table 5-43: Minimum gap distance expected Group -A

Soil class B	Lateral displacement (cm)		Expected free gap distance (cm)
	G+5 @ roof	G+10 @ 6th floor	
Fixed base	8.373	8.592	12.00
Code method	18.957	20.761	28.11
Spring Model	25.387	26.554	36.74

Soil class C			
	G+5 @ roof	G+10 @ 6th floor	
Fixed base	15.478	9.543	18.18
Code method	26.558	20.612	33.62
Spring Model	32.54	30.255	44.43

Soil class D			
	G+5 @ roof	G+10 @ 6th floor	
Fixed base	22.263	13.729	26.16
Code method	33.816	25.267	42.21
Spring Model	39.552	39.795	56.11

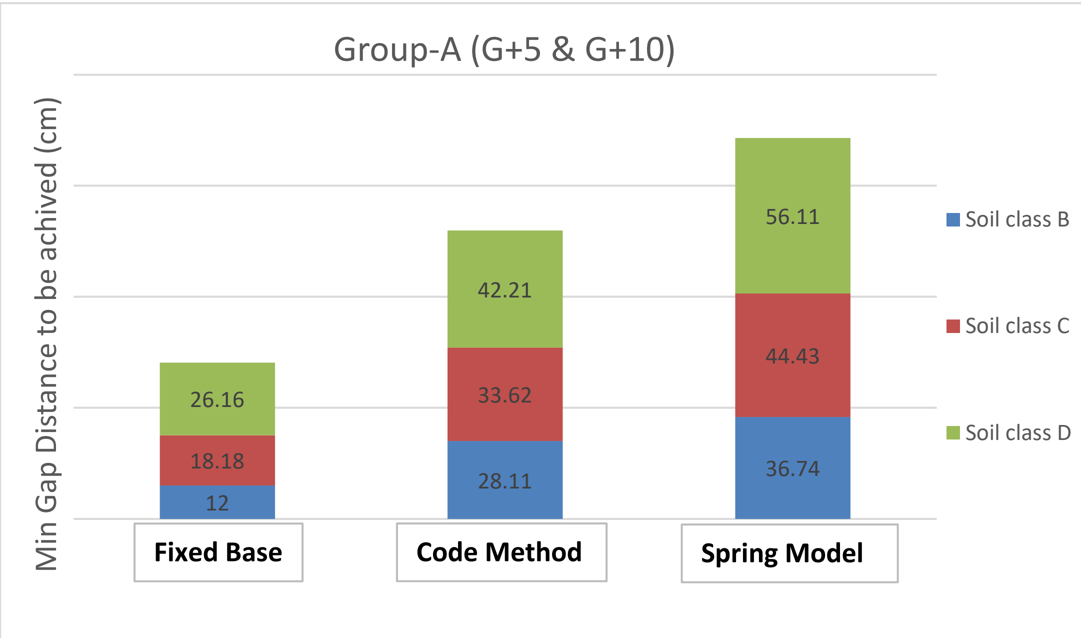


Figure 5-22: Relative minimum gap distance expected in Group -A

5.3.2 Group-B (G+5 and G+15)

The system of analysis is the same as 5.3.1 but here we see the results between G+5 and G+15 expected free gap distance at the critical level of the structure.

Table 5-44: Minimum gap distance expected Group -B

Soil class B	Lateral displacement (cm)		Expected free gap distance (cm)
	G+5 roof	G+15 6th floor	
Fixed base	8.373	7.097	10.98
Code method	18.957	17.892	26.07
Spring Model	25.387	27.084	37.12

Soil class C			
	G+5 roof	G+15 6th floor	
Fixed base	15.478	7.827	17.34
Code method	26.558	18.704	32.48
Spring Model	32.54	29.554	43.96

Soil class D			
	G+5 roof	G+15 6th floor	
Fixed base	22.263	11.055	24.86
Code method	33.816	22.293	40.50
Spring Model	39.552	32.325	51.08

