Evaluation of Response Modification Factor in Consideration of Soil-Structure Interaction

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MASTER OF SCIENCE IN CIVIL ENGINEERING
(WITH SPECIALIZATION IN STRUCTURES)

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CERTIFICATE OF APPROVAL

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This is to certify that Miss. Farheen Kanwal has incorporated all observations, suggestions and comments made by the external evaluators as well as the internal examiners and thesis supervisor. The comments are attached as Annexure I with the thesis. The title of her Thesis is: Evaluation of Response Modification Factor in Consideration of Soil- Structure Interaction.

Dr. Munir Ahmed
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Declaration

I declare that this research is my original work and has been generated as a result of my own research. I have acknowledged the work of others wherever contributions are involved, every effort is made to indicate this clearly, with due reference to the literature and acknowledgement of collaborative research. The work has been carried out under the supervision of Associate professor Engr. Dr. Munir Ahmed at Capital University of Science and Technology, Islamabad, Pakistan.

Farheen Kanwal
(MCE153025)

Dedicated

TO MY MOTHER

Her endless support, encouragement, prayers and constant love has sustained me throughout my life.

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List of Abbreviations

R	Response Modification Factor
SSI	Soil Structure Interaction
RC	Reinforced Concrete
DoF	Degree of Freedom
SDoF	Single Degree of Freedom
MDoF	Multi Degree of Freedom
E-SDoF	Equivalent Single Degree of Freedom
T_{STR}	Period of Structure in Fixed Base
T_{FIX}	First Mode Period of Fixed Base
Δ_t	Target inter story Displacement Ductility Ratio
P	Pressure of Structure
k	Modulus of Subgrade reaction
w	Deflection
SD	Stiff Soil
SB	Rock Soil
A_b	Area of Foundation
I_{bx}	Moment of Inertia about x axis
I_{by}	Moment of Inertia about y axis
V_s	Shear Wave Velocity
G	Shear Modulus
V	Poison's Ratio
B and L	Half Width and Length of Foundation
FEM	Finite Element Modelling
ESA	Equivalent Static Analysis
PoA	Non Linear Push over Analysis
GWT	Ground Water Table

Acknowledgment

Every piece of work is rewarded according to the nature of devotion in it, Allah never spoils any effort. I am humbly thankful to most gracious, merciful and **almighty ALLAH** who gave me health, affectionate and talented teachers, helping friends and opportunities to complete my study.

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ABSTRACT

Since 1990s, great efforts have been made to replace the classical seismic design philosophy by the new ones. In both the equivalent linear static force and code based response spectrum procedure the seismic design philosophy is incorporated using response modification factor “R”. The ratio of elastic strength demand to inelastic strength design is defined as “R” factor. During earthquake, the response of the structure is influenced by three interlinked systems i.e., the super structure, the foundation and the soil medium. This phenomenon is called soil structure interaction (SSI). In conventional seismic design philosophy structural analysis is performed assuming that the structure is fixed at the foundation level (rigid support). However, in actual the structure has foundation flexibility depending upon the type of soil medium supporting the structure. Code based values of “R” factor does not reflect the SSI problem. Thus, there is a strong need to redefine the “R” factor values considering the effect of SSI. In this study, the seismic behaviour of mid-rise mix use buildings located in seismic zone 2B and soil type “S_D” and “S_B” in consideration with SSI (assuming no water ground water table has encountered) has been investigated. The main objectives of this research work is to evaluate “R” factor for moment resisting frame (MRF) buildings situated on different soils with shear wave velocity 300m/sec (S_D) and 1200m/s (S_B)considering SSI and to compare R-factor values with SSI system to that of fixed based system. A 10 storey MRF building with two basements + ground + 7 storeys has been designed by equivalent linear static force method using structural analysis software SAP 2000 v15.0.0. Code based value of “R” factor 5.5 for building frame system has been used. Soil medium is modelled using horizontal and vertical closely spaced, linear elastic springs which are identical but mutually independent. Nonlinear static pushover analysis using guidelines of FEMA 356 and ATC 40 has been performed. The modified “R” factor values are evaluated as ratio of elastic base shear strength from linear static analysis (without using code based “R” factor) to inelastic base shear from non-linear static pushover analysis. Different design parameters such as base shear, storey shear, storey drift and displacements have also been compared with SSI system to that of fixed base system. The storey shears and moments for the soil type S_D have increased whereas storey shears and moments for the soil type S_B remain same. For soil type S_D storey drifts and displacements with SSI

have negligible difference as compared to that of fixed base systems whereas for soil type S_B storey drifts and displacements remain same. It can be concluded that structures placed on soil medium with shear wave velocity greater than and equal to 1200m/sec are not affected by SSI whereas, structures placed on hard soils with shear wave velocity equal 300m/s are greatly affected by SSI.

List of intended publication

"Response modification factors in moment resisting frame buildings considering soil-structure interaction" (In progress)

CHAPTER 1

INTRODUCTION

1.1 Background

Structural response to earthquake ground shaking is dynamic. When the earthquake phenomenon occurs the super structure does not experience any applied forces. The lateral forces in the super-structure are created due to the dynamic motion of the soil on which structure is being supported. The structures are analysed against these stresses and strains. They are designed to withstand large inelastic displacement demands imposed by the earthquake. For economy reasons, the seismic design philosophy is based on allowing damage at specified locations in the structural elements such as at beams ends and bottom of the lowest story columns in moment resisting frame systems as shown in Figure 1.1a. In both equivalent linear static force and code based response spectrum procedure, this design philosophy is incorporated using response modification factor (IBC-2012; UBC-97; FEMA-451 & BCP-2007).

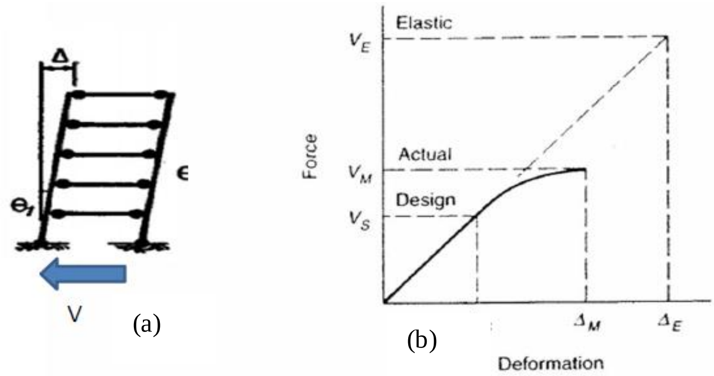


Figure 1.1Assumed Force- Displacement Relationship (Alan Williams, 2000)

The ratio of the structural elastic strength demand to inelastic design strength is defined as response modification factor or force reduction factor (R) (UBC-97& FEMA P-750). "R" factor accounts for over-strength, energy absorption (ductility) and system capacity to redistribute forces from inelastic high stressed regions to other less stressed regions within the structure. It shows the capability of the structure to dissipate energy through inelastic behaviour. This factor is unique and different for different types of structures and materials used (UBC-97; BCP-2007; IBC06/07& ASEC-7).

It has been well recognized that the soil medium, on which a structure is being supported interact dynamically with the super-structure during its response to ground shaking. The structural response to seismic movement is created due to the interaction of three inter-dependent systems: the superstructure, the foundation and the geologic media supporting the foundation (FEMA, 451). This phenomenon leads to the soil structure interaction (SSI). As a result of this SSI, the stresses and deflections in the system are modified significantly from the values which would have been developed, if it were resting on rigid strata.

During the past and recent earthquakes, it has been understood that the SSI impact plays an essential role in determining the response of building structures. A seismic SSI analysis determine the collective behaviour of the structure, the substructure and the soil medium under and surrounding the substructure to a specified seismic free field ground motion (Shehata et al., 2015).

The classical seismic analysis and design philosophy does not consider the flexibility of the substructure and surrounding soil medium. In current design practice, structure and foundation are analysed as separate systems and the superstructure is assigned fixed support at the bottom of column(Li et al., 2014; Veletsos & Prasad, 1989). This design approach assumed that building (structure) is resting on rigid support (fixed base condition). The design base shear calculated by following seismic design codes without SSI is usually less than the required lateral strength to keep the structure in the elastic range (Ghannad & Jahankhah, 2004).This design procedure is easy to handle but seismic performance of buildings with SSI may not be same as that of fixed base system when seismic waves propagate through the underlying soil, which may be harmful in case of actual structure to soil interaction (Mylonakis &Gazetas, 2000).

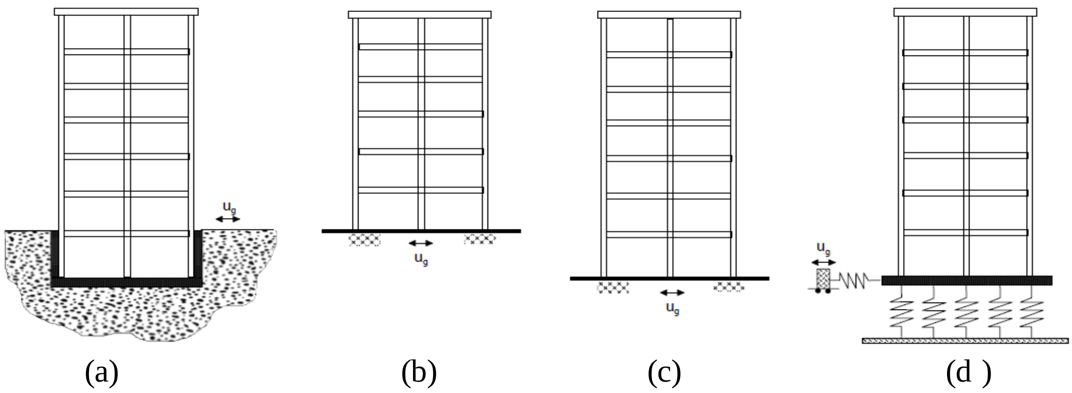


Figure 1.2 (a) Real system (b) Fixed base system at ground level (c) Fixed base at bottom of substructure (d) Flexible base system

Figure 1.2 shows the sketch of three structural conditions used in seismic design procedures. Veletsos & Newmark (1960) and Newmark & Hall (1973) are the pioneers in the work of response modification factor "R". The formulas formulated by them are functions of structural period and displacement ductility for acceleration, displacement and velocity sensitive regions. Values of the "*R*" factor with fixed base support for different structural systems used in equivalent linear static force procedure for seismic design have been well defined in building design codes (UBC-97; BCP-2007 & IBC 06/07).

1.2 Problem Statement

Values of "R" factor used in lateral force procedure are determined based on the fixed base system and are very high. These values of "R" factor are based on engineering judgement, performance of buildings in previous seismic ground motions and expected performance of buildings designed in the same way. The reason behind these high values of "R" factor is said to be nonlinear behaviour of earthquake resistant designed buildings. However, this design philosophy may not be true. After the decimation of recent earthquakes, the seismic SSI of multi-story structures has turned out to be vital. During the 1985 Mexico City earthquake, several buildings collapse due to site amplification (Romo & Seed, 1986). Seed (1986) studied the damage caused by 1985 Mexico City earthquake due to SSI. In case of ground motion, the movement of the base of the structural system will not be same as it is assumed to be in case of conventional fixed base system. The reason behind this difference is the coupling of the soil and structure interaction. So, the response of the structure will be modified when SSI is considered. The time-period of the structural system increases due to flexibility of structure at foundation level. Other parameters e.g. base shear, storey forces, storey drifts and displacements are also modified. Soil media also acts as damping system; it tends to normalize the general shaking impacts to an intensity that is lower than maximum value used in analytical methods (FEMA, 440). The kinematic effects due to soil structure interaction depend upon the configuration of the building and depth into the soil, i-e number of basement provision and time-period (FEMA, 356 & ATC, 440). It is true that SSI does complicate the analysis procedure but SSI can be damaging and

ignoring its effects can result unsafe design for building structures especially for structures placed on soft soil. (Khalil et al., 2007)
Code based values of the R-factor in lateral force procedures do not show SSI effects. Therefore, there is a dire need to redefine "R" factor values considering the SSI effects to ensure that these values reflect the structural ductility (NIST GCR 12-917-21).

1.3 Objective

The main objective of this study is to evaluate "R" factor for moment resisting frame (MRF) buildings considering SSI and to compare R-factor values with SSI system to that of fixed based system.

The overall objective of this research is to discuss the seismic behaviour of mid-rise buildings with and without consideration of SSI.

1.4 Research Methodology

Following tasks shall be performed to achieve the objectives of research work:

1. Code-based design of multistory (MRF) building shall be done (two Basements+ Ground +seven storey) with fixed base system for two soil types S_D& S_B.

2. Foundation shall be modeled as shell element under the superstructure and soil springs shall be assigned.

3. If the modal Participation is more than 75% contribution (first mode dominant) nonlinear push over analysis shall be performed.

4. Non-linear static pushover model shall be prepared for pushover analysis by assigning the hinges at beam-ends and column lower ends of bottom storey (Ground floor).

5. Non-linear static pushover analysis shall be performed for both SSI and fixed base systems.

6. Results of the both cases shall be interpreted to evaluate the actual "R" factor.

7. Comparison of storey shears, overturning moments, storey displacements, storey drift and plastic hinge rotation of both SSI and fixed base model for both S_D and S_B soil type shall be made.

Total eight numerical modals shall be prepared using structural analysis commercial software SAP-2000 as shown in table 1.1.

Table 1.1 Numerical models generated for this study

Sr. no.	Soil type	S_D(stiff soil)	S_B (Rock)
	Structural system	Building frame system	
01	Code based Design (lateral force procedure)	01	01
02	Fixed base modal (push over analysis)	01	01
03	Flexible base modal (push over analysis)	01	01
04	Flexible base modal (lateral force procedure)	01	01

1.5 Limitations of the Study

The limitations of this study are:

1. Only the numerical modelling, analysis and design have been done.
2. Only linear equivalent static (ESA) analysis and Static pushover analysis (non-linear) have been performed.
3. Only two types of soil S_D and S_B with shear wave velocity 300 m/s and 1200 m/s respectively, have been considered.
4. The effects of non-structural components have not been taken into account.

1.6 Organization of the thesis

<u>CAPTER 1:</u> INTRODUCTION: In this chapter, the research gap has been identified and outline of research methodology has been presented.

<u>CAPTER 2:</u> DYNAMICS OF SOIL STRUCTURE & LITERATURE REVIEW: This chapter presents detail literature review of soil structure interaction analysis and "R" factor evaluation with and without SSI leading to research gap.

<u>CAPTER3:</u> SOIL STRUCTURE INTERACTION MODELLING AND ANALYSIS METHODOLOGY: In this chapter, different modelling techniques for SSI have been discussed. The formulation of wrinkler's modal has been discussed. Calculation of soil springs for accurate modelling of soil has been obtained. Nonlinear static pushover analysis has been discussed.

<u>CAPTER4:</u> DESCRIPTION OF CASE STUDY BUILDING, RESULTS AND DISCUSSION: In this chapter, details of 10 storied MRF case study building, modelling and design methodology has been described. Values of "R" factor are calculated for different cases and compared. Different parameters i.e. story drift, time period; deflection and base shear have been compared.

<u>CAPTER5:</u> CONCLUSIONS AND RECOMMENDATIONS: This chapter summarizes the whole research work. Conclusions of the research work have been described. Future research needs have been presented.

CHAPTER 2

DYNAMICS OF SOIL STRUCTURE & LITERATURE REVIEW

2.1 Introduction

Soil Structure Interaction (SSI) is an arrangement of soil and structural dynamics, earthquake engineering, material science, computational and numerical methods. Since 1990s, extensive research work has been performed for replacing the conventional methods of seismic design by the modified procedures, which are established on the theory of performance-based earthquake design. Moreover, the analysis of built structures and defining precise methods for their strengthening has attracted the attention of researchers to consider SSI in their analysis.

The structural system is affected by seismic excitations, which is a function of three factors namely source characteristics, propagation paths of waves and local site effects. These factors result in a free field motion of the ground. The behaviour of the structure under the influence of free field ground motion is affected by SSI. In actual flexible foundation support causes acceleration within the structure. The seismic demand and capacity of structures is greatly affected by SSI (Hosseinzadeh & Nateghi, 2002). Moreover, SSI effects can increase the lateral displacements and corresponding storey drifts of building structures situated on soft soils. The increment of lateral displacements and storey drifts may change the behaviour of building structures (Tabatabaiefar, 2012). Hence, structural efficiency and safety cannot be assured without considering the effects of SSI especially for the structures resting on soft soils (Far, 2016). Therefore, to assess the accurate effects of inertial forces and lateral displacements in structures due to SSI effects, requires the consideration of the foundation flexibility (Stewart et al., 1999).

In addition, Veletsos and Meek (1974) elucidated that the SSI may have beneficial effects to the buildings under earthquake loading because of lengthening of time period, which increases the system's damping capacity. However, Gazetas and Mylonakis (1998) pointed out that in actual, due to natural flexibility of supporting soil medium, it allows some movements. The foundation flexibility increases the natural periods of the structural system due to reduced structural stiffness. This soil flexibility at the foundation level changes the response of structure (Stewart, 1999).

Indeed, case studies and post seismic observations shows that the SSI can be damaging and ignoring its effects could generate risky design for structures situated on soft soils (Kobayashi et al., 1986; Stewart &Seed, 1998).

Studies have shown that the dynamic response of structures supported on flexible base differ to a great extent from the structures supported by rigid base. A rigid base is a support of soil, which has infinite stiffness and foundation elements are not deformable. On the other hand, in flexible base analysis both the foundation elements and the soil are deformable (NIST GCR 12-917-21).

The scales of socio–economic damages caused by an earthquake depend largely on the characteristics of the seismic excitations. The Soil-Structure Interaction (SSI) problem has become an important feature of Structural Engineering due to massive constructions on different types of soils such as nuclear power plants, concrete and earth dams. Multi-storey buildings, bridges, tunnels and underground structures also require SSI consideration. Mylonakis and Gazetas (2000) identified three incidents of ground motions during Bucharest 1977, Mexico City 1985 and Kobe 1995 earthquake in which the seismic-induced response of structures increases due to SSI irrespective of a possible increase in damping. They described that Mexico earthquake was more significantly damaged the buildings with l0–12 storied situated on soft clay, and their period of vibration enhanced from 1 to 2 seconds because of foundation flexibility.

Studies have shown that the effects of soil structure interaction on the seismic response of structural systems are significant, particularly for MRF building, when the shear wave velocity (average shear wave velocity) of the supporting soil medium is less than 600 m/s (Agrawal and Hora, 2012; Far et al., 2011; Galal & Naimi, 2008; Gazetas & Mylonakis, 1998, Tabatabaiefar & Massumi, 2010; Wolf & Deeks, 2004). Literature review shows that SSI has the following effects on structural response:

(i) Lengthening of time natural time period and increase damping of the system.

(ii) Amplification of structural lateral displacements.

(iii) Modification of base shears which depending upon frequency seismic waves and dynamic characteristics of the soil and the structure.

Thus, ordinary building structures are the most susceptible to SSI effects therefore, there is necessity of a better insight into the physical phenomena that is occurring due to SSI (Wolf & Deeks, 2004).

Furthermore, NIST-GCR-12-917-21 gives the ratio of structure-to-soil as relative measuring parameter for determining when SSI response will be prominent.

