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İki Boyutlu Çelik Çerçevelerin Sismik Yükler Altında Zemin Yapı Etkileşimli Analizi

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Soil Structure Interaction Analysis of Two Dimensional Steel Frames under Seismic Loads

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Abstract

In this research work, a linear and nonlinear time history analysis for 2D multi-story steel frames on gravel soil with and without soil structure interactions have been done. Frames have different height start from 3 stories and up to 20 stories. A substructure method is used to model soil and shallow (footing) foundation system by linear springs and dashpots. The response spectrum analysis has also been done. Düzce earthquake accelerograms records are selected and scaled to specific response spectrum with PGA 0.35 and soil type B according to Euro code 8 is considered. Fundamental period and base shears have been investigated with and without soil-structure interactions; the results have been presented and compared.

***Keywords*:** Steel frames; soil structure interaction; response spectrum analysis; time history analysis; seismic loads; fundamental period

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Özet

Bu araştırmada, zemin yapısı etkileşimli ve etkileşimsiz, 2 boyutlu çok katlı çelik çerçeveler için doğrusal ve doğrusal olmayan zaman adım analizi yapılmıştır. Çerçeveler 3 kattan 20 katına kadar farklı yüksekliklere sahiptir. Zemin ve tekil temel sistemi doğrusal yaylar ve sönümleyici ile modellemek için alt yapı analiz metodu kullanılmıştır. Ayrıca tepki-spektrum analizi yapılmıştır. Düzce deprem ivme kaydı seçilerek spesifik tepki spektrumuna ölçeklendirildi PGA 0.35 ve Euro code 8'e göre zemin tipi B (katı) seçildi. Temel periyot ve zemin kesmesi, zemin-yapı etkileşimi dahil edilerek ve edilmeyerek araştırılmış ve sonuçlar karşılaştırılmıştır.

Anahtar kelimeler: Çelik çerçeve; zemin yapı etkileşimi; tepki sepktrum analizi; zaman adım analizi; sismik yük; temel periyot.

1. Introduction

The seismic design of structures has advanced steadily by deepening of theoretical foundations. The significant progress in field of seismic design in last four decades is a result of parallel progress in structural analysis and computer hardware. Nowadays, time history analyses has become popular in seismic analyses and design. The accuracy of these analyses depends on many factors. One of the most important points in structure modeling is how to model foundation. This is because a lot of high rise buildings which are built in many seismic zones and it is very important to understand their behavior under seismic loads and model buildings with a more realistic view to keep people in safe. Normally bases in seismic analysis of high rise buildings are considered fixed without any take into account foundation system or soil beneath the building. But according to the recent earthquake, the effect of soil and foundation system is very clear on the behavior of buildings, huge efforts are directed to simulate Soil Structure Interaction (SSI) and study their effect on buildings response. Wolf [1] presented foundation on deformable soil by spring-dashpot-mass model with frequency-independent coefficients. Mulliken and Karabalis [2] presented discrete model for predicting the dynamic through the soil interaction between adjacent rigid, surface foundations supported by a homogeneous, isotropic and linear elastic half-space. Minasidis et al. [3] reviewed the studies on SSI in steel frames subjected to near-fault earthquakes.

One of most popular approach to model SSI is the substructure method which divides the structure into two parts, superstructure and substructure. The part over grade level is called superstructure while the part under grade level is called substructure. Normally the substructure part is modeled by a system of springs and dashpots in many directions. Those springs and dashpots could be linear or nonlinear, connect not connected. This system also may consider the single degree of freedom or multi-degree of freedom [4,5].

The objective of this study is to investigate 12 steel frames planner response under seismic loads using linear and nonlinear time history analysis, and response spectrum analysis with and without soil-structure interaction. P-Delta effect is also considered. In present study, seismic records are scaled to response spectrum with Peak Ground Acceleration (PGA)=0.35g and soil type B (stiff soil) according to Euro code 8 [6]. Fundamental period and base shear, will be investigate for each frame. The steps for current research are constructed an appropriate mathematical model of the building, selection of a target spectrum for analysis, selection and scaling of earthquake ground motions consistent with the target spectrum, performing analysis and discussion of the results [7].