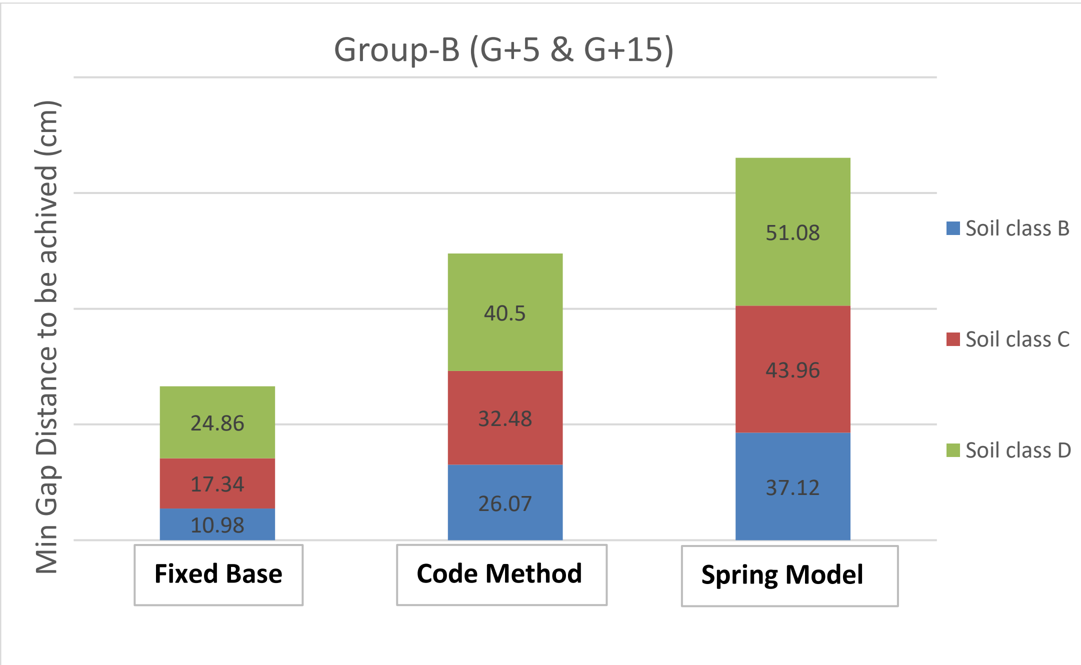


Figure 5-23: Relative minimum gap distance expected in Group -B

5.3.3 Group-C (G+5 and G+20)

The system of analysis is the same as 5.3.1 but here we see the results between G+5 and G+20 expected free gap distance at the critical level of the structure.

Table 5-45: Minimum gap distance expected Group -C

Soil class B	Lateral displacement (cm)		Expected free gap distance (cm)
	G+5 roof	G+20 6th floor	
Fixed base	8.373	7.082	10.97
Code method	18.957	17.875	26.06
Spring Model	25.387	26.438	36.65

Soil class C			
	G+5 roof	G+20 6th floor	
Fixed base	15.478	7.671	17.27
Code method	26.558	18.53	32.38
Spring Model	32.54	28.402	43.19

Soil class D			
	G+5 roof	G+20 6th floor	
Fixed base	22.263	10.243	24.51
Code method	33.816	21.417	40.03
Spring Model	39.552	31.399	50.50