Structure-to-soil $ratio = h/(VsT)$ Equation 2.1

Where, h is the $2/3_{rd}$ of building height,

Vs = shear wave velocity of soil

T = Time period with fixed-base building

When $h/(V_sT) > 0.1$, the building time period increases considerably and system damping is modified. The design base shear shall be modified (increase or decrease) depending upon the spectral shape. Similarly, the distribution of forces and displacements also modify relative to a fixed-base analysis.

When $h/(V_sT) < 0.1$, the distributions of shear forces and moments in SSI system are modified as compared to rigid base system depending upon the stiffness of super structure and soil media. Especially, the structures with core walls in which higher-mode responses are dominant and basements show more SSI effects. This ratio is not an exact principle but gives the approximate initial results.

2.2 Inertial and Kinematic interactions

The structure interacts with the foundation and the soil underlying, when it is subjected to seismic excitation thus, changes the motion of ground. In general, SSI consists of inertial-interaction effects and kinematic interaction effects (Veletsos & Prasad, 1989).

2.2.1 Inertial Interaction Effects

Inertial interaction is fundamentally related to the vibrating structures, which induce inertial interaction effects due to the foundation rotations and displacements as well as the energy dissipation (Tileylioglu et al., 2008). Similarly, the vibrating structure induces base shear, moments and torsion due to inertial effects. Displacements and rotations are generated at the base of foundation due to these forces because of foundation flexibility. During this process, energy is dissipated through radiation damping and hysteretic soil damping hence, overall damping of structure is affected. These effects are referred as inertial interaction effects (NIST GCR 12-917-21). Considering SSI effects, structural designer can evaluate the inertial effects and actual deformations of the structural system precisely due to free field ground motion (Stewart et al., 1999; Prakash et al., 2016).

An SDoF system with stiffness, k and mass, m, with rigid base, is illustrated in Figure 2.1a. A static force, (F) produces lateral displacement "Δ":

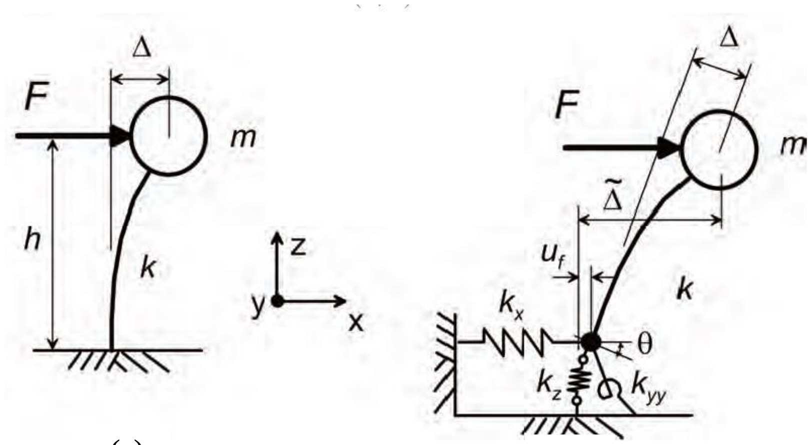
Δ= F/K Equation 2.2

In dynamics of structure, the undammed natural frequency"ω" and time period "T" of the building structure proposed by (Veletsos, 1989) is:

$$\omega = \sqrt{\frac{k}{m}}\ , \quad T = \frac{2\pi}{\omega} = 2\pi\sqrt{\frac{m}{k}}$$ Equation 2.3

Solving Equation 2.2 and Equation 2.3, square of time period is obtained as:

$$T^2 = (2\pi^2)\frac{\Delta m}{F}$$ Equation 2.4



(a) (b)

Figure 2.1 Shows the displacements produced by the applied force to:

fixed-base structure; and (b) structure having flexibile vertical, horizontal, and rotational base.

The vertical, horizontal, and rotational springs at the base of SD oF system depicting the possible effects of flexible soil vs a fixed base are shown in figure 2.1. The vertical and horizontal spring stiffness is denoted by k_z and k_x respectively whereas; rotational spring is denoted as k_{yy}. Now if the structure is displaced by the same force "F" in the in the horizontal direction, the structure will be deformed in the same way as in the fixed base system. The horizontal spring will be deformed by "u_f" due to base shear while the rotational spring will be deformed by angle,"θ" due to overturning moment. Hence, the total displacement with respect to the free-field motion at the top of the structureΔ' is modified. The modified time period "T" can be calculated by equation 2.5.

$$T'^2 = (2\pi)^2 m\left(\frac{1}{k} + \frac{1}{kx} + \frac{h2}{kyy}\right)$$ Equation 2.5

Combining the Equation 2.5 and 2.2 gives the classical period lengthening expression i.e. equation 2.6 (Veletsos and Meek, 1974):

$T'/T = 1 + \frac{F}{kxx} + kh^2/kyy$ Equation 2.6

Using Equation 2.6 time period lengthening of MDoF structures can be calculated by taking the height, h= effective height (2/3rd or 0.7 times the total height of structure) (ASCE/SEI 7-10:ASCE, 2010). In such cases period lengthening is significant for the first mode period.

(Veletsos and Nair (1975) and Bielak (1975), have shown that the dimensionless parameters which controls the time period lengthening are:

$\frac{\mathrm{h}}{VsT}$, $\frac{\mathrm{h}}{B}$, $\frac{\mathrm{B}}{L}$, $\frac{\mathrm{m}}{psABLh}$ and υ Equation 2.7

Where h= effective height of structure, B and L are half-width and half-length of the foundation, m=effective modal mass,

ρ_s= soil mass density, and υ_s = Poisson's ratio of the soil.

Period lengthening can be estimated from the equations 2.6 & 2.7 (Stewart et al., 1999b).

In equation 2.6 period lengthening does not depend on mass. To relate period lengthening to mass ratio and shear wave velocity equation 2.7 has been introduced. The effect of mass ratio is commonly taken as 0.15 (Veletsos, 1974). The stiffness and damping characteristics of the foundation is affected by Poisson's ratio of the soil.

In tall buildings period lengthening increases with h/B ratio due to increased overturning moment and foundation rotations but this is not the case. Since tall buildings have low

h/ (V_sT) ratios, which controls the inertial SSI effects. Overall the time period increment in tall buildings is approximately equals to one.

In addition to period lengthening, foundation damping B_f also affects the response of structure. Damping is composed of two parts:

(1) Hysteretic damping

(2) Radiation damping

Foundation damping contributes directly to the SSI systems.

2.2.2 Kinematics interaction effects

The frequency content and amplitude of foundation motions due to free-field motions are significantly affected by kinematic interaction (Veletsos & Prasad, 1989). The free-field ground motions and foundation input motions differ mainly due to kinematic effects. The stiff foundation structure causes the foundation motions to diverge from free-field motions due to base slab averaging, wave scattering and embedment effects in the absence of structure and foundation inertia (NIST GCR 12-917-21; Stewart, 2000).

2.3 "R" factor literature review

During the past four decades, vast research has been carried out on "R" factor. Veletsos and Newmark (1960) and Newmark and Hall (1973) are pioneers who work on "R" factor. A simplified "R" factor equation was formulated by Newmark and Hall (1973) on the basis of elastic and inelastic response spectrum of EL-centro1940 earthquake, which depends upon target period and ductility ratio of that structure. Whereas, the equation proposed by Lai and Biggs (1980) also depends upon target ductility and time period. They evaluated the equation using mean inelastic spectra of 20 artificially generated seismic motions. There equation is closely related to Newmark and Hall. Ordaz and Pérez-Rocha (1998) and Lam et al. (1998) also proposed similar equation which depends on relative inelastic displacement and ductility. All of these studies were performed without considering SSI.

Influence of the underlying soil medium on "R" factor was first discussed by Elghadamsi and Mohraz (1987) similarly, Krawinkler and Rahnama (1992) and Miranda (1993) also studied the influence of soil condition on R factor, especially for the structure founded on soft soils. However, their research work does not reflect direct effect of SSI on "R" factor.

Massumi and Tabatabaiefar (2008) studied the effect of SSI on reinforced concrete (RC) MRF systems with the variation of storey height. It was concluded that structures situated on soil Type II with Vs>375m/s and Vs<750m/s, the average ratio of storey displacements and drift with and without SSI are less than unity.

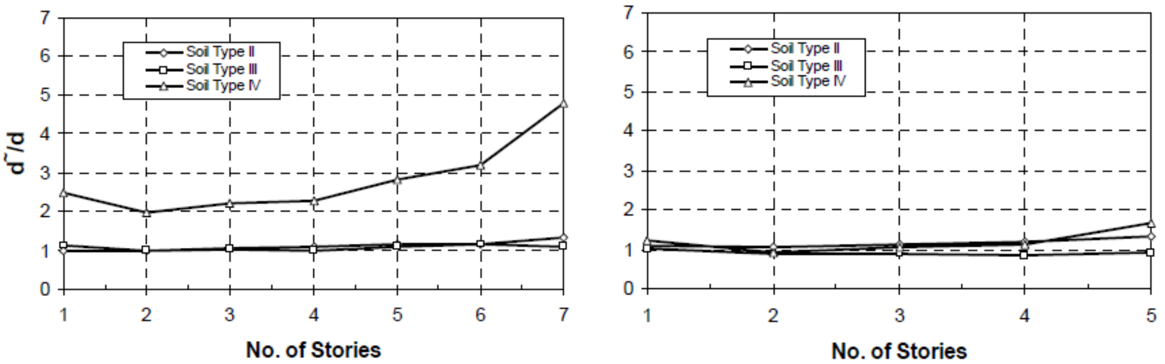


Figure 2.2 Average Ratio of story displacement of SSI to fixed-base on soil types II, III and I for 7 and 5 storied buildings

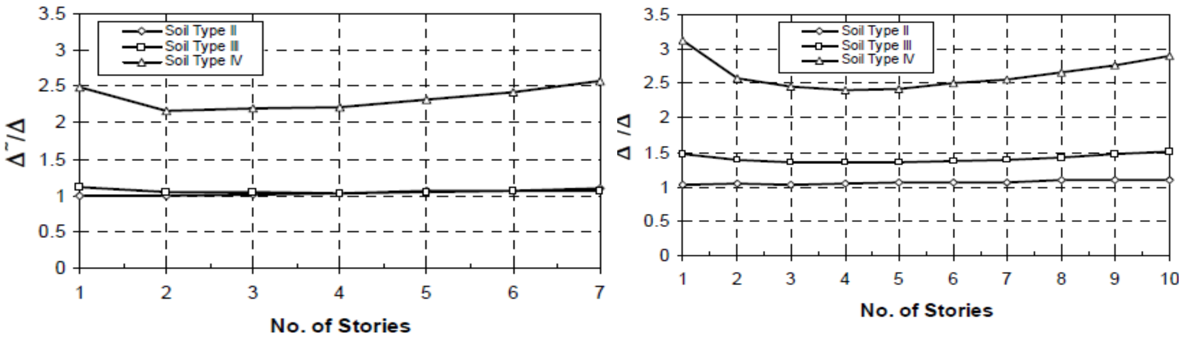


Figure 2.3 Average Ratio of story displacement of SSI to fixed-base on soil types II, III and IV for 7 and 10 storied buildings

G. Saad, F. Saddik and S. Najjar (2012) investigated the performance of building structure considering the SSI effects with varying number of basements. Soil class S_C and S_D with shear wave velocity 500 m/s and 275 m/s had been considered. Following parameters had been compared in their study i.e. base shear, inter-story shears and moments.

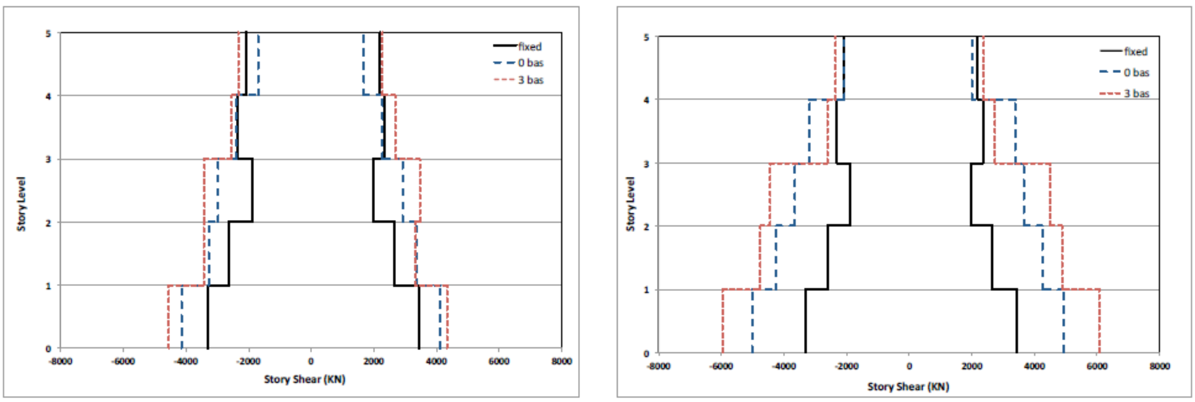


Figure 2.4 Story shear demands on the five-story building; (a) Soil Class SC and (b) Soil Class SD

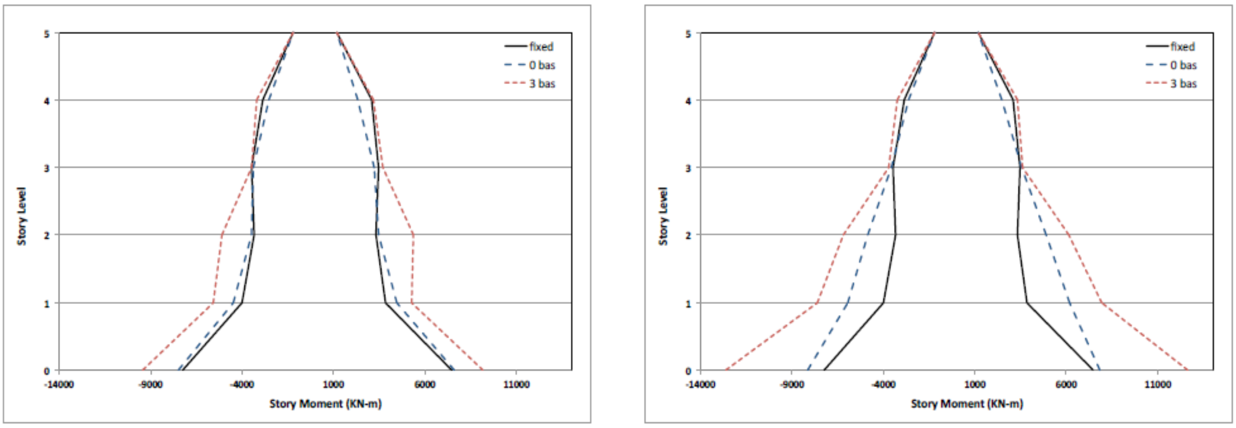


Figure 2.5 Story Moment demands on the five-story building; (a) Soil Class SC and (b) Soil Class SD

They concluded that storey shear increases with SSI while storey moments at bottom levels increased significantly.

Eduardo L. and AVILES J. (2004) investigated the influence of foundation flexibility on response modification factor using a simplified reference model based on Mexican buildings codes. They studied SSI system of a single-story elasto-plastic building structure, which is situated on a rigid foundation. Soil media is modelled as constant springs and viscous dampers. Linear springs depict the effect of soil inertia and stiffness whereas; viscous dampers depict the energy dissipation phenomena. The equation for "R" factor is: $(Te, \beta s) = \frac{Vm(1)}{Vy(\mu e)}$, where $V_m(1)$ is the strength required for elastic behaviour and V_y (μ_e) = target ductility. The main differences between the "R" factor with and without SSI occurred, when the structural time-period is close to the site time-period. ESER et. al. (2011) investigated the response modification factors for SDoF systems considering soil structure interaction. He used the SDoF replacement oscillator with effective period and damping of the system. The time-period of SDoF had varied from 0.1-3.0 s, aspect ratios varies as (h/r = 1, 2, 3, 4, 5), where "h" and "r" are building height and equivalent foundation radius respectively and five types of ductility demand (μ = 2, 3, 4, 5, 6). The said soil-structure model is analysed in time domain. He assumed the soil profiles with shear wave velocity of 150 m/s for site class D.

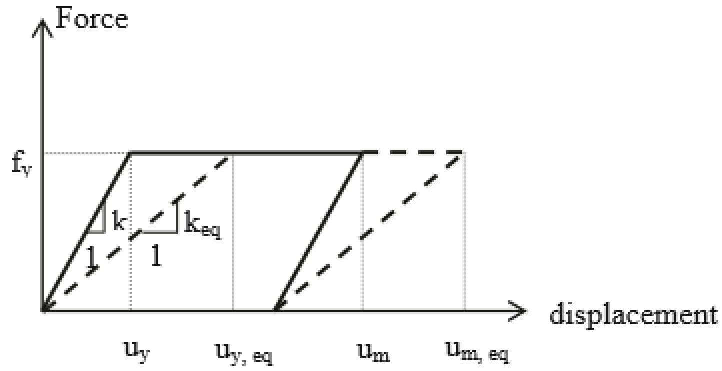


Figure 2.6 Force-displacement relationships of the SSI system (solid line) and equivalent fixed-base system (dashed line)

Figure 2.6 shows the displacement versus force relationship. The systems were subjected to 20 earthquake motions recorded on site class D. Figure 2.7 shows the variations of mean strength-reduction factors versus period on soft soils with (solid line) and without (dashed line) SSI for a system with ductility demands of 4 and 6 and aspect ratio of 3. He concluded that SSI reduces strength reduction factors for soft soils hence, the use of fixed-base “R” factors for SSI systems lead to unsafe design forces.

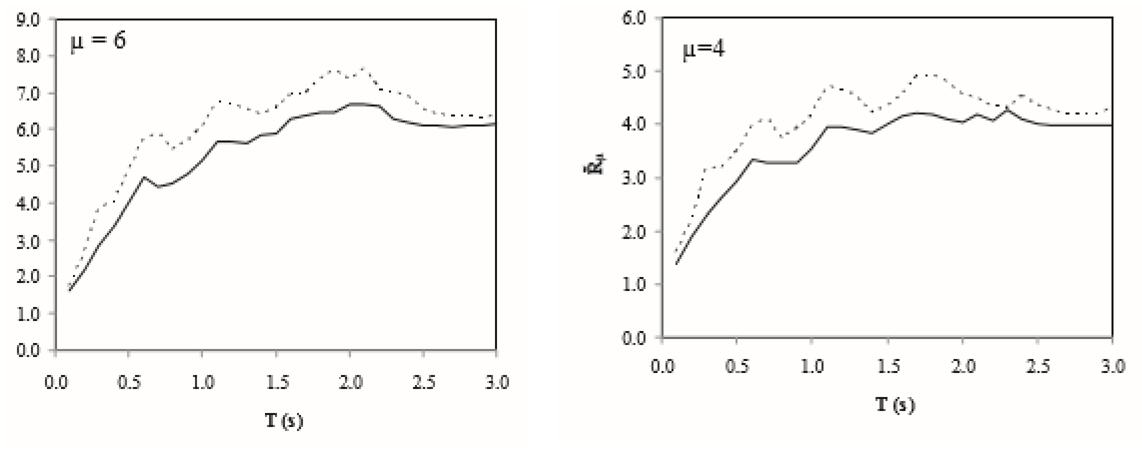


Figure 2.7 Variations of period versus mean strength-reduction factors with (solid line) and without (dashed line) SSIfor P = 4 and 6, with h/r = 3 (Eser et. al, 2011)

Effects of SSI on fundamental time-period, story drifts, total base shear, story displacements, force at inner columns and moments at beams ends has been examined. Nonlinear regression analyses were performed to evaluate a simplified equation for estimating$\widetilde{R\mu}$. The simplified regression formula is expressed as;

$R_u = 1+a(u-1)(u^b + T^c)^{1/T}$ Equation 2.8

They concluded that most important factors, which affect the $\widetilde{R\mu}$ factor values, are ductility demand and aspect ratio. The minimum ratio of $\widetilde{R\mu}$ with SSI to the fixed base case is 0.56 for site class D in short period region. Values of $\widetilde{R\mu}$ factors for structures situated on soft soils have been reduced considerably (Eser et. al., 2011).