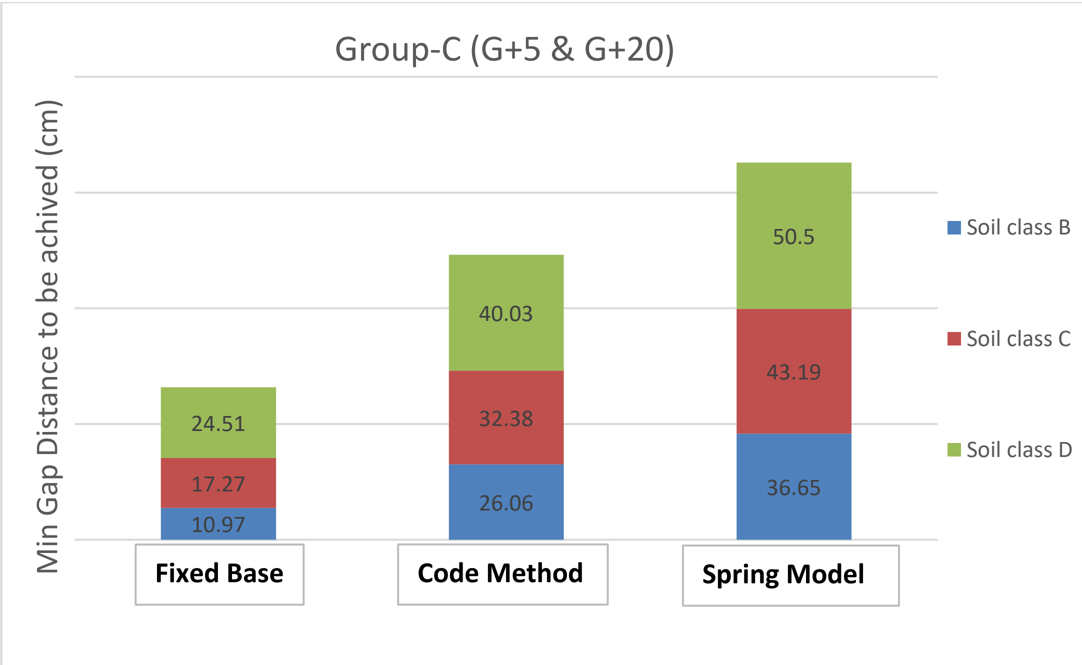


Figure 5-24: Relative minimum gap distance expected in Group -C

5.3.4 Group-D (G+10 and G+15)

The system of analysis is the same as 5.3.1 but here we see the results between G+10 and G+15 expected free gap distance at the critical level of the structure.

Table 5-46: Minimum gap distance expected Group -D

Soil class B	Lateral displacement (cm)		Expected free gap distance (cm)
	G+10 @ roof	G+15 @ 11th floor	
Fixed base	18.705	15.15	24.07
Code method	30.801	26.847	40.86
Spring Model	38.289	36.255	52.73

Soil class C			
	G+10 @ roof	G+15 @ 11th floor	
Fixed base	18.705	16.7	25.08
Code method	30.801	28.571	42.01
Spring Model	38.289	40.076	55.43

Soil class D			
	G+10 @ roof	G+15 @ 11th floor	
Fixed base	18.705	23.522	30.05
Code method	30.801	36.157	47.50
Spring Model	38.289	44.16	58.45

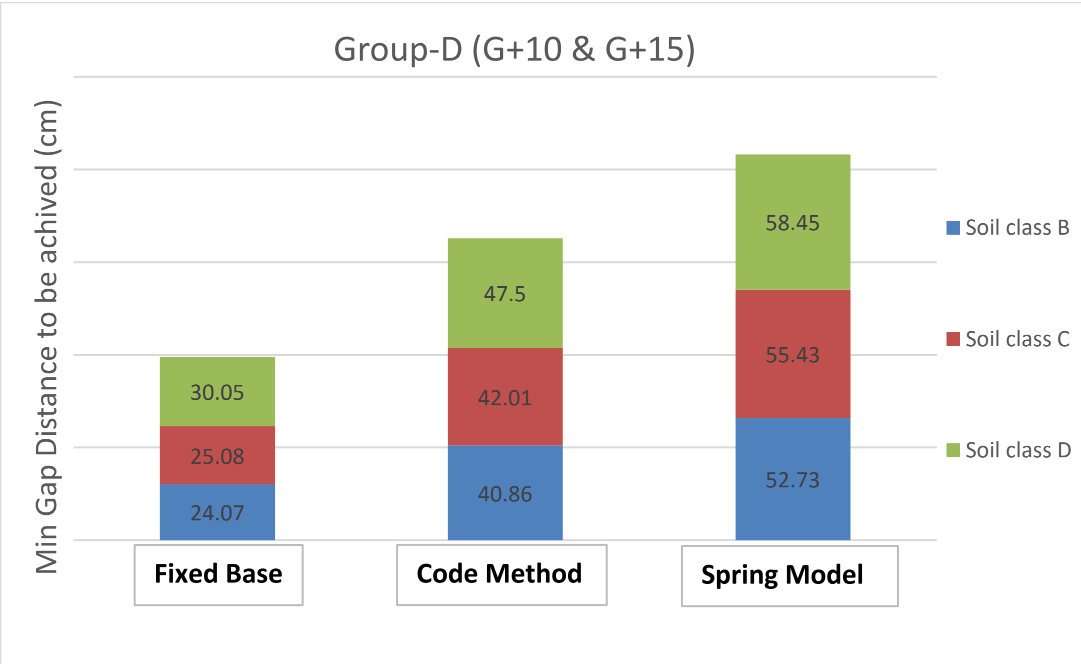


Figure 5-25: Relative minimum gap distance expected in Group -D

5.3.5 Group-E (G+10 and G+20)

The system of analysis is the same as 5.3.1 but here we see the results between G+10 and G+20 expected free gap distance at the critical level of the structure.