Ghannad M. and Jahankhah H. (2004) investigated the Soil-Structure Interaction (SSI) effect on R factor. They modelled the superstructure as an elasto-plastic SDoF system and foundation as a circular rigid disk. The soil beneath the foundation has been modelled in three layers as 3-DoFs system, for sway and rocking modes as well as an internal DoF for considering the frequency dependency of soil stiffness.

Considering an idealized elasto-plastic SDoF system, as shown in Figure 2.8, R factor is defined as follows.

$R = f_e / f_y$

Schematic illustration of their model is shown in figure 2.9.

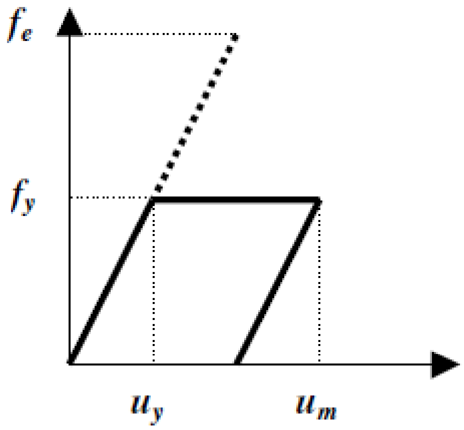


Figure 2.8 Idealized elasto-plasticbehavior

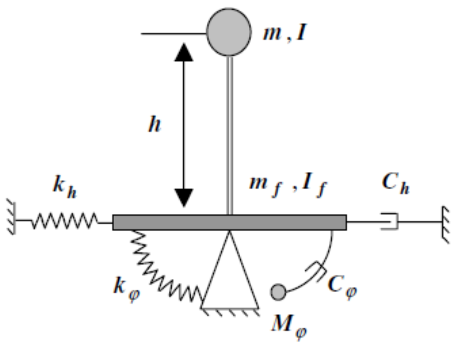


Figure 2.9 Soil-structure model Ghannad M. and Jahankhah H., (2004).

The following main parameters are used to define the soil in their model:
*m (*Structure-to-soil mass ratio index)=0.5 , *mf/ m (*ratio of the mass of the foundation to that of the Structure)=*0.1, v (*Poisson's ratio of the soil) = *0.4* , ξo= ξs (Material damping ratios of the soil and the structure)=*0.05*

The whole 4-DoFs model is analysed under 24 strong motions recorded on alluvium deposits. Analyses have been performed for three values of aspect ratio (*h/r*=1, 3, 5), three values of non-dimensional frequency (a_0=0, 1, 3) and three values of ductility demands (μ=1, 2, 6). Values a_0=0 and μ=1 are related to the fixed-base and elastic states respectively. Although, for alluvium sites a_0 is approximately limited to the range of 1 to 2 because of the uncertainty on a_0 values for this type of soil, the results are presented for a_0=1, 3 in comparison to the fixed base case (a_0=0). The results for R factor is shown in Figure 2.10 whereas, the abscissa is the period of structure in the fixed-base state (T_{STR}). It is clearly seen in this figure that SSI reduces strength reduction factor (SRF) values and the reduction becomes more significant as a_0 increases. However, it should be noted that although SSI affects the inelastic strength demands, this reduction in "R" factor values is mainly due to SSI effect on elastic response of structures.

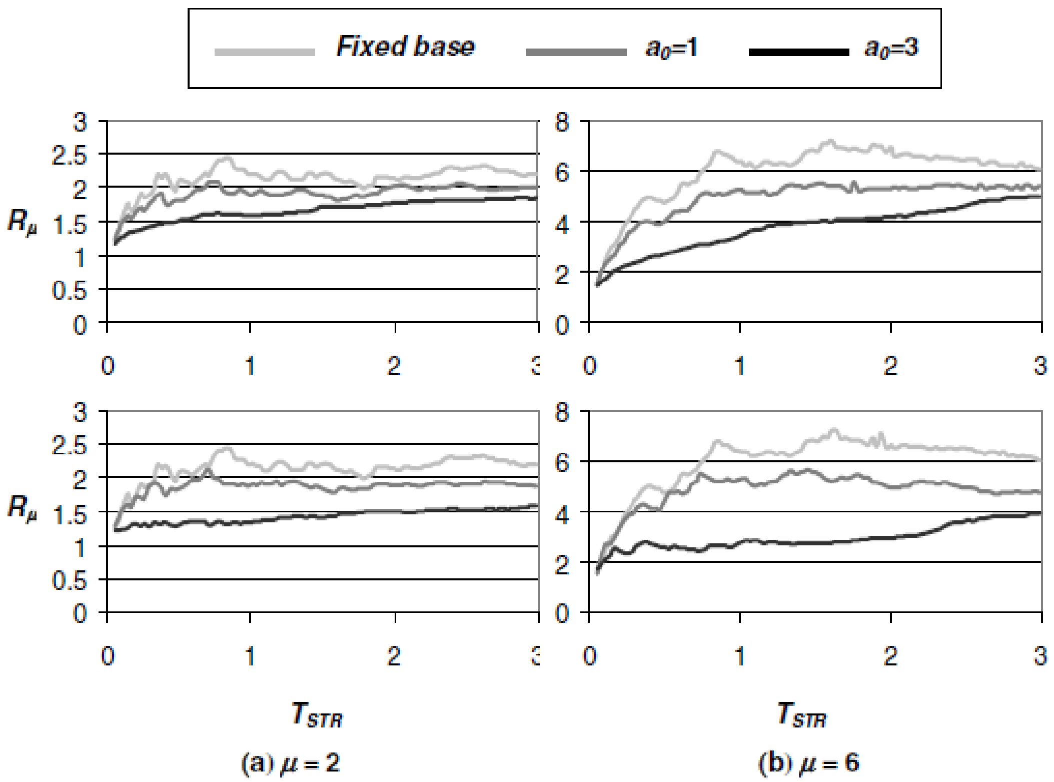


Figure 2.10 Strength reduction spectra (Gannad and Jahankhan, 2004)

However, all of these studies do not cosider MDoF structures in their research work. The relationship between MDoF and SDoF system with fixed base was first carried out by Veletsos and Vann (1971). He considered shear-beam models with equal story masses connected by weightless springs in series from SDoF to 5-DoFs systems. They inferred that the relations for SDoF systems cannot be used for MDoF systems. The proposed design regulations for SDoF systems were not sufficiently accurate for MDoFs system and errors increase as the number of degrees of freedom increased. Nassar and Krawinkler (1991) conducted parametric study to find the relationship between SDoF and MDoF for a SSI system. They limit the story ductility demand in the first story of the MDoF systems to a predefined value and then evaluate the changes required to incorporate the inelastic strength demand from SDoF system to Equivalent MDoF. They concluded that when the time period and target ductility ratio increases, the response of MDoF system also deviate greatly from SDoF system response. However, all of the works were performed on presumed assumption that soil beneath the structure is rigid i.e. fixed base system.

Halabian and Erfani (2010) studied the SSI effects on general reinforced concrete frame (RC) models due to the relative stiffness of soil and structure. They inferred that considering the foundation flexibility values of "R" factor changes and ignoring the effects of SSI does not show actual results.

In a more recent study Ganjavi and Hao (2011) investigated the effect of SSI on the strength and ductility demands of steel frame MDoF systems as well as its equivalent SDoF (E-SDoF) models considering both elastic and inelastic behaviours. They inferred that the common SDoF systems do not reflect the strength and ductility demands of MDoF soil–structure systems especially for mid-rise and high-rise building due to the significant contributions from higher modes of vibration.

In this study the well-known shear-beam model is utilized to include the effects of higher modes, the number of stories and lateral strength and stiffness distribution on inelastic response of MDoF buildings with SSI. For the MDoF shear-building models each floor is assumed as a lumped mass to be connected by elasto-plastic springs. Story height of each building is 3 m.

By using sub-structure method they developed equivalent linear discrete model for SSI (E-SDoF), which is based on the cone model concept. Cone model depends on frequency coefficients and equivalent linear elastic properties. A typical MDoF soil structure system and the corresponding E-SDoF system are shown in Figure 2.11. The stiffness and energy dissipation of the supporting soil are modelled by springs and dashpot respectively.

“R” factors of E-MDoF SSI systems models are calculated by ratio of elastic shear strength to the inelastic base shear strength.

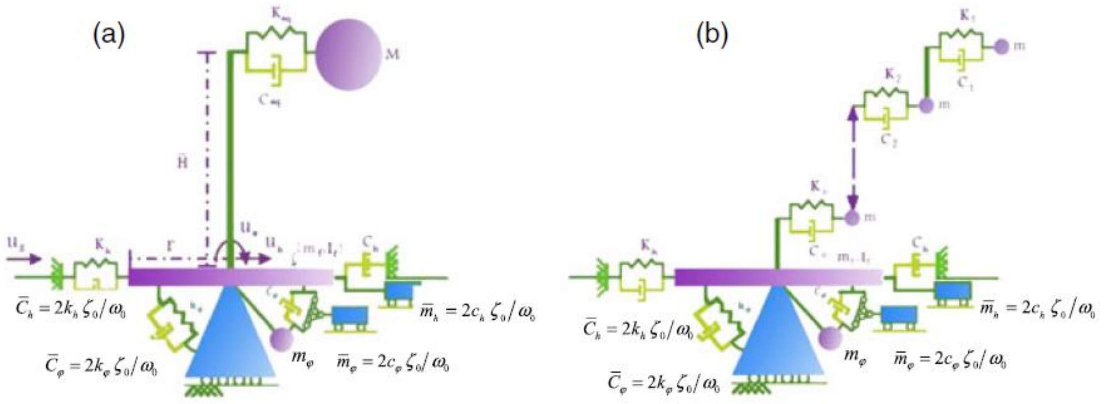


Figure 2.11 SSI models for sway and rocking motions (a) E-SDoF, (b) Typical MDoF system (Ganjavi B. and Hao H, 2014)

Figure 2.12 shows the comparison of average ratio of “R” factor with and without SSI having aspect ratio of 3, frequency a0 = 2 and the target story drift ratio asμt = 4. The abscissa shows the first mode time period (T_{fix}) of fixed base and flexible base systems respectively while ordinate shows the average “R” factor values of E-MDoF system. They concluded that the “R” factor of fixed base system with shear walls is independent of height of building.

They inferred that SSI reduces the “R” factor and establish a simplified equation to evaluate the “R” factor of E-MDoF system, which is as follows:

$R_{u(MDoF)} = a_i T^{b_i}{}_{\mathrm{fix}}$ Equation 2.9

Equation 2.9 depends upon the fixed-base fundamental period & a_i and b_i. In this equation a_i and b_i are constants, which depend upon frequency, the number of stories, ductility ratio and slenderness ratio of structure.

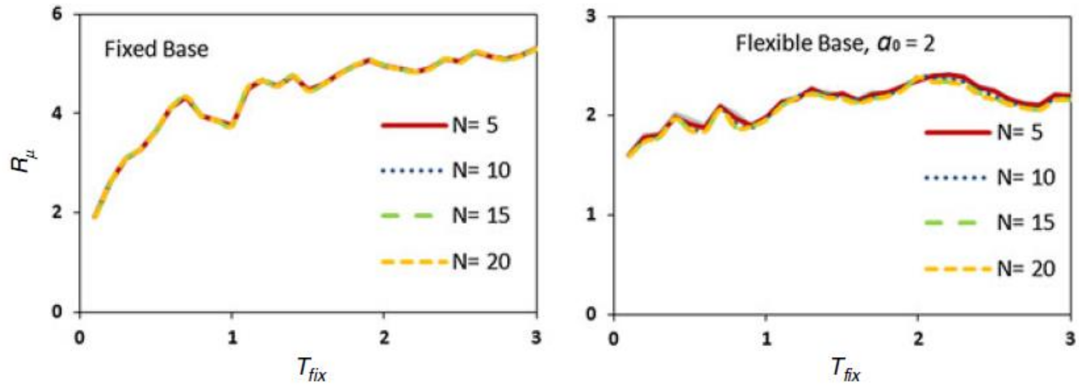


Figure 2.12 Comparison of the averaged strength reduction factor for different E-MDOF systems (m = 4) (Ganjavi B. and Hao H 2014)

2.4 Summary

Effects of SSI on structural seismic response have been discussed in the light of previous research studies. SSI lengthens the time period and amplifies the structural response such as story shear, story displacement and storey drift. Literature review shows that most of the studies considering SSI for the evaluation of “R” factor have been conducted considering the behaviour of SDoF system whereas, the response of MDoF needs different approach. In this way the contribution of higher modes in the inelastic response of MDoF system cannot be evaluated properly. The studies conducted by Ganjavi B. and Hao H. (2014) for the evaluation of“R” factor of MDoF system was E-SDoF not the exact MDoF. Hence, “R” factor values for actual MDoF considering SSI still needs to be investigated.

CHAPTER 3

SOIL STRUCTURE INTERACTION MODELLING AND ANALYSIS METHODOLOGY

3.1 Background

Modelling the soil medium beneath the structure is one of the most important parameter in the soil structure interaction (SSI) analysis. The true behaviour of the SSI system can be assessed, if an appropriate modelling methodology has been adopted to model the soil medium. The most important parameter of foundation design is the distribution of contact pressure at the foundation and soil interface. The distribution of pressure varies depending upon the foundation behaviour (i.e., rigid or flexible) and rigidity of supporting soil (clay or sand etc.). The foundation design philosophy is to transfer the load of the super structure on to the supporting soil. In ideal foundation design modelling, the distribution of contact pressure should be depicted in an accurate manner (Taylor, 1948). Various methods have been used to model the SSI on shallow and embedded foundations which are as follows:

i. Wrinkler's model (spring model)
ii. Lumped parameter on elastic half space
iii. Numerical methods

3.1.1 Wrinkler's model (spring model)

According to Bowls (1996), Wrinkler's approach represents the soil medium using horizontal and vertical closely spaced, linear elastic springs which are identical but mutually independent. According to this theory foundation deformation occurs only at loaded areas. In Wrinkler's approach, linear springs are used to model soil layer (Wrinkler, 1867). Physical representation of wrinkle foundation is shown in Figure 3.1.

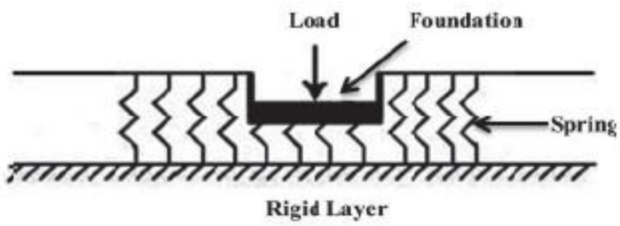


Figure 3.1Winkler foundation

The pressure–deflection relation at any point is given by:

$$p = kw \qquad \text{Equation 3.1}$$

Where, p = pressure from superstructure, k = modulus of subgrade reaction and w =deformation.

"k" is the ratio of pressure p to deformation at any given point of the contact surface i.e. k=p/W (SC Dutta -2002)

Baker (1957); Vesic (1961); Kramrisch et al. (1961); Bowles (1996); and Brown (1977) conducted research following Wrinkler's methodology because of its simplicity. The value of "k" depends on the following parameters i.e. nature of the soil, dimensions of the foundation area and depth from natural surface level. In the Wrinkler's method stiffness of the associated elastic springs is the only parameter to model the physical behaviour of the soil medium. Hence, the numerical values of soil springs must be determined with care in order to use in a practical problem.

Dutta and Roy (2002) recommended that Wrinkler hypothesis despite its limitations yields reasonable performance and it is very easy to model. Thus for practical purposes, this idealisation is preferred because of its simplicity

The values of subgrade modulus can be also be evaluated by experimental procedures, which is as follows:

(a) Plate load test (Terzaghi, 1955; Bowles, 1996; Kurian, 1982)

(b) Consolidation test (Yong, 1960; Yong, 1960)

(c) Triaxial test (Vesic, 1961)

(d) CBR test (Nascimento, 1957)

Following some suitable method a reasonable value of subgrade modulus "k" can be found. But in the absence of suitable test data, representative values for the same may be calculated by the following Gazetas (1991) method. Gazetas (1991) proposed that the method to be selected must consider the properties of the SSI system and the excitation as follows:

• The layout of the foundation at soil boundary line i.e. arbitrary, strip, rectangular, circular

• The depth of foundation

• The nature of the soil medium i.e. deep uniform or layered deposit, shallow stratum over bedrock

• The mode of vibration and the frequency of excitation.

He formulated an entire set of formulas to compute the values of soil spring "k" as shown in table 3.1 & 3.2. This covers all foundation type of forms, all types of embedment i.e. shallow, partially and whole for all modes of vibration and frequency content. Table 3.1 & 3.2 shows the horizontal and vertical spring's formulas used in the present study. Table 3.3 shows the corresponding soil parameters.

Table 3.1 Dynamic stiffness of springs for any type of foundation on surface of assumed Homogeneous Half-Space (Gazettas, 1991)

Degrees of freedom	**Stiffness of equivalent soil spring on surface**
Vertical	$[2GL/(1-\nu)](0.73+1.54\chi^{0.75})$ with $\chi = Ab/4L2$
Horizontal (lateral)	$[2GL/(2-\nu)](2+2.50\chi^{0.85})$ with $\chi = Ab/4L2$

Table 3.2 Dynamic stiffness of springs for any type of foundation with embedded in assumed Homogeneous Half-Space (Gazettas, 1991)

Degrees of freedom	**Stiffness of equivalent soil spring of embedded foundations**
Vertical	$K_{z,\ \mathrm{embedded}} = K_z + (1/21)(D/B)(1 + 1.3x)][1 + 0.2(A_W/A_b]^{0.67}$ Where, $K_z = K_z$, are given in Table 3.1 $Aw = (d)_X$ (perimeter); $x = Ab/4L2$ Where "d" is the depth of embedment.
Horizontal (lateral)	$K_{y,\ \mathrm{embedded}} = K_y + 0.15(D/B)^{0.5}]\{1 + 0.52[(h/B)(A_w/L^2)]^{0.4}\}$ $K_{x,\ \mathrm{embedded}} = K_x(K_{y,\ \mathrm{embedded}}/K_x)$ Where, K_x & K_y, are given in Table 3.1

Where, A_b = area and I_{bx}, I_{by}, and I_{bz} = area moments of inertia about the x, y and z axes of the actual soil-foundation contact surface.

• B and L = half-width and half-length of the foundation

• G, v and V_s the shear modulus, Poisson's ratio and the shear-wave velocity respectively.

The values of soil parameter for different soil profile type are given in table 3.3 (Fema, 1997) and (Fema, 2000).

Table 3.3 Details of soil parameters used for the calculation of soil spring values (Fema, 1997) and (Fema, 2000)

Soil type	Soil property	Shear wave velocity (Vs) m/sec	Poission' ratio	Unit Weight (KN/m³)
S_B	Rock	1200	0.3	22
S_C	Dense	600	0.3	20
S_D	Stiff Soil	300	0.35	18
S_E	Soft soil	150	0.4	16

3.1.2 Lumped parameter on elastic half space

Where, A_b = area and I_{bx}, I_{by}, and I_{bz} = area moments of inertia about the *x*, *y* and z axes of the actual soil-foundation contact surface.

• *B* and *L* = half-width and half-length of the foundation

• *G*, *v* and *Vs* the shear modulus, Poisson's ratio and the shear-wave velocity respectively.

The values of soil parameter for different soil profile type are given in table 3.3 (Fema, 1997) and (Fema, 2000).

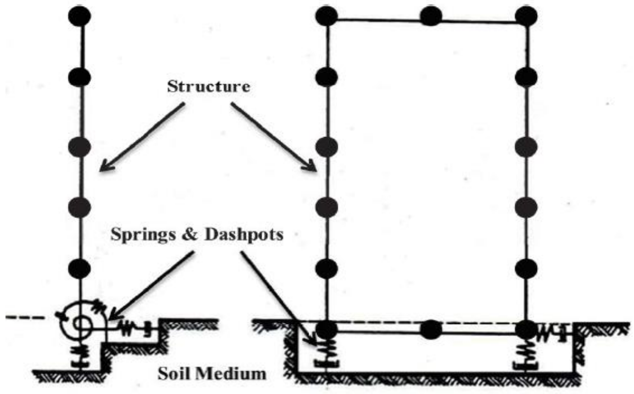


Figure 3.2 Schematic representation of springs and dashpots in lumped parameter on half space (Tabatabaiefar, 2012)

The stiffness of spring depends on the frequency of the forcing function, especially when the foundation is long and resting on saturated clay. Since frequency exerts inertia force, which represents the deformable behaviour of soil (Tabatabaiefar, 2012). Wolf (1994) developed a series of cone model parameters, which have been widely used in practical applications. Foundation stiffness coefficients of the proposed cone model are similar to the stiffness parameters proposed by Gazetas (1991). Bowles (1996) describes that in the Lumped Parameter method the effect of frequency dependent soil-flexibility on the behaviour of overall structural system is higher than the springs values obtained from frequency independent behaviour determined by Wrinkler's model.

Table 3.4 The cone model properties proposed by Wolf (1994)

Motion		**Spring Stiffness Coefficient(k)**	**Viscous Damping Coefficient (c)**
Vertical	$v \leq 1/3$	$4Ga/(1-v)$	$P.V_p.A$
	$1/3 < v \leq 1/2$		$P.(2V_Z).A$
Horizontal		$8Ga/(2-v)$	$P.V_Z.A$
Rocking	$v \leq 1/3$	$8Ga^3/3(1-v)$	$P.V_Z.I_r$
	$1/3 < v \leq 1/2$		$P.(2V_Z).I_r$

A is the foundation area, I_r is the moment of inertia for rocking motion, G is the shear modulus of the soil, v is the Poisson's ratio of the soil, Vs is the shear wave velocity of the soil and Vp is the compression wave velocity of the soil. Dutta and Roy (2002) elucidated that the effects of soil-structure interaction on the dynamic behaviour of structures may conveniently be analysed using the Lumped Parameter approach. However, numerical modelling may be required for important structures where more difficult analyses are necessary.

3.1.3 Numerical methods

Numerical methods further have two approaches i.e. Substructure method and direct approach. In numerical methods the effect of Soil is considered by modelling them in two or three dimension using finite element (FEM). The advantage of numerical methods is that inelastic behaviour of soil can be considered by numerical integration using equations of motion in time domain.

3.1.3.1 Substructure Method

In substructure approach SSI system is divided into three steps, which are then combined to analyse the actual SSI system. Sketch of substructure method is shown in figure 3.3. Kramer (1996) reported that the superposition for this method assumes linear soil and structure behaviour. Varun (2010) described the three steps for the analysis of SSI using substructure approach.

- Estimation of foundation Input Motion (FIM)by assuming that the structure and substructure is massless
- Impedance function which is stiffness and damping characteristics of SSI system is calculated.
- Dynamic analysis of the structure with SSI.

Several researches (Kutan and Elmas, 2001; Yang et al., 2008; Carbonari et al., 2012) have been carried out by using substructure method in analysing the seismic response of SSI structural systems. Chopra and Gutierres (1978) found that the most important benefit of the substructure approach is its independency. However, Wolf (1998) mentioned that due to superposition principle it is only true for linear soil and structural system. Soil nonlinearity approximations with the help of iterative wave propagation analyses are true only for moderately-nonlinear systems. Therefore, the effect of exact nonlinearity of the subsoil in the dynamic analysis may not be easily achievable by this method. Kutan and Elmas (2001) stated that the influence of various other factors on the response of a soil-structure system is still needs to be explored. Moreover, the material damping of foundation media needs to be investigated.

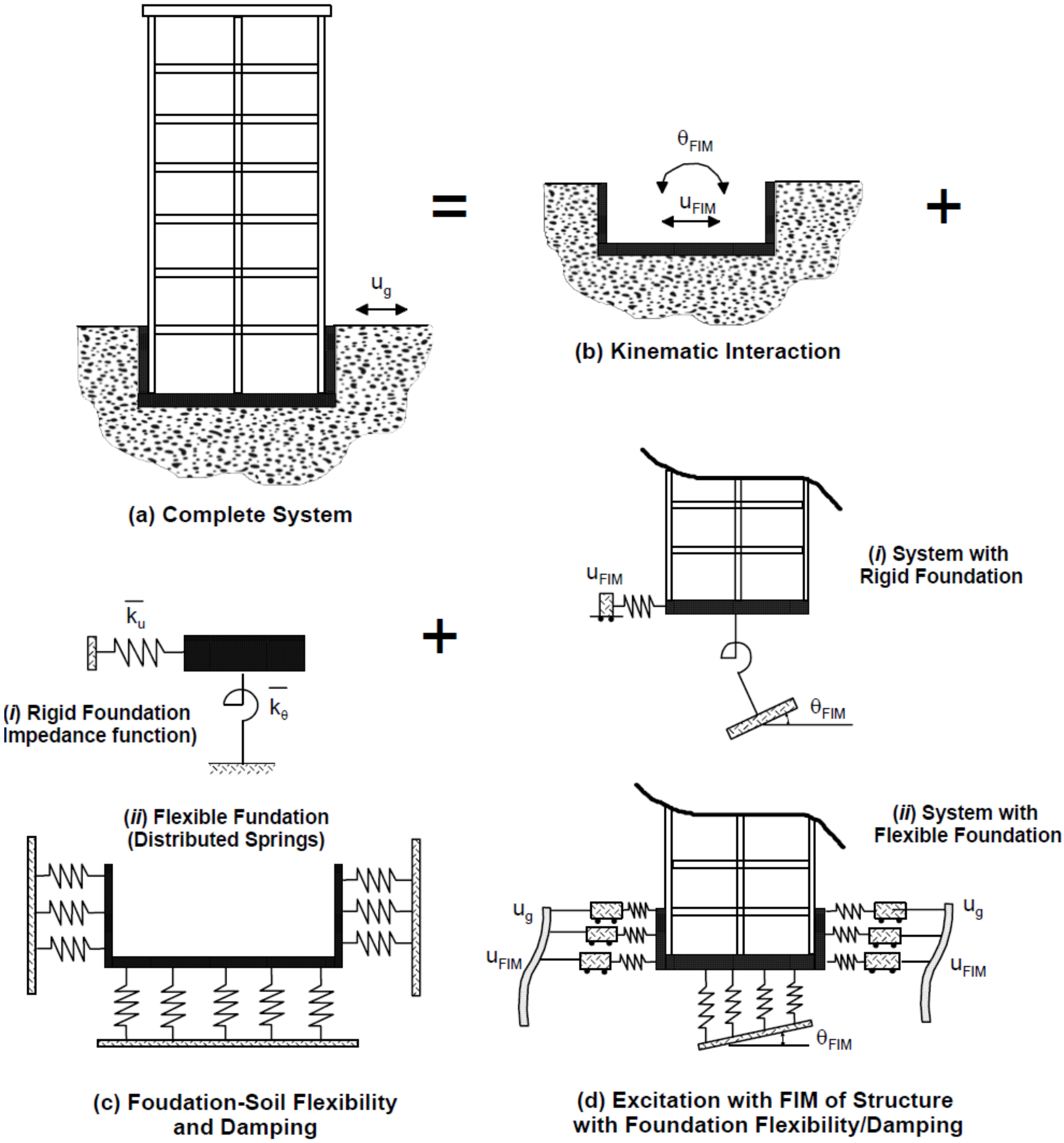


Figure 3.3 The sketch of substructure approach (NIST GCR 12-917-21).

3.1.3.2 Direct Approach

It is the direct method of soil-structure system in which analysis is being performed in a single step. Sketch of a direct approach of SSI system is shown in Figure 3.4. Numerical methods such as the finite element method (FEM) [the boundary element method (BEM)] are used for analysis (Wang et al., 2017). Typically, the soil is modelled as solid finite elements and the super structure as finite beam elements. Several researchers (e.g. Desai et al., 1982; Mirhashemian et al., 2009; Tabatabaiefar & Massumi, 2010; Gouasmia1 & Djeghaba, 2010) have studied dynamic response of soil-structure systems adopting direct method for modelling soil-structure interaction to achieve accurate and realistic analysis outcomes. Carr (2008) believes that the advantage of this method in fact is its versatility to deal with complex geometries and material properties. However, data preparation and complexity of the modelling makes it difficult to implement it in every-day engineering practice. In addition, advanced computer programs are used for analysis. In this method exact nonlinear analyses is possible (Borja et al., 1992).

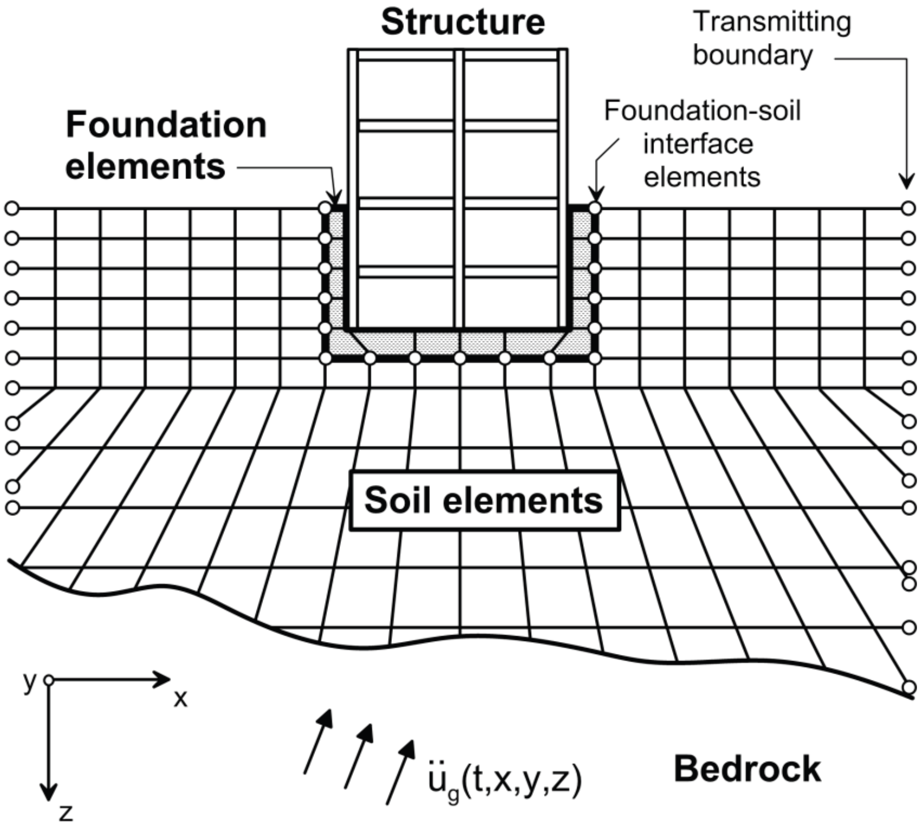


Figure 3.4 Sketch of a direct analysis of SSI system (NIST GCR 12-917-21)

3.2 Performance based seismic design

The key objective of performance based design is to analyse structure with predictable damage. In performance-based design methodology two quantities are required to be evaluated for analysis and design purposes i.e. the seismic capacity and the seismic demand. The seismic capacity is the ability of structure to resist the seismic effects whereas; seismic demand is the earthquake effects imposed to the building. The structure is designed in way that the capacity is more than the demand (ATC-40, 1996). Following analytical methods for design and analysis purposes are mentioned in the guidelines of FEMA 356 and ATC 40, namely Linear Static method, Linear Dynamic method, Nonlinear Static method and the Nonlinear Dynamic method. In this study nonlinear static 'pushover' analyses has been used to assess the inelastic seismic design.

3.2.1 Nonlinear Static pushover analysis method (PoA):

Pushover analysis is a static nonlinear procedure. In this method magnitude of the structural loading is increased in accordance with a certain predefined displacement or force. This method was first adopted by Freeman et al.(1975) which is called as Capacity Spectrum Method. The key purpose of this was to evaluate the seismic performance of a series of 80 buildings located a shipyard in the USA using a simplified method. As the magnitude of load application increases weak links and failure modes of the structure are found, which is called plastic hinge formation. The magnitude of load is consistent with the effects of the cyclic behaviour and load reversals. PoA is an effort by the structural engineering profession to assess the actual strength of the structure and it is a useful and effective performance based design tool.
The results of the PoA are a base shear versus roof displacement curve, which is called the capacity curve as shown in Figure 3.5. This capacity curve estimates how structure will behave after exceeding its elastic limit.

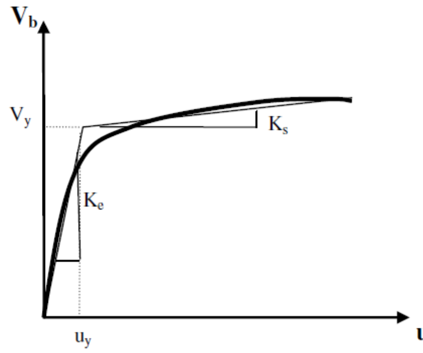


Figure 3.5 Capacity curve for MDOF structure(Spyridon Themelis, 2008).

Most commonly used POA methods are; Capacity Spectrum Method (CSM) and Displacement Coefficient Method (DCM).

The DCM requires the estimation of the target displacement. The target displacement Δ_t for a building with rigid diaphragms at each top floor is assessed using a well-known procedure that accounts for the nonlinear response of the building (FEMA, 273).

$$\Delta t = C_0 C_1 C_2 Sa Te^2 / 4\pi^2 g \qquad \text{Equation 3.2}$$

Where:

Te = Effective fundamental period of the building in the direction under consideration, sec

C_o = Modification factor to relate spectral displacement and likely roof displacement.

C_1 = Modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response.

C_2 = Modification factor to represent the effect of hysteresis shape on the maximum displacement response.

Sa = Response spectrum acceleration, at the effective fundamental period and damping ratio of the building in the direction under consideration, "g".

The force-deformation curve used in PoA is shown in Figure 3.6The points labelled as A, B, C, D, and E on the pushover curve are used to define the force deflection behaviour of the hinge while the points labelled as IO, LS and CP are used to define the acceptance criteria for the hinge. Point A represents the unloaded condition, whereas from A to B is elastic state. From point B to C, the stiffness reduces. Point C has a resistance equal to the nominal strength then at point D there is sudden decrease in lateral load resistance, the reduced resistance at E and then final loss of resistance. The regions IO, LS and CP represent Immediate Occupancy, Life Safety and Collapse Prevention respectively. Values of each reach are different depending upon type of member, type of material and several other parameters which is defined in the guidelines of ATC-40 and FEMA-273 documents. The slope of the BC line is usually taken between 0 and 10% of the initial slope.

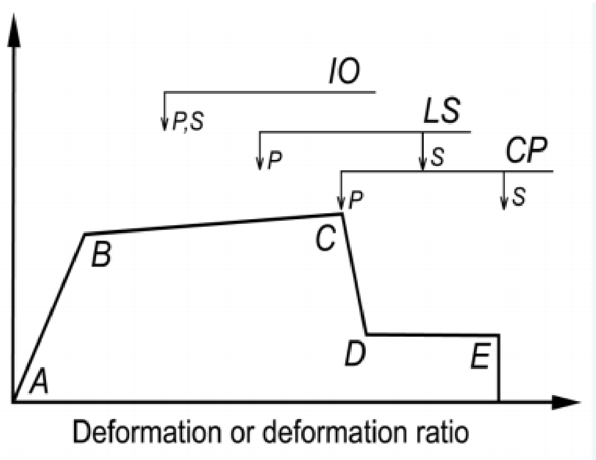


Figure 3.6 Displacement-Deformation for Pushover Hinges

3.3 Summary:

Different approaches to mode land analyse SSI system has been discussed in accordance with literature i.e. Wrinkler's model, lumped parameter on elastic half space and numerical method. Wrinkler's is most widely used due to its simplicity, whereas Lumbed parameter on elastic half space is the modified form of Wrinkler's model. Numerical methods are the advance form of SSI modelling using the finite element or finite difference in two or three dimensions. This method is advantageous because soil nonlinearity is also taken into account.

Also nonlinear static pushover analysis method has been discussed.

CHAPTER 4

CASE STUDY

4.1 Introduction

The present study attempts to cover a wide range of interaction problems in terms of the superstructure and the soil characteristics. To do so, an extensive parametric study using non-linear static pushover analysis along with two different types of soils, namely S_D (stiff) and S_B (Rock) has been carried out to evaluate seismic response of midrise MRF frame buildings. Soil properties have been modelled as wrinkler's spring. Values of soil spring have been computed by using formulas proposed by Gazetas (1991).

Commercial Computer Structure International (CSI) Software SAP-2000 v15.0.0. has been used for this analysis. Equivalent linear static and non-linear static pushover analyses have been performed. In this chapter, Values of "R" factor have been evaluated for soil type S_D and S_B and different parameters i.e. storey shear force, storey moments, storey displacements and storey drifts have been compared with and without SSI.

Using the equation 2.1 (NIST GCR 12-917-21) the structure-to-soil ratio (h/v_sT) can be calculated to determine the extents of SSI effects on the structural responses.

For the case study building, this ratio is 0.035 and 0.017 for S_D (V_s=300m/s) and S_B (V_s=1200m/s) respectively. According to NIST GCR 12-917-21 when, $h/v_sT > 0.1$, SSI affects has significant effects on the structural response while for, $h/v_sT < 0.1$, SSI still affects the response of structures, which shall be examined in the case study building.