Table 5-47: Minimum gap distance expected Group -E

Soil class B	Lateral displacement (cm)		Expected free gap distance (cm)
	G+10 @ roof	G+20 @ 11th floor	
Fixed base	18.705	16.426	24.89
Code method	30.801	28.267	41.81
Spring Model	38.289	36.477	52.88
Soil class C			
	G+10 @ roof	G+20 @ 11th floor	
Fixed base	18.705	17.761	25.79
Code method	30.801	29.751	42.82
Spring Model	38.289	39.689	55.15
Soil class D			
	G+10 @ roof	G+20 @ 11th floor	
Fixed base	18.705	23.364	29.93
Code method	30.801	36.281	47.59
Spring Model	38.289	45.228	59.26

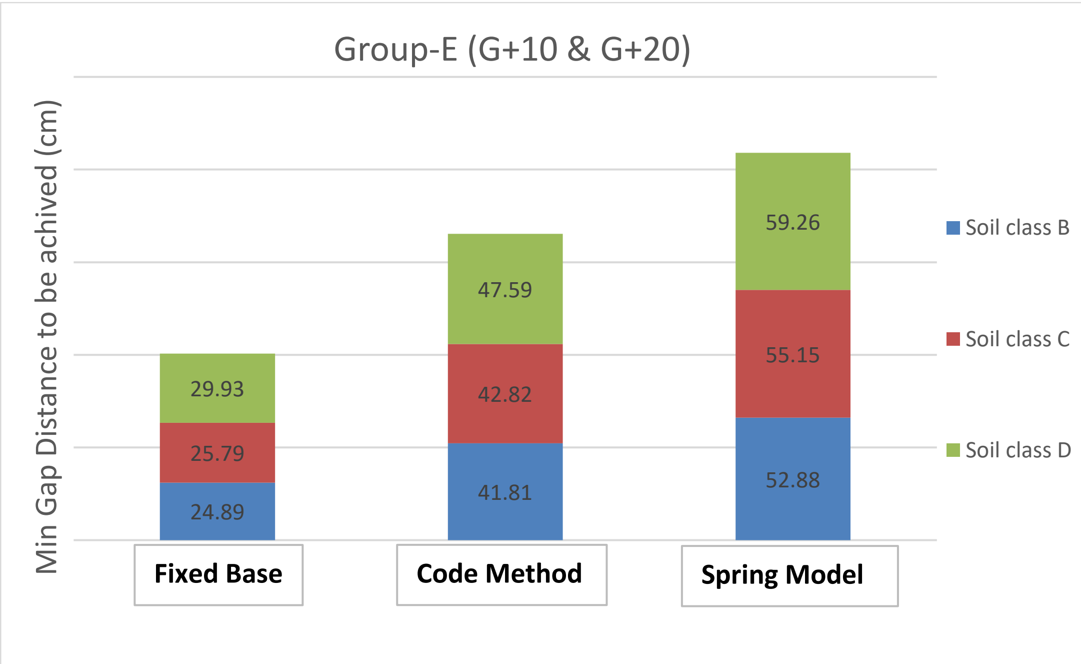


Figure 5-26: Relative minimum gap distance expected in Group -E

5.3.6 Group-F (G+10 and G+25)

The system of analysis is the same as 5.3.1 but here we see the results between G+10 and G+25 expected free gap distance at the critical level of the structure.

Table 5-48: Minimum gap distance expected Group -F

Soil class B	Lateral displacement (cm)		Expected free gap distance (cm)
	G+10 @ roof	G+25 @ 11th floor	
Fixed base	18.705	17.737	25.78
Code method	30.801	29.725	42.81
Spring Model	38.289	37.521	53.61

Soil class C			
	G+10 @ roof	G+25 @ 11th floor	
Fixed base	18.705	18.248	26.13
Code method	30.801	29.713	42.80
Spring Model	38.289	39.271	54.85

Soil class D			
	G+10 @ roof	G+25 @ 11th floor	
Fixed base	18.705	21.498	28.50
Code method	30.801	33.906	45.81
Spring Model	38.289	53.214	65.56