4.2 Description of case study building

The building has two basement and G+7 stories. The building plan is symmetrical with length and width of 80 feet and three equal bays in both directions. Beams are placed on all grids. Typical height of each storey is taken as 12 ft. The building is mix-used commercial building with shops and departmental store on basement, ground and first floor, offices on 2nd, 3rd& 4th floor, and residential apartments on 5th, 6th and 7th floor. The architectural plans are shown in 4.1(a), (b) & (c).

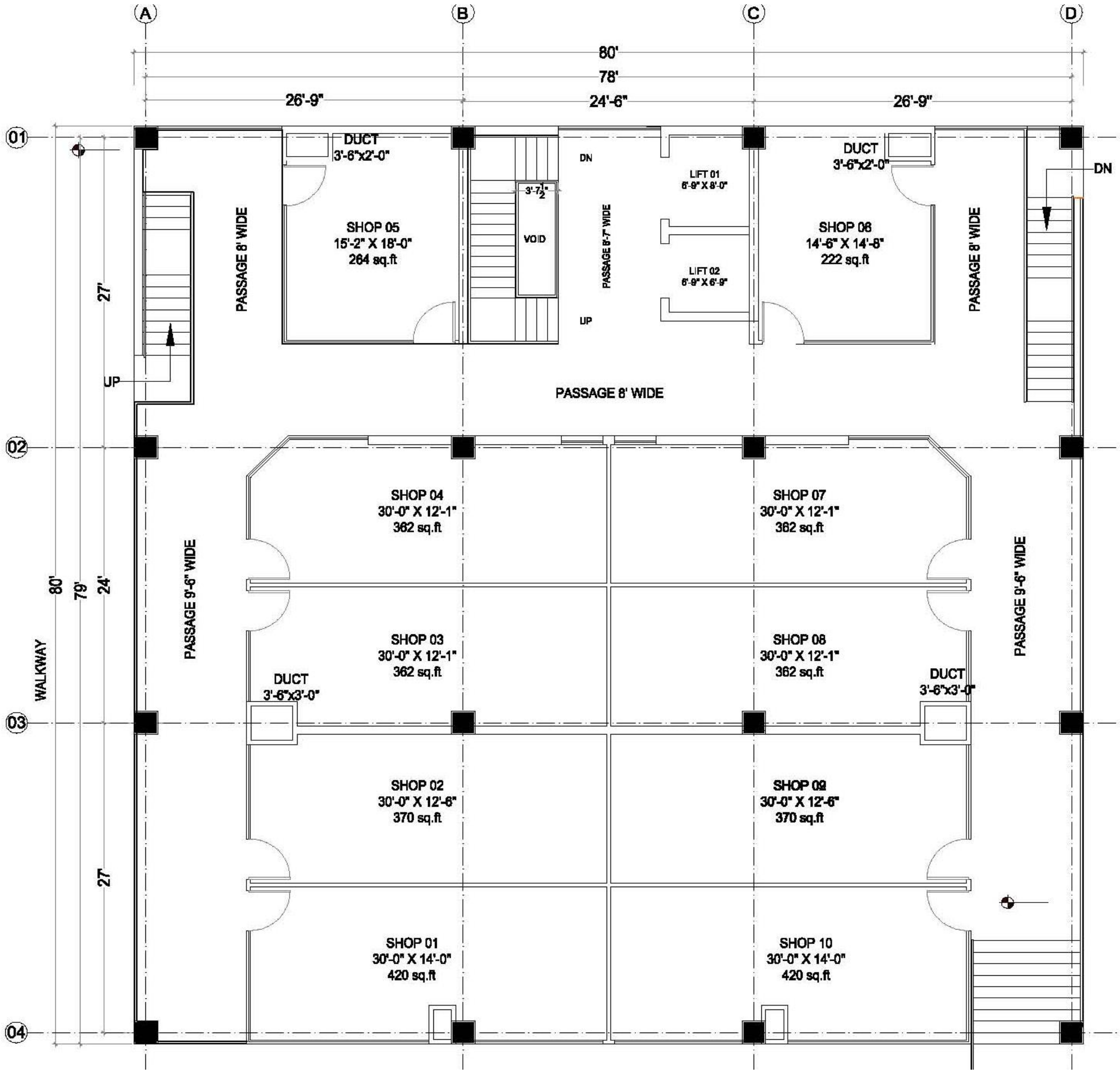


Figure 4.1(a) ground floor plan (shops)

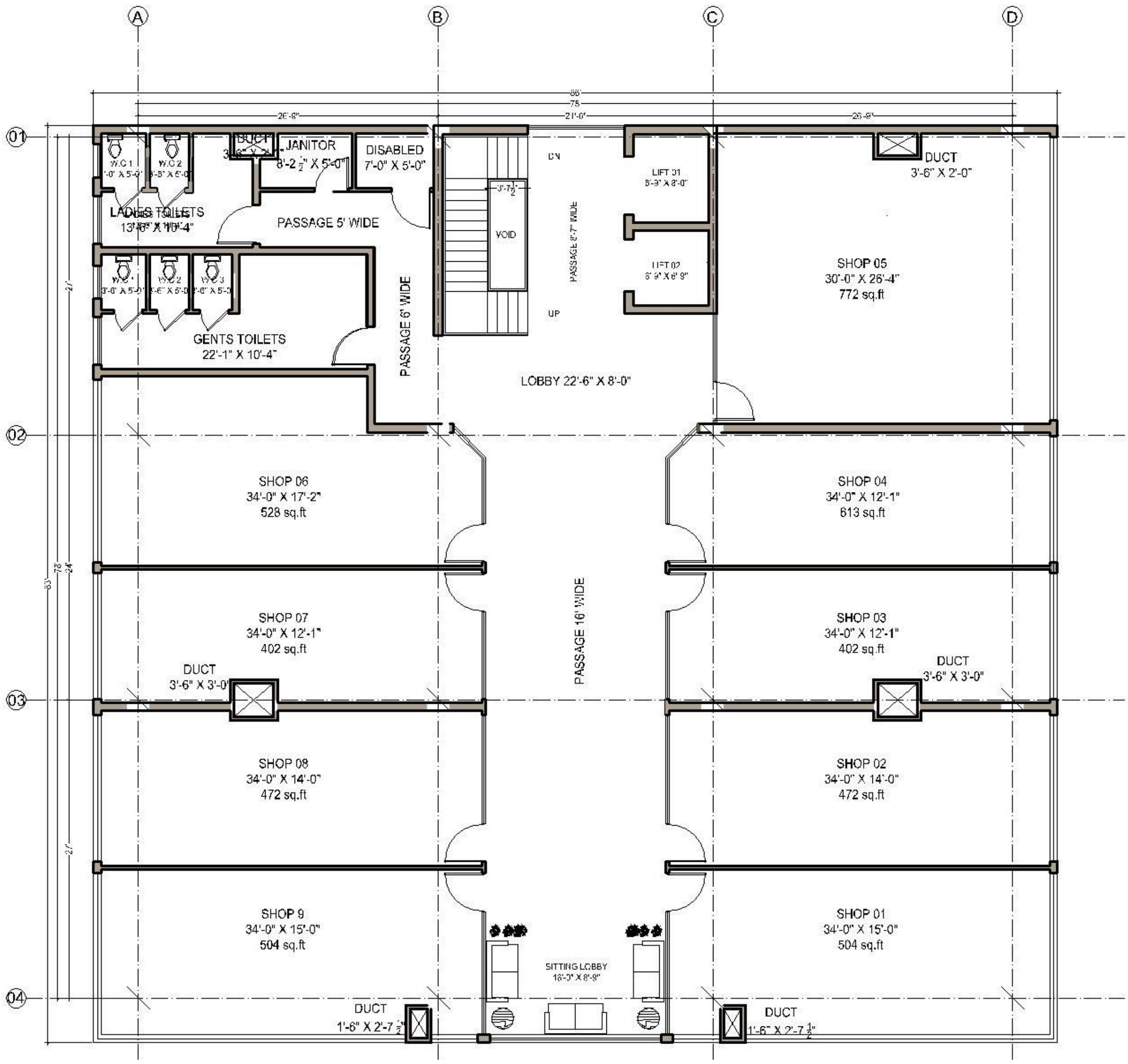


Figure 4.1(b) Typ. office floor plan

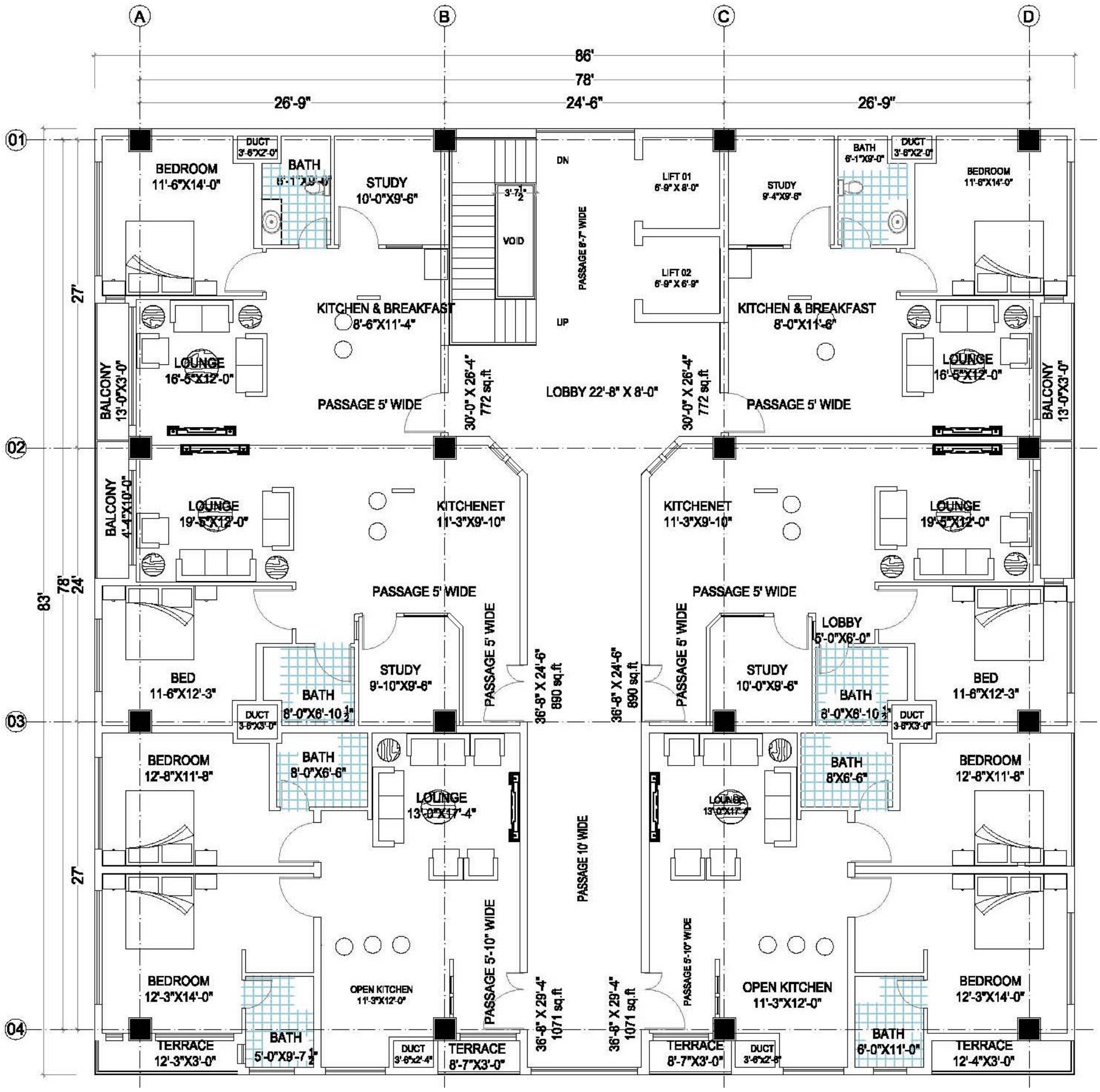


Figure 4.1(c) Typ. Apartment plan

Table 4.1 Cross-sectional details of building structural elements

Structural member	B1	B2	C1	C2	C3	C4
C/S area(inch^2)	12x24	12x30	30x30	27x27	24x24	21x21
Floors to which Assigned	1st,2nd,3rd	B1,B2, G,4th,5th,6th&7th	B1,B2, G,1st	2nd,3rd	4th ,5th	6th, 7th

Slab thickness is taken as 6 inches. Total eight models have been prepared, four with fixed base and four with soil springs for S_D and S_B soil profile type. The building structural elements have been first designed according to building codes UBC-97 by equivalent static force method.

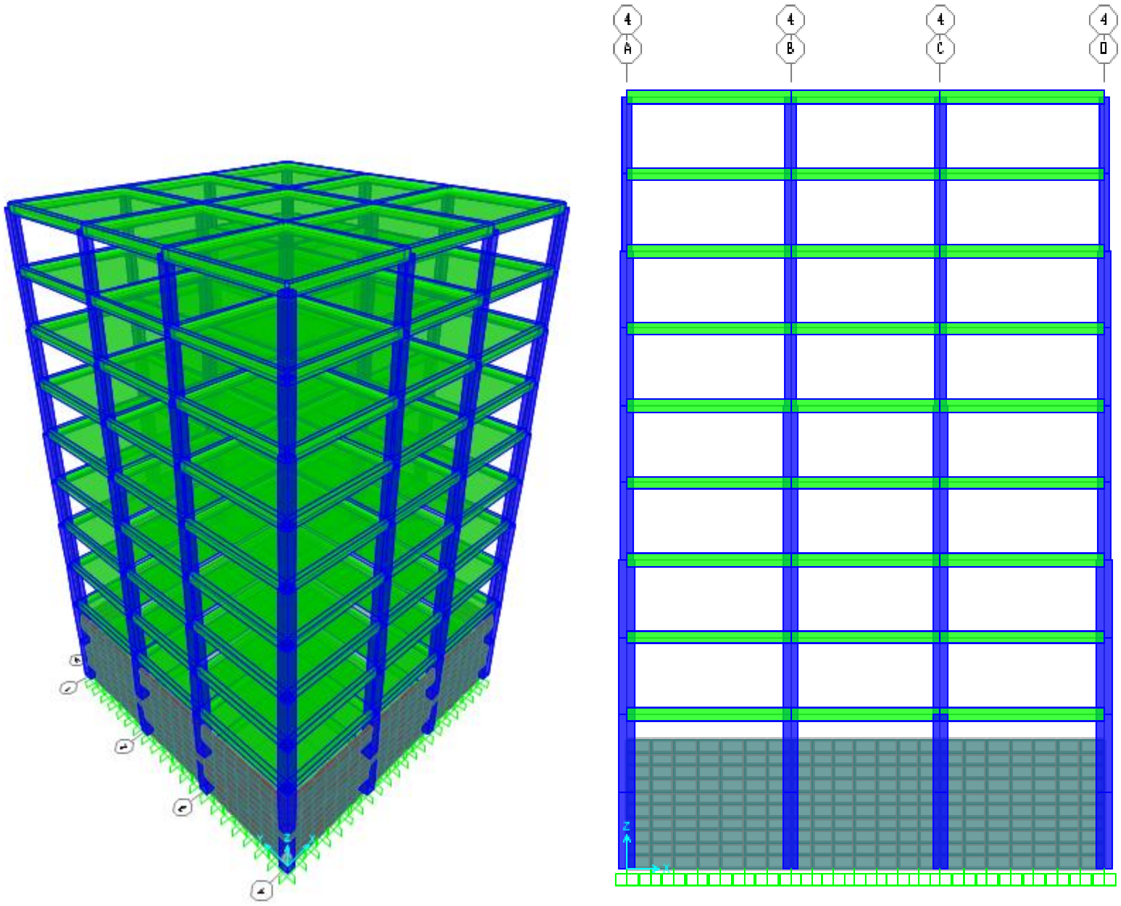


Figure 4.2 3D View & Elevation of case study building with fixed base

4.3 Equivalent Static analysis (ESA)

The equivalent static lateral force method is a simplified procedure to replace the effect of dynamic loading of an expected earthquake by a static force that is distributed laterally on a structure. The total applied seismic force (*V*) is generally estimated in two horizontal directions X and Y. In this method the assumption is made that the building will responds in its fundamental lateral mode. The structure must be able to resist effects caused by seismic forces in each direction (Bourahla, 2013).

For gravity load design, dead load includes self-weight of the structure, a typical load of 3 inch finishes and partitions wall loads. According to UBC-97 live load for shop floors is taken as 100psf and 50 psf for offices, and 40 psf for apartment floors and roof. Two load cases are defined as mass source (including self-weight) and 25% for live load of shops. Building is situated in seismic zone 2B. Importance factor is 1. For linear static analysis code based (UBC-97) value of “R” factor is taken as 5.5 considering the moment resisting frame building. Static load combinations of UBC-97 are followed. The time period from code based procedure (using method- A and method-B) comes out to be 1.776 sec. for both soil type soil type S_D and $S_{B.}$ This time period is believed to be based on stiffness contribution from both structural and non-structural components (Williams A, 1997). The equivalent static base shear for soil type S_D and S_B with and without SSI is 502 kips and 270 kip respectively. The equivalent static response quantities are denoted as “QX” in the following sections. The equivalent static base shear is multiplied with “R” factor to get the elastic base shear. Thus, the elastic base shear shall be 2761 Kips and 1485 Kips respectively. This elastic base shear shall be divided by the inelastic base shear obtained from Pushover Analysis to get the actual values of “R” factors for different cases.

4.4 Soil Structure Modelling

To model soil structure interaction direct approach has been used, in which superstructure, foundation and soil are modelled as single unit. Soil medium has been modelled as Wrinkler approach (Wrinkler, 1867), which is a system of identical but mutually independent, closely spaced, discrete, linearly behaving elastic springs. The effect of soil flexibility is accounted through consideration of springs of specified stiffness’s (Dutta and Roy, 2002). The stiffness along horizontal and vertical axis is determined with help of Gazetas (1991) formula as shown in table 4.2(Bhattacharya et al., 2004; Gazetas, 1991). Detail of soil parameters used for the calculation of soil spring values is shown in table 4.1.

Table 4.1Detail of soil parameters considered in this study

Soil profile type	**Description**	**Shear wave velocity (Vs)m/sec**	**Poission’ ratio**
$\mathbf{S_B}$	Rock	1200	0.3
$\mathbf{S_D}$	Stiff Soil	300	0.35

Following values of soil springs as shown in table 4.2 is used for the modelling of soil medium.

Table 4.2 Values of vertical soil spring for soil type Sd & Sb

Soil Type	K_z (Vertical)	Unit	Embedment
S_D	177.9	Kip/feet	24
S_B	2640.251	Kip/feet	24

Table 4.3Values of horizontal soil spring for soil type Sd &Sb

SOIL TYPE	SD	SB	
Unit	Kip/feet	Kip/feet	feet
Sr. no.	Values of soil spring K_x& K_y		Height of wall
1	113.3635	1684.258	20
2	119.7981	1779.857	18
3	126.2326	1875.456	16
4	132.6672	1971.056	14
5	139.1018	2066.655	12
6	145.5364	2162.254	10
7	151.9709	2257.854	8
8	158.4055	2353.453	6
9	164.8401	2449.053	4
10	171.2746	2544.652	2
11	177.7092	2640.251	0

A schematic illustration of a building foundation with the soil spring is shown in Figure 4.3.

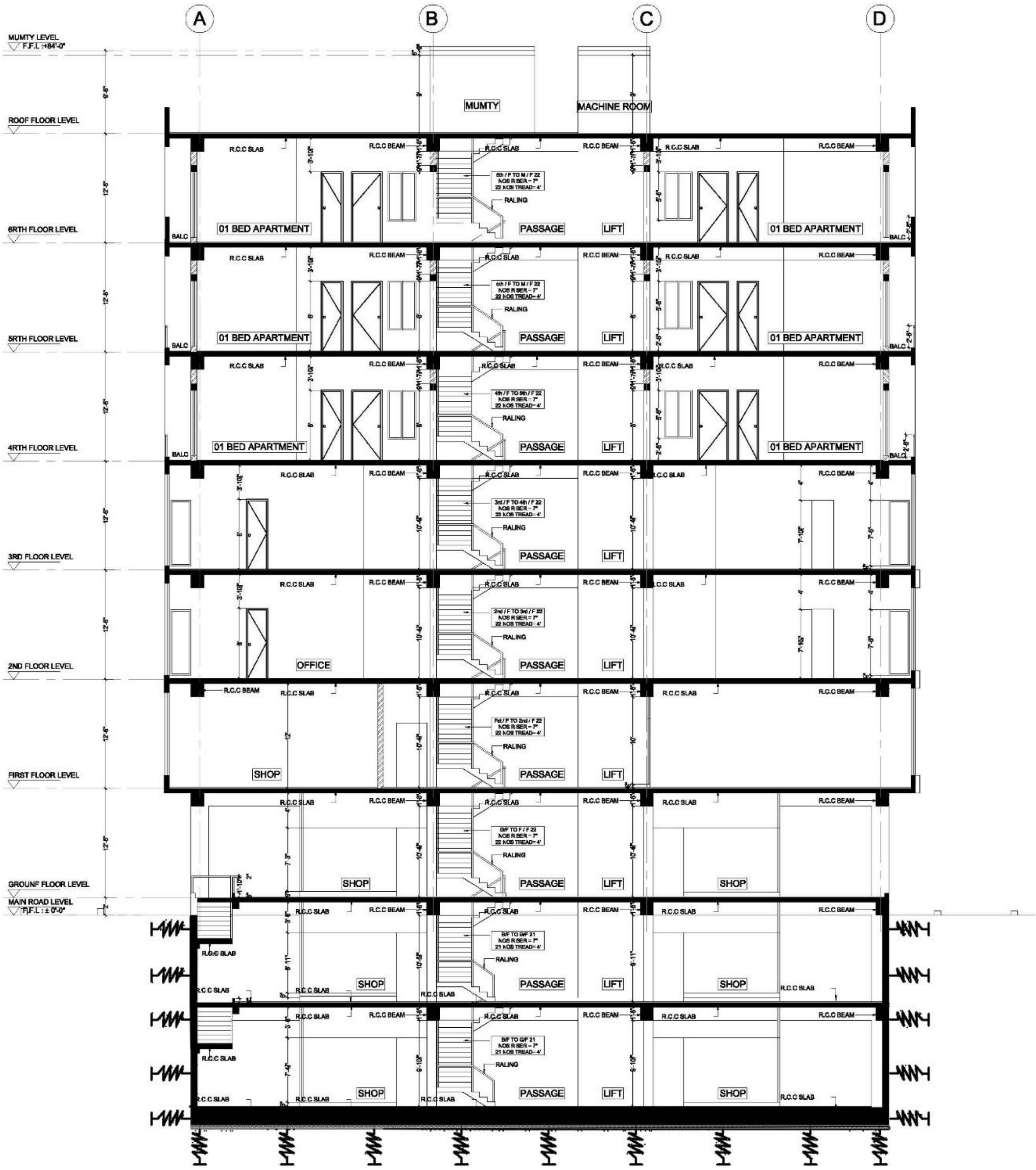


Figure 4.3 Schematic illustration of soil foundation modal

Three dimensional view and elevation of models with soil medium modelled as Winkler springs is shown in the Figure 4.4.

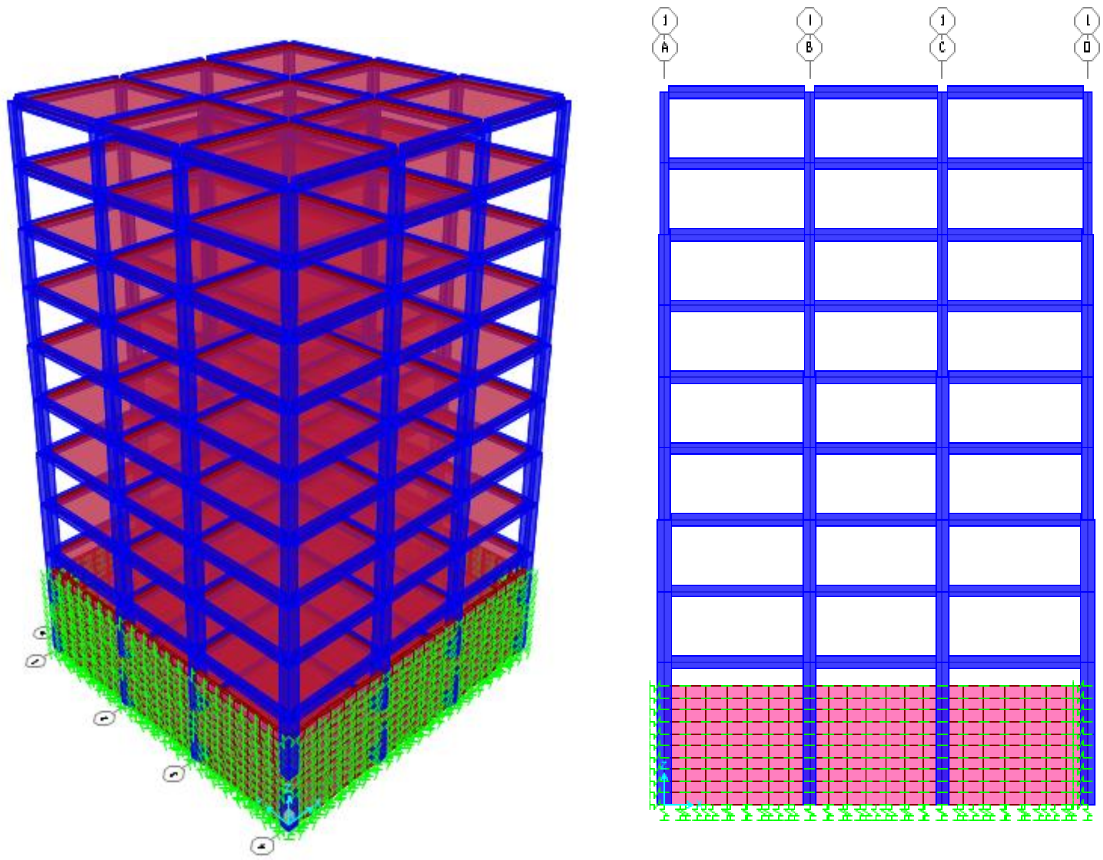


Figure 4.4 3D View & Elevationof case study building with SSI

4.5 Static Pushover analysis (PoA)

The pushover analysis is a nonlinear static method as discussed in Chapter 3. For analysis of case study building capacity spectrum method is followed. In this procedure, the structure is pushed horizontally with a defined loading pattern (First Mode Load Pattern in this case) until it reaches a predefined target displacement. Plot of the total base shear versus roof displacement is then obtained, which shows any premature failure or weakness. This plot is called capacity curve (M. Belgasmia et al, 2014). To perform the non-linear PoA, guidelines of FEMA-273 and ATC 40 regarding modelling procedure have been followed. The modal participation factor for the first mode from modal analysis was 75% i.e. first mode dominant hence; nonlinear static pushover analysis is sufficient and is used to evaluate the seismic performance of the case study structure. The numerical analysis was done by SAP-2000. Target displacement was calculated using the equation 3.1 given in FEMA 273. The values of effective time period T_e, 1st mode displacement and modal participation factors are obtained from modal analysis using cracked section stiffness's of the structural members whereas, the values of spectral acceleration S_a are taken from response spectrum against the effective time period. Target displacements for each model are shown in table 4.3. It can be seen from Table 4.4 that for stiff soil (S_D) time period has been increased by 1% with fixed base to that of SSI, whereas for Rock (S_B) time period remains almost the same. Auto hinge property integrated in SAP-2000 was assigned to beam ends of each storey and lower ends of ground floor column. The hinges are based on the X-section and reinforcement obtained from the load combinations including the gravity and seismic moments and shears from equivalent static analysis. The structure under consideration is then pushed in the X direction by defining the non-linear load case push-X which continues from nonlinear 1.2dead+Ls+0.5L case. Once the capacity curve is defined the performance of the structure can be assessed by comparing the base shear, deflection, storey drift, and stages of number of hinges formed.

Table 4.4 Values of target displacement for nonlinear static pushover analysis

Soil profile type	S_D		S_B		unit
Structural system	with fixed base	with SSI	with fixed base	With SSI	
C0	1.40	1.39	1.40	1.397	-
C1	1.00	1.00	1.00	1.000	-
C2	1.00	1.00	1.00	1.000	-
Sa	63.14	62.25	31.11	31.569	-
Time period	2.46	2.48	2.45	2.450	sec
Target displacement	13.51	13.50	6.66	6.714	in
	1.13	1.13	0.55	0.560	ft

Total eight numerical models have been investigated. Results have been compared and discussed in the following section.

4.6 Results and discussion

Based on the results of static linear and static non-linear analysis, the effect of SSI on design parameters have been compared to that of fixed base system and the values of "R" factor with and without SSI have been evaluated.

4.6.1 Evaluation of "R" factor values

Strength reduction factors are used to account for the non-linear structural behaviour as discussed earlier in Chapter2. It is the ratio between the strength required for elastic behaviour and that for which the ductility demand equals the target ductility.

$R=V_{E\text{ (elastic base-shear)}}/V_{\text{Inelastic (inelastic base shear)}}$ (Avilés and Pérez-Rocha, 2005) Equation 4.1

Table 4.5shows the modified values of "R" factor. Code based (UBC-97) value of "R" factor used in linear static and nonlinear static analysis was 5.5 for both S_D and S_B soil type. The values of "R" factor with fixed base are 3.79 & 3.1 for soil type S_D& S_B respectively, while with SSI values are 2.92 & 3.1 for soil type S_D& S_B respectively. Two important conclusions can be drawn from these results as follows:

(i) The "R" factor values used for the design are 5.5, whereas actual values of the "R" even with fixed base systems are less than the designed value by 31% and 43% for soil type S_D& S_B respectively. This may be due to strain hardening effect of the rebar, effect of vertical seismic effects added to the dead load and multiplication of 1.1 factor to all loads including seismic effects. This shows that load combinations as well as "R" factors need rationalization to avoid any determinant effects to the members which are intended to be elastic such as upper stories columns and foundations in case of moment resisting frame buildings. c

(ii) The actual values of the "R" with SSI systems are less than the designed value by 47% and 43% for soil type S_D& S_B respectively. This shows that SSI effect is prominent for softer type of soils such as "S_D" whereas, it is negligible for hard soil such as S_B. It can also be inferred that "R" factor depends not only on the natural period "T", ductility and redundancy of structure but also on the foundation flexibility measured by the shear wave velocity of soil.

Table 4.5 Values of "R" factor Code based and modified for fixed base & SSI

Soil Type/"R" Factor	Code Based "R" Factor	"R" factor Evaluated using Fixed base=($V_{elastic}/V_{Inelastic}$)	"R" factor Evaluated using SSI=$V_{elastic}/V_{Inelastic}$)
S_D	5.5	3.79	2.92
S_B	5.5	3.1	3.1

Figure 4.5 &Figure 4.6 shows the Comparison of $V_{inelastic}$, $V_{elastic}$ and V_{design} with fixed base to that of SSI for Soil type "S_D" and "S_B" respectively. For stiff soil, base shear with SSI system has increased as compared to fixed base system while for rock base shear remains same for both the SSI and fixed base system. The reason for increased base shear in case of soil type "S_D" is attributed to softening effect of soil which has reduced the number of locations in beams at all levels where plastic rotation (yielding of steel rebars) has occurred. The values of "R" factor have been reduced considerably which is in close agreement with the results of Ganjavi B. and Hao. H (2014).

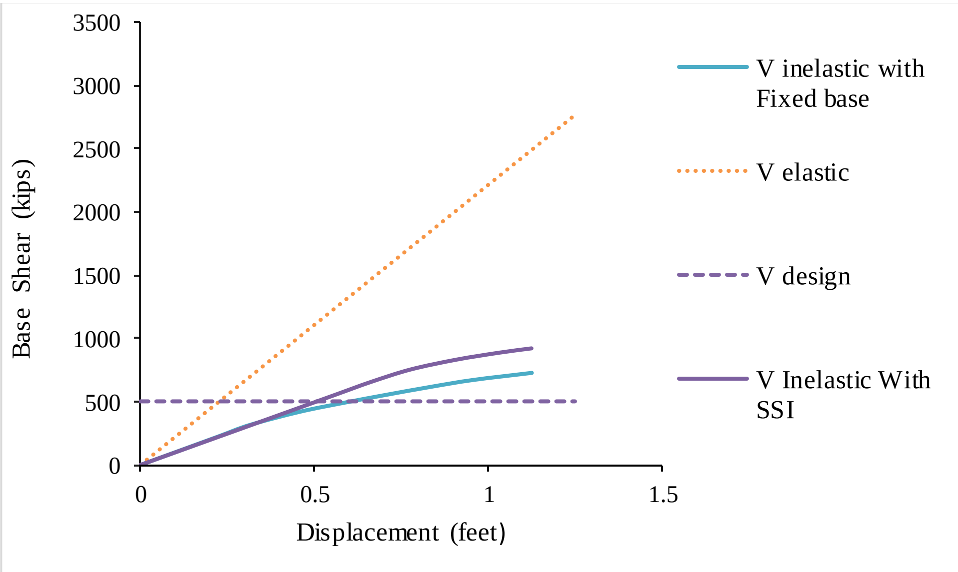


Figure 4.5 Comparison of $V_{inelastic}$, $V_{elastic}$ and V_{design} Vs displacement with fixed base to that of SSI for Soil type "S_D"

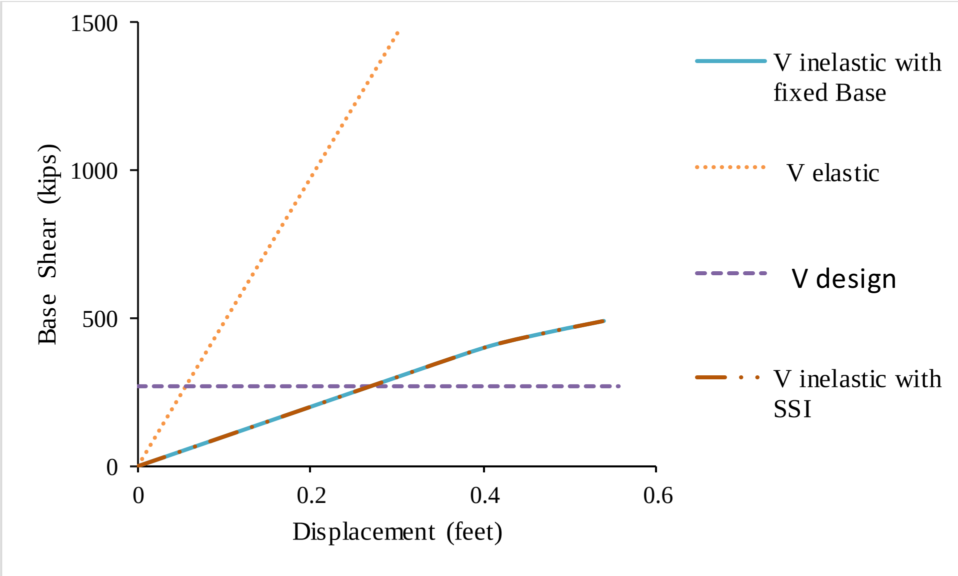


Figure 4.6 Comparison of $V_{inelastic}$, $V_{elastic}$ and V_{design} Vs displacement with fixed base to that of SSI for Soil type"S_B"

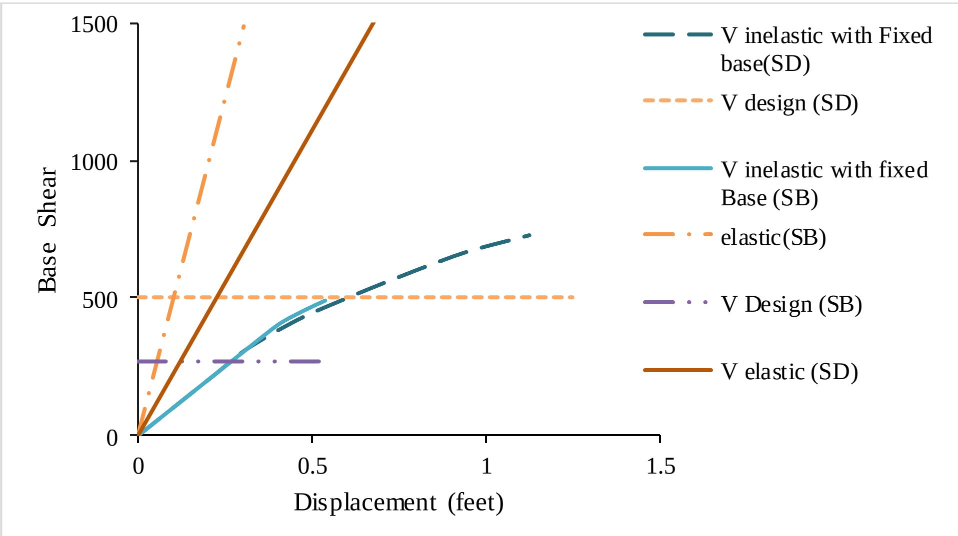


Figure 4.7 Comparison of $V_{inelastic}$, $V_{elastic}$ and V_{design} Vs displacement with soil type "S_D to that of "S_B" with Fixed base

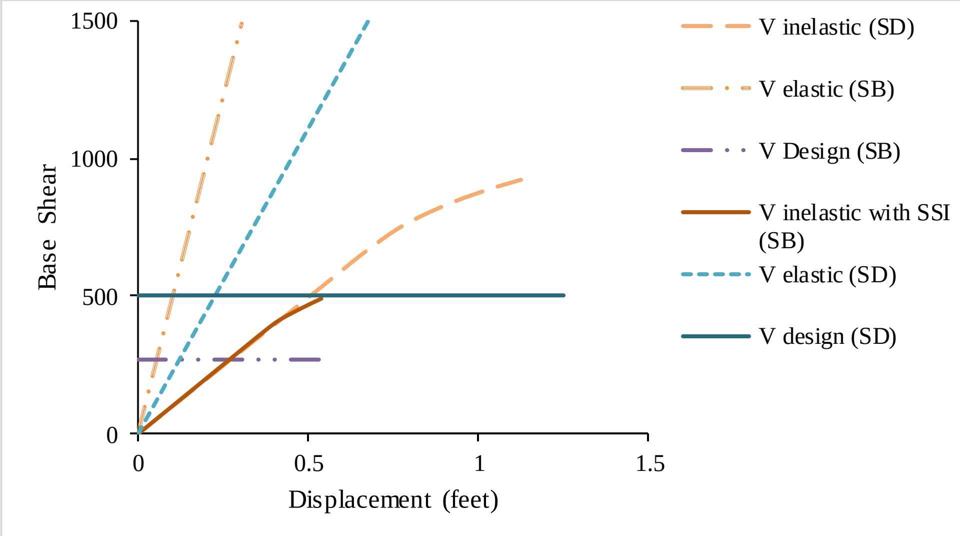


Figure 4.8 Comparison of $V_{inelastic}$, $V_{elastic}$ and V_{design} Vs displacement with soil type “S_D”to that of “S_B”with SSI

It can be observed from the Figure 4.5 & 4.6 that $V_{inelastic}$ for “S_D” with SSI has increased by 34.2% as compared to that of fixed base, while in case of S_B $V_{inelastic}$ is approximately same for both fixed base and SSI systems. Figure 4.7 & 4.8 shows comparison of $V_{inelastic}$, $V_{elastic}$ and V_{design} Vs displacement with soil type S_D to that of S_B with fixed base and SSI systems respectively. For soil type S_D $V_{Inelastic}$ is much more as compared to that of S_B with both fixed base and SSI cases. It can be inferred that variation in base shear, between the conventional design practice (fixed base) and SSI increases with increase in flexibility of underlying soil as can be seen from Figure 4.5, 4.6, 4.7 and 4.8.