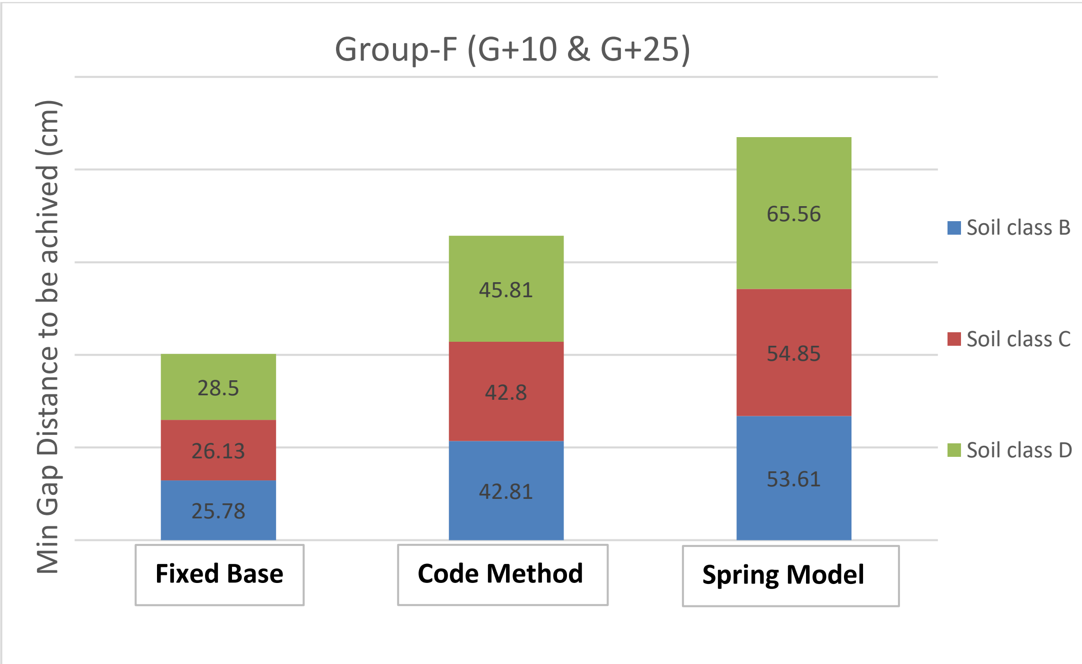


Figure 5-27: Relative minimum gap distance expected in Group -F

CHAPTER 6 CONCLUSION AND RECOMMENDATION

6.1 Conclusion

In order to account for the SSI effect in pounding the paper uses two approaches code method and spring model analysis. The first method mainly relays on changing the dynamic property of the structure to be analyzed and the later tries to change the routine fixed restraint base of the structure by 6 degree of freedom spring with coined stiffness value in consideration of foundation size, embedment and soil property.

Depending on the two approaches the paper investigate the minimum free pounding gap distance with and without consideration of the soil structure interaction, and based on the analysis of the paper these conclusions can be drawn:

The fundamental natural period of a structure considering soil structure interaction increases with the consideration of both approach code method (modification of T and β) and spring model method. In consideration of SSI by code method the change in the fundamental period value is not that much significant for the lower and medium height structures especially considering the sub soil class B and C, but for the sub soil class D and for high rise structures the result of the modified fundamental period is high. When we see the results from spring model method it also shows a steep increase change of fundamental period as the structure height increases and the ideal sub soil class flows from B to D simultaneously.

By the perspective of lateral joint displacement, the general result almost shows the same characteristics in consideration of the SSI, in both code and spring model method, it increases with the change in the height of the structure and the ideal sub soil class changes from B to C and D.

To avoid structural pounding by considering free pounding gap distance the analysis clearly shows that it must be a requirement to assess the gap distance between closely spaced, constructed or to be constructed structures. The assessment of the groups case, assumed to be constructed in closely spaced, shows the consideration of SSI is significant as it increases the requirement of the free gap distance and it goes suddenly high from sub soil class B to C and D and as the height of the structure increases. In general, it can be concluded that the influence of SSI on lateral displacement of a structure is seen to be a major course shift in achieving pounding free gap distance, at a time of seismic event, to prevent the loss of life and property damage that comes from the pounding action.

6.2 Recommendation

Lately structures are constructed closely, with small or no free gap distance, specially in the growing big cities like the capital Addis Ababa. This non-free gap distance construction will face danger if seismic events are inevitable.

These problems can be solved for the future by giving enough perspective on structural pounding problems for the two major stake holders. The first stake holder are the professionals that are involved from the design to the construction of the structures and the other will be the government body that they must control and implement every precaution when it comes to these structures that are to be constructed closely.

For further study the thesis can be continued by additional future considering the limitations. Therefore, one can extend this study on the following points

- this study mainly based on regular structures, that it can be extend by analyzing different irregular structure
- the paper mainly relays on consideration of ideal sub soil classes, that one can extend the scope of the study by taking actual soil lab test result.
- The paper only considers the lateral displacement effect of SSI on the effect of structural pounding, that one can extend the effect of this lateral displacement with the increased secondary design force associated P-delta effect.

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