Using Code based R factor in structural analysis may lead to the following effects:

- Chance of shear Failure in beams and columns before flexural failure.
- Development of plastic hinges in upper storey columns, these phenomena may lead to soft storey effects.
- Soil Pressure may increase.
- Reinforcement in foundation may also increase.

4.6.2 Storey shear

Sum of all the design lateral forces at each level above the storey under consideration is called storey shear. Representative values of storey shear for linear equivalent static case (QX) and non-linear static (Push-x) case for both soil types are shown in Figures 4.9 & 4.10.

From Figures- 4.9 & 4.10, it is evident that there is significant variation in values of storey shear for both cases for soil type "S_D" (Fixed base & SSI). The elastic storey shear for both fixed base & SSI system is obviously same due to limitation on the code based period and subsequently storey shears, whereas inelastic storey shear for SSI has been increased by 24%. There is 5% difference between storey shears for soil type "S_B" in case of fixed and SSI system.

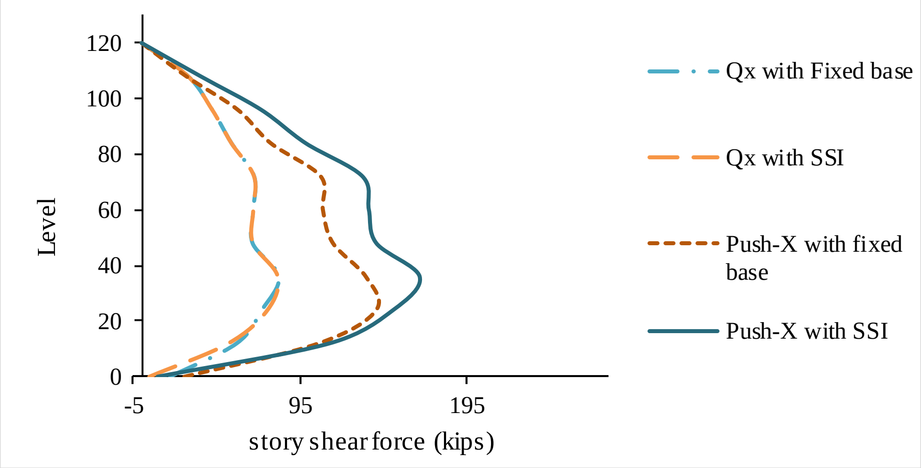


Figure 4.9 Storey shear comparison for soil type S_D

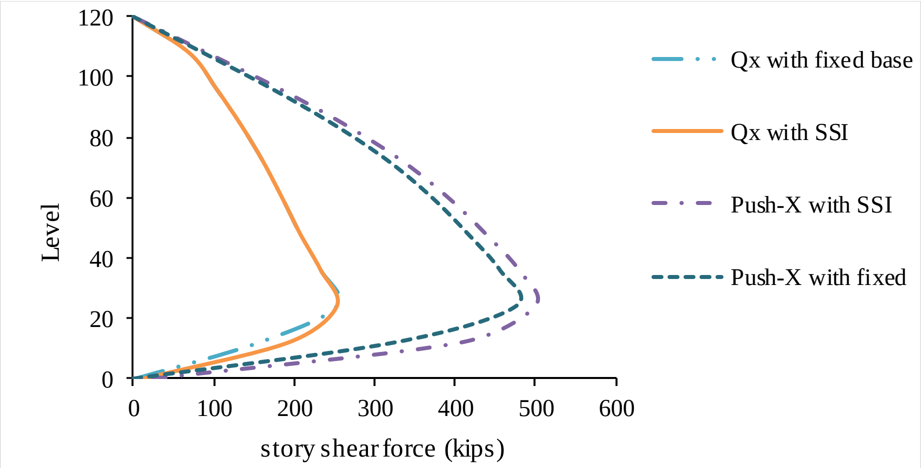


Figure 4.10 Storey shear comparison for soil

4.6.3 Overturning Moment

The inertial force created by an earthquake lateral force acts through the centre of mass of a building; there is a tendency of structure to overturn above the base. The overturning moments with SSI system having S_D soil type varies significantly at the lower storey and top storey with SSI system has increased as compared to fixed base for both linear and non-linear analysis. This variation of moments shows the SSI effects. Whereas, for soil type S_B with SSI system moments at lower storey have increased while, it is same at top storey.

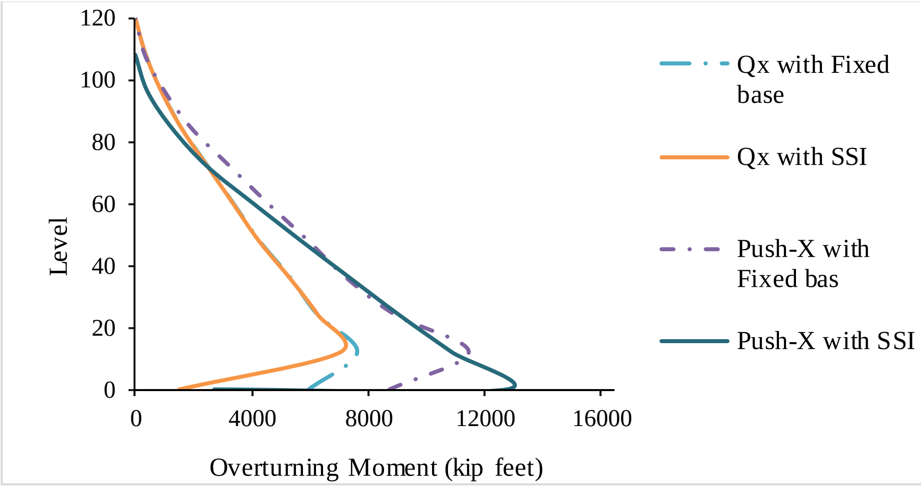


Figure 4.11 Comparison of overturning moment for soil type S_D

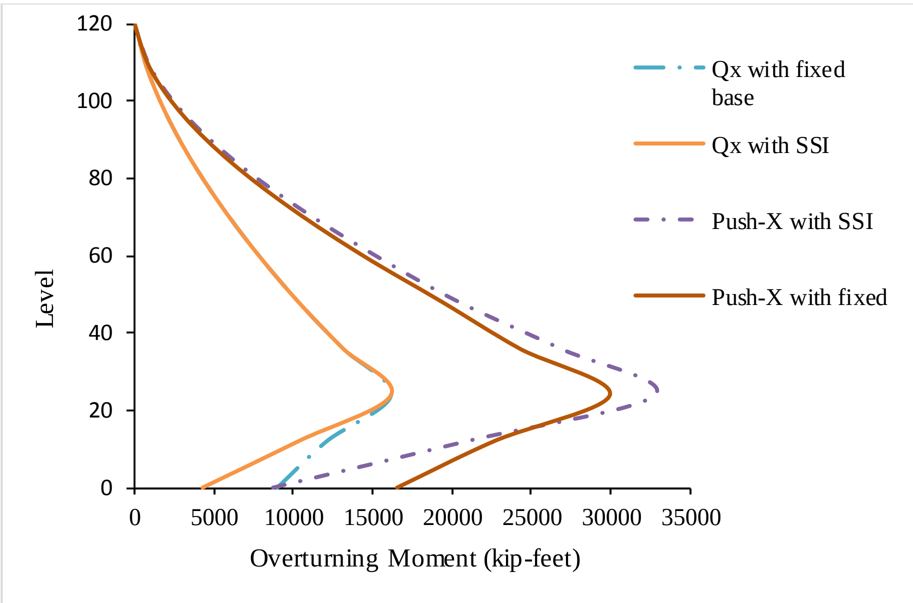


Figure4.12Comparison of overturning moment for soil type S_B

4.6.4 Storey drift

Storey drift is the ratio of difference between displacement of one storey and the storey immediately below to the storey height under the lateral forces. SSI can amplify the storey drifts and lateral displacements significantly especially for the structured placed on soft soils with vs<300m/s. Figure 4.13 shows that for soil S_D there is little difference between storey drifts of fixed base and flexible base systems. For soil type S_D storey drift with SSI for linear static analysis (QX) has been increased by 2.73%. In Push-X case the trend is varying, storey drift with SSI has increased by 19% at lower level as compared to that of fixed base system, whereas top storey drift with SSI has decreased by 8.5%. For Soil type S_B, storey drift is same for both the SSI and fixed base systems. This is because of the rigidity of the rock type soil i.e. low rotation of foundation.

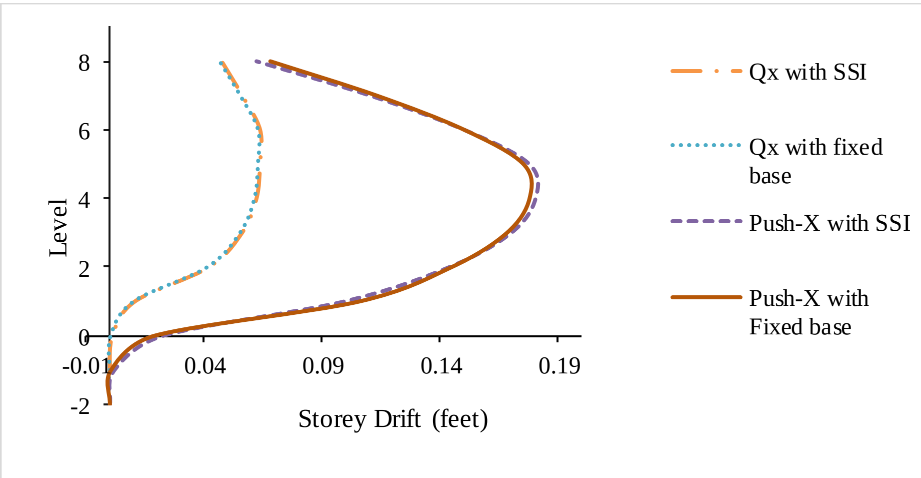


Figure 4.13 comparison of storey drift for soil type S_D

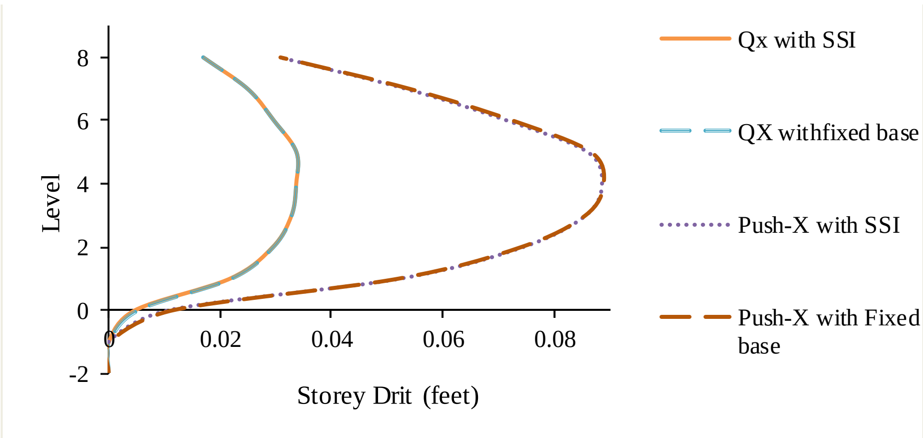


Figure 4.14 comparison of storey drift for soil type S_B

4.6.5 Storey displacement

Figure 4.15 & 4.16 shows the storey displacement over the height of building for soil type S_D and S_B. Storey displacement for soil S_D with SSI for linear static and nonlinear pushover analysis has been increased by 1.94% and 0.3% respectively. Whereas, for soil type S_B there is negligible difference in the response of SSI and fixed base structural systems. The reason for the negligible difference is mainly because of the fact that the structure is pushed to the same target displacement with almost same load pattern.

These results are in accordance with literature review. Massumi and Tabatabaiefar, (2008) also concluded that for soils with vs> 375m/s story displacements are same. Although there is minor amplification of storey displacements in soil S_D. This is due to the fact that both the soils S_D and S_{B} are have shear Vs>300. We can conclude that structures with Soil S_B shows fixed base behaviour.

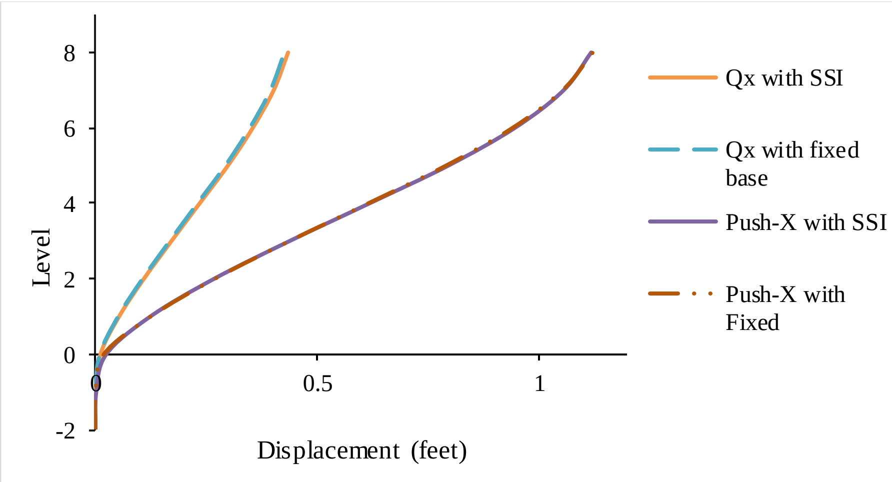


Figure 4.15 comparison of storey displacement for soil type S_D

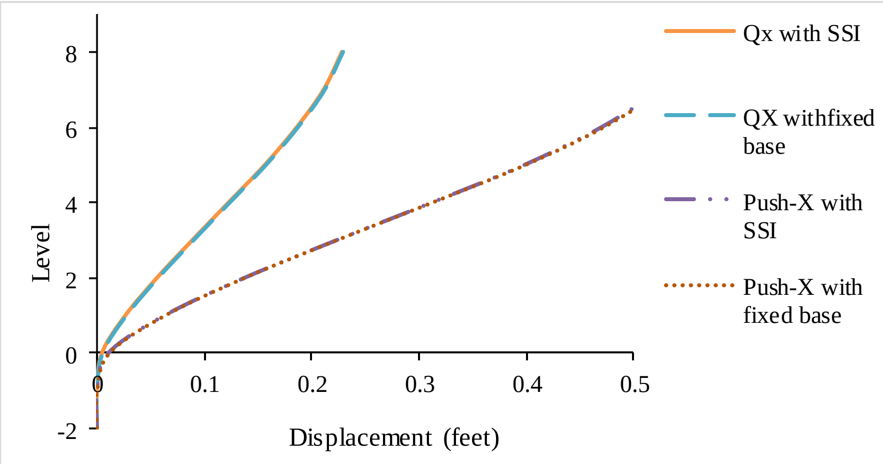


Figure 4.16 Comparison of storey displacements for soil Type S_B

4.6.6 Formation of Plastic hinges in nonlinear static pushover analysis

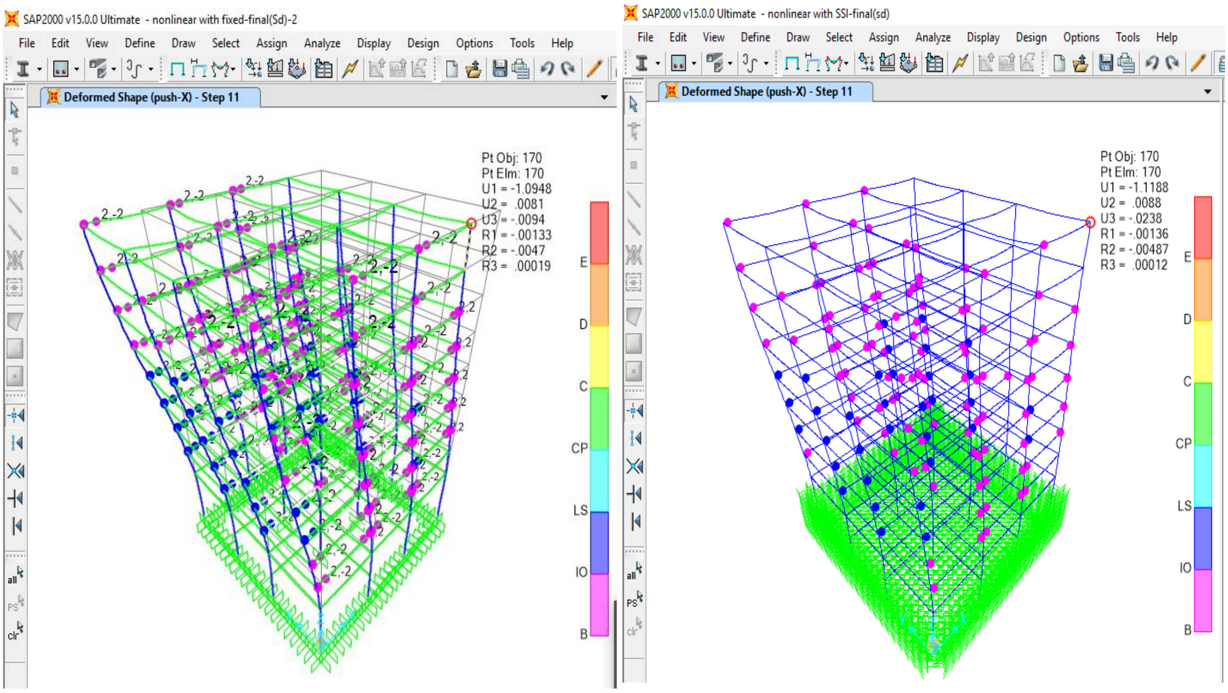


Figure 4.17 State of hinges for soil typeSD in (a) SSI models and (b) Fixed Base

Figure 4.18 Position of Hinge 275H1 at grid 2 & A horizontal & vertical respectively.at 5th floor

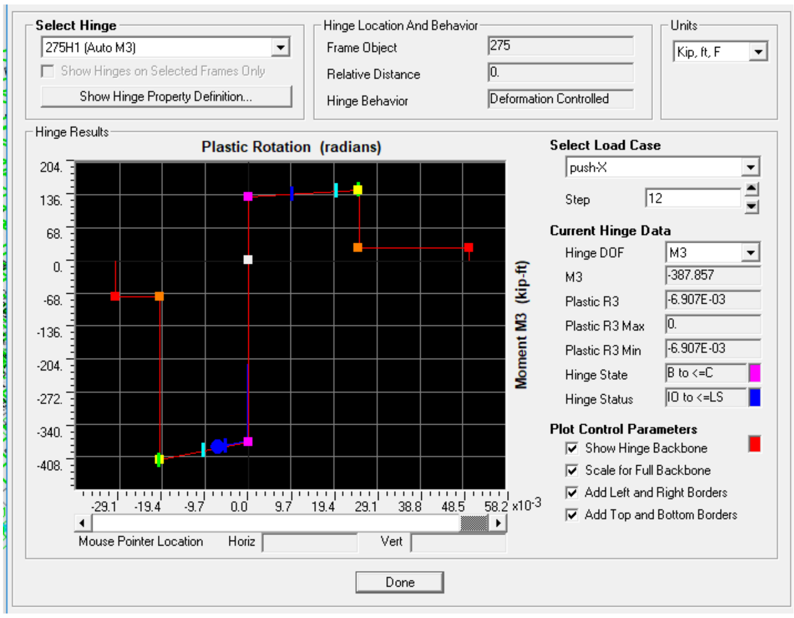


Figure 4.19 State of hinge 275H1 for Fixed base

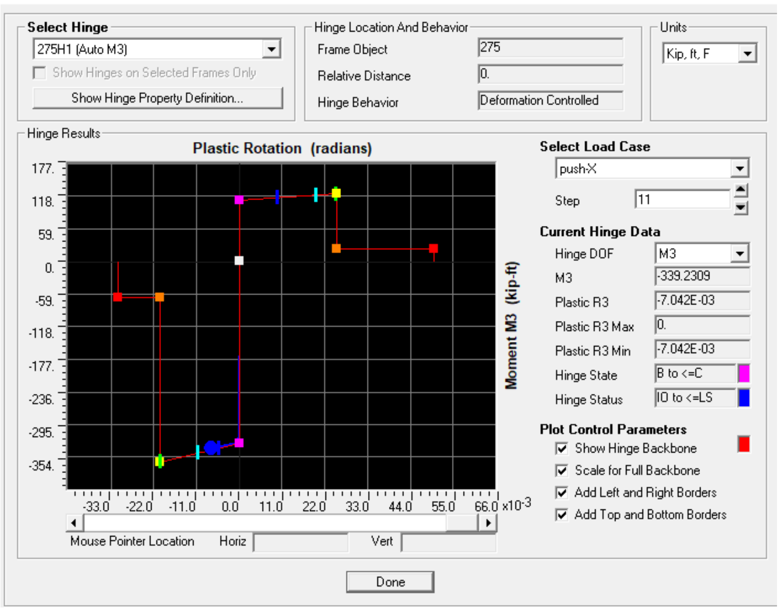


Figure 4.20 State of hinge 275H1 for SSI

Figure 4.17 shows the formation of plastic hinges in nonlinear static pushover analysis for fixed base and SSI systems for the soil type “S_D”. It can be seen that hinges have formed in beams at almost all level for fixed base as well as SSI system. Some of the hinges yield up to the life safety region, whereas some hinges have been yielded up to immediate occupancy as per criteria set in FEMA, 356, Table 6-7. Column Hinges remained elastic for both cases. One representative hinge is shown in the Figure-4.19 and 4.20 for fixed base and SSI system respectively. The plastic hinge rotation for fixed base system is 0.0069radian, while it is 0.007radian for SSI system.

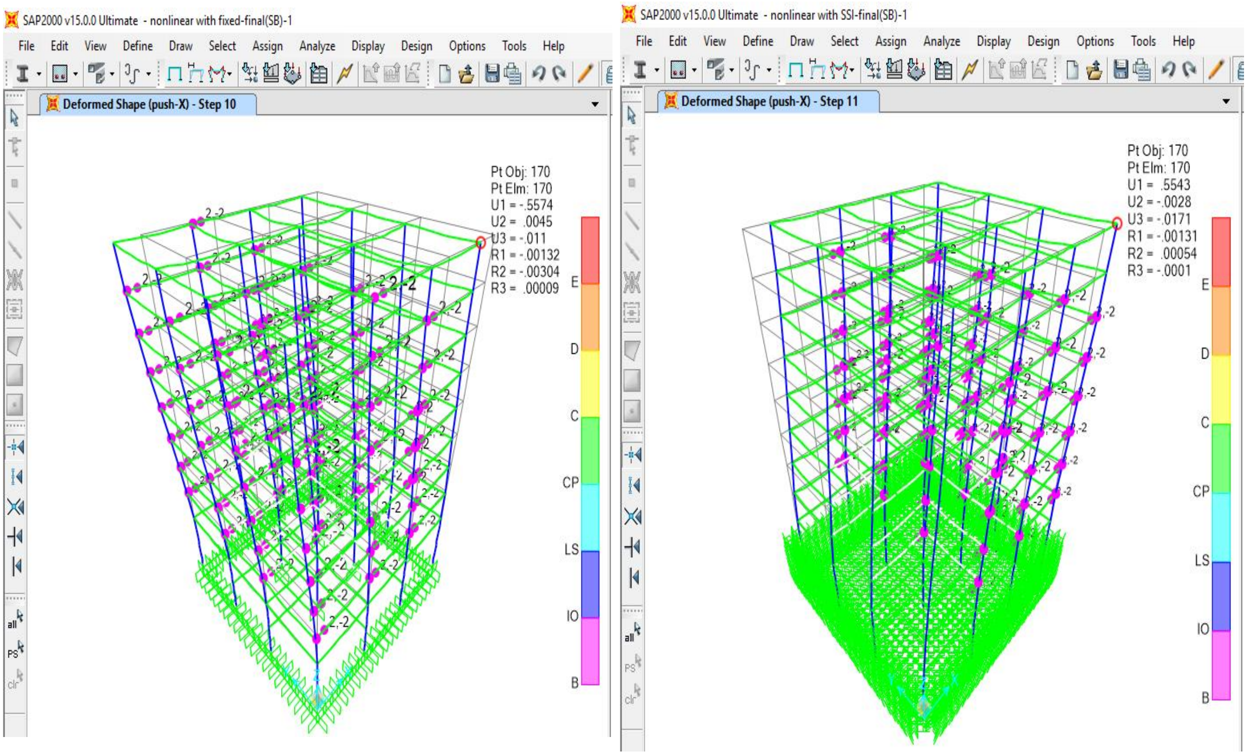


Figure4.21 Push-X State of hinges for soil type Sb in (a) Fixed Base and (b) SSI models

Figure 4.22 Position of Hinge 275H1 at grid 2 & A horizontal & vertical respectively at 5^{th} floor

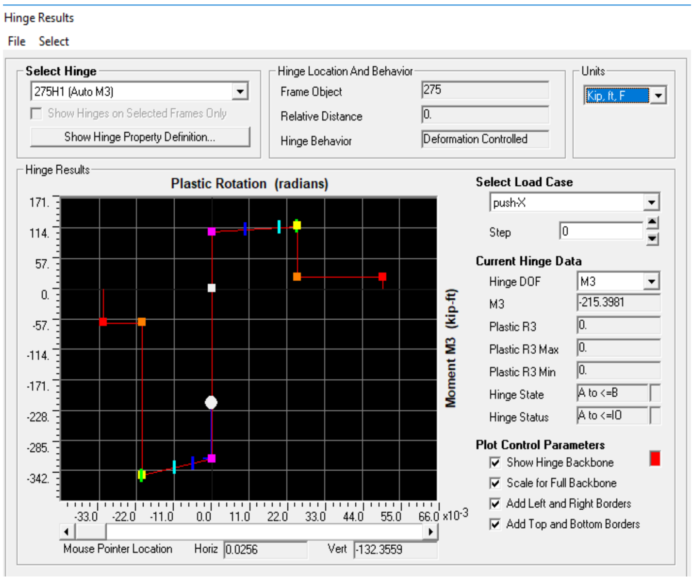


Figure 4.23 State of hinge 275H1 for Fixed base system

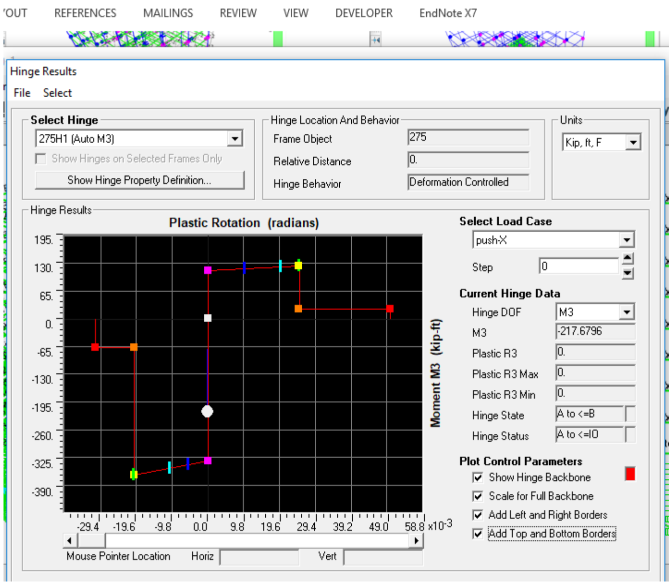


Figure 4.24 State of hinge 275H1 for flexible base system

Figure 4.21 shows the formation of plastic hinges for both the fixed base and SSI system for soil type "S_B". Comparison of hinges shows that in both systems the plastic hinges are formed and plastic rotation reached up to immediate occupancy level as per criteria set in FEMA, 356, Table 6.7. The numbers of the locations where plastic hinges are formed as well their plastic hinge rotation are less than those for soil type "S_D". This explains the reason for low "R" factor values in case of soil type "S_B". Figure 4.23 & 4.24 shows the plastic hinge rotation for both the fixed and SSI system for one specific hinge which is same as shown in Figure-4.19 and 4.20 for soil type “S_D”. This hinge did not go beyond the elastic limit for soil type “S_B”.

4.7 Summary

A 10 storey case study building has been investigated with and with SSI for two soil types S_D and S_B. Soil medium has been modelled following wrinkle’s approach. Linear static and non-linear static pushover analysis has been performed to ascertain the actual “R” factor values. Different parameters e.g. storey shear, storey displacement, storey drift, overturning moments and plastic hinge states has been compared with SSI to that of fixed base system. In general results show that SSI effects are prominent for S_D soils however, its effect negligible in case of “S_B” $_{soil}$ for almost all parameters.

CHAPTER 5

CONCLUSIONS AND RECOMMENDATIONS

5.1 Summary

The importance of soil structure interaction (SSI) for seismic design of moment resisting frame building has been discussed and the related literature review to solve soil–structure interaction problems has been presented. Since 1990s, great effort has been made for substituting the classical methods of earthquake design by the new ones. In both the equivalent linear static force and code based response spectrum procedure, the seismic design philosophy is incorporated using response modification factor "R" (IBC-2012, UBC-97, FEMA-451and BCP-2007). "R" factor accounts for over strength, ductility and system capacity to redistribute forces from inelastic high strength regions to less stressed regions within the structure. The ratio of elastic strength demand to inelastic strength design is defined as "R" factor. During earthquake, the response of the structure is created due to the three-interlinked systems i.e., the super structure, the foundation and the soil medium. This phenomenon is called soil structure interaction (FEMA, 451). In conventional seismic design philosophy, the structure seismic structural analysis is performed assuming that the structure is fixed at the foundation level (rigid support). However, in actual the structure has foundation flexibility depending upon the type of soil medium supporting the structure. Code based values of "R" factor does not reflect the SSI problem. Thus, there is a strong need to redefine the R factor values considering the effect of SSI.

In this study a 10 storied mid-rise moment resisting frame building situated in earthquake zone 2B, with two different soil types S_D and S_B having shear wave velocity 300 m/s and 1200 m/s respectively, has been discussed. Soil medium has been modelled based on Wrinkler's approach. Values of soil spring have been calculated by formulas proposed by Gazetas, (1991). To ascertain the actual value of "R" factor equivalent linear static and non-linear static pushover analysis have been performed while for the code based design "R" factor value from UBC-97 have been used. Furthermore, the response of different parameters i.e. storey shear, moment, displacement and drift have been compared for above mentioned cases.

5.2 Conclusions:

Following important conclusions have been drawn from this study:

- The time period for soil type SD with SSI has been elongated by 0.02 seconds only as compared to that of fixed base, while it is approximately same for soil type SB for both systems i.e. fixed base and flexible base.
- The modified values of "R" factor with fixed base for soil type S_D and S_B are 3.79 and 3.1 respectively, while with flexible base "R" factor values for soil type S_D and S_B are 2.92 and 3.1 respectively.
- The "R" factor values used for the design are 5.5, whereas actual values of the "R" factor even with fixed base systems are less than the designed value by 31% and 43% for soil type S_D& S_B respectively. This may be due to strain hardening effect of the rebar, effect of vertical seismic effects added to the dead load and multiplication of 1.1 factor to all loads including seismic effects. This shows that load combinations as well as "R" factors need rationalization to avoid any determinant effects to the members which are intended to be elastic such as upper stories columns and foundations in case of moment resisting frame buildings.
- The actual value of the "R" with SSI systems for both types of soil is less than the designed value by 47% and 43% for soil type S_D& S_B respectively. This shows that SSI effect is prominent for softer type of soils such as "S_D", whereas it is negligible for hard soil such as S_B. It can also be inferred that "R" factor depends not only on the natural period "T", ductility and redundancy of structure but also on the foundation flexibility measured by the shear wave velocity of soil.
- The inelastic base shear with SSI system has increased by 34.2% as compared to that of fixed base system for "S_D" soil type whereas for "S_B" the base shears for both the systems are same.
- The inelastic storey shear by non-linear static force procedure with SSI for soil type S_D has increased by 24% as compared to that of fixed base system, whereas, for soil type S_B the same has increased by 5.25% only. The story shear by linear static force procedure is approximately same for both fixed and flexible systems considering S_D and S_B soils.

- The overturning moments at ground level has been increased by 11.35% with SSI system as compared to that of fixed base system for soil type S_D, whereas, for soil type $S_{B,}$ the same has been increased by 8.68%. The story shear by linear static force procedure is approximately same for both fixed and flexible systems considering S_D and S_B soils
- The storey drift and displacement for S_D with SSI system is approximately same as compared to that of fixed whereas for S_B story drift and displacement are exactly same for both SSI and fixed base system.
- Plastic hinges have formed in beams at almost all level for fixed base as well as SSI system for soil type "S_D". Some of the hinges have yielded up to the life safety region, whereas some hinges have been yielded up to immediate occupancy as per criteria set in FEMA, 356, Table 6-7. Column Hinges remained elastic for both cases.
- The plastic hinges are formed and plastic rotation reached up to immediate occupancy level as per criteria set in FEMA, 356, Table 6.7 for soil type "S_B". The number of the locations where plastic hinges are formed as well as their plastic hinge rotations is less than those for soil type "S_D". This explains the reason for low "R" factor values in case of soil type "S_B".
- Code based "R" factor in structural analysis may lead to soft storey effect due to formation of plastic hinges in upper storey columns. Plastic hinges in both SSI and Fixed base models have yielded to same level but there is chance of shear failure in beams and columns before flexural failure. Soil pressure and reinforcement in foundation may increase. But these effects have not been studied in detail in this research work, which shall be studied in PhD Studies.

It can be concluded that the ignoring SSI effects for the structural analysis may not assure structural safety of regular midrise moment resisting building frames placed on soil deposits with shear wave velocity less than and equal to 300m/s, while for the structures resting on S_B soil deposits, due to the high stiffness of underlying soil medium structural system is behaving as fixed base system. Soil with shear wave velocity less than 1200m/s and greater than 300 m/s has not been investigated in this study. Structures placed on these soils may or may not be affected by SSI, which shall be studied in PhD studies.

5.3 Limitation of the study

The limitations of this study are:

1. Only the numerical modelling and analysis have been performed.
2. Only two types of soil S_D and S_B with v_s 300 m/s and 1200 m/s respectively, have been considered.
3. Only linear equivalent static (ESA) analysis and non-linear Static pushover analysis (non-linear) have been performed.
4. The effects of non-structural components such as Infill Walls have not been taken into account.

5.4 Future Recommendations

The aim of this study was to evaluate the "R" factor in consideration of SSI for regular mid-rise moment resisting frame buildings using equivalent static analysis and pushover analysis. In this study only two soil type stiff clay and rock have been considered. The effect of non-structural elements such as infill walls on the time period in equivalent static analysis has been used implicitly by using code based method of determination of periods. Future research work shall be carried out in the following area:

Evaluate the "R" factor values considering SSI with the variation of soil types, soil non-linearity and effects of foundation embedment depth for mid-rise as well as high rise buildings. The effect of non-structural elements such as infill walls on the time period shall also be considered using more rigorous non-linear response history analysis.

All these recommendations shall be incorporated in PhD studies.

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APPENDICES

Annexure I

Sr. No.	Internal Examiner's Comments	Response
01	Geo Technical properties of the soil have not been investigated experimentally.	Geo Technical properties were taken from the FEMA-2000 document for stiff (S_B) and hard (S_D) soil.
02	Why soft soil has not been considered?	Due to low bearing capacity of soft soils, pile foundation may be required to transfer the structural load to the hard strata below, which is not the scope of this study. In PhD studies Different soil types including soft soils as well as soil nonlinearity shall be studied.
03	If structural foundation encounter ground water table (GWT), what will be the effect on foundation and structural stability.	In this research study regular midrise moment resisting frame considering two soil types S_D and S_B (assuming that GWT is not encountered) has been investigated. The mentioned phenomena shall be investigated in PhD thesis.
04	Format and Font Corrections are required throughout the thesis.	Necessary corrections have been incorporated in this thesis.
05	Grammatical Corrections are also required throughout the thesis.	Necessary corrections have been incorporated in this thesis.

Sr. No.	External Examiner's Comments	Response
01	Format and Font Corrections are required throughout the thesis.	Necessary corrections have been in corporate in this thesis.
02	Figure Quality corrections.	Necessary corrections have been incorporated in this thesis.

03	Table 3.4 needs to be corrected.	The mentioned table format has been corrected.
04	What will be the effect of modified "R" factor value on structural reinforcement?	Code based "R" factor in structural analysis may lead to soft storey effect due to formation of plastic hinges in upper storey columns. Plastic hinges in both SSI and Fixed base models have yielded to same level but there is chance of shear failure in beams and columns before flexural failure. Soil pressure and reinforcement in foundation may increase. But these effects have not been studied in detail in this research work and shall be studied in PhD Studies.