2. Seismic Analysis and Soil-Structure Interaction Modeling

2.1. Frames Modelling

Steel frames which have been designed according to Euro code 3 [8] and Euro code 8 [6] by Karavasilis et al [9] are considered in the present study. These frames have 3 or 6 bays, and 3, 6, 9, 12, 15 and 20 stories. The length of each bay and the height of each story are 5 and 3 m, respectively. Special moment resistant frames are designed assuming PGA=0.35g with soil type B. Dead and live loads on Loads on beams are assumed to be equal to 27.5kN/m. The yield stress of the steel is equal to 235 MPa.

Beams and columns are connected together by rigid joints without the effect of panel zone. Superstructure viscous damping ratio is assumed to be $\xi = 3\%$ for the natural modes of the system. Beam axial forces are assumed to be zero since all floors are considered to be rigid in a plan to account the diaphragm action of slabs. P-Delta effect is calculated by using dead loads only. Figure1 shows special moment resistant frames with 3 bays and 3 stories. Used section of all studied frames are listed in Table 1 [3]. The plastic hinges are assumed at the ends of the beams [10]. The hinge proprieties are considered flexure type and calculated automatically according to ASCE [11] and acceptance criteria by according to Euro code 8 using ETABS 2016 pro.

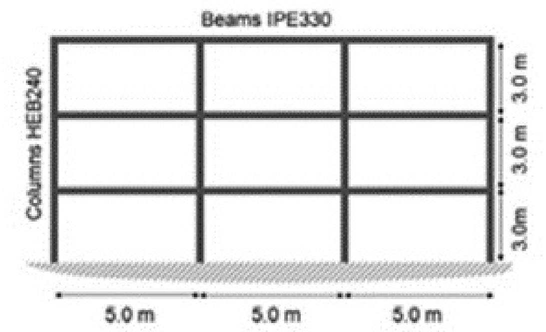

Figure 1 Typical SMRF with 3 bays and 3 stories [3].

Table 1 Designed section for special moment resistant frames considered in the study [3].

NS[1]	NB[2]	SECTIONS
3	3	240–330(1–3)
3	6	240–330(1–3)
6	3	280–360(1–4), 260–330(5–6)
6	6	280–360(1–4), 260–330(5–6)
9	3	340–360(1), 340–400(2–5), 320–360(6–7), 300–330(8–9)
9	6	340–360(1), 340–400(2–5), 320–360(6–7), 300–330(8–9)
12	3	400–360(1), 400–400(2–3), 400–450(4–5), 360–400(6–7), 340–400(8–9), 340–360(10), 340–330(11–12)
12	6	400–360(1), 400–400(2–3), 400–450(4–5), 360–400(6–7), 340–400(8–9), 340–360(10), 340–330(11–12)
15	3	500–300(1), 500–400(2–3), 500–450(4–5), 450–400(6–7), 400–400(8–12), 400–360–(13–14), 400–330(15)
15	6	500–300(1), 500–400(2–3), 500–450(4–5), 450–400(6–7), 400–400(8–12), 400–360–(13–14), 400–330(15)
20	3	600–300(1), 600–400(2–3), 600–450(4–5), 550–450(6–10), 500–450(11–13), 500–400(14–16), 450–400(17), 450–360(18–19), 450–330(20)

[1] NS is number of story, [2] NB is number of bay.

2.2. Soil Structure Interaction Modelling

Rigid base model and SSI model are used for analysis in current study. Figure 2 illustrated rigid base model presented in FEMA 440 [12]. SSI is modeled by using substructure method [2]. Foundation system is considered single footing. Foundation and soil around are modeled by springs and dashpots as shown in Figure 3. Springs have stiffness K, and dashpots have a coefficient, C. Relationship for spring and dashpot parameter are presented in Table 2. Description of soil parameter and assumed values are presented in Table 3.

Table 2 Mass, stiffness and damping coefficient for foundation structure interaction system [2].

Direction	β^1	${r_0}^2$	${m_v}^3$	K^4	C^5
Vertical	$\frac{(1-\nu)}{4}\frac{m}{\rho r_0^3}$	$\frac{2a}{\sqrt{\pi}}$	$\frac{0.27m}{\beta}$	$\frac{4.7Ga}{1-\mu}$	$\frac{0.8a}{V_s}K$
Horizontal	$\frac{(7-8\mu)}{32(1-\mu)}\frac{m}{\rho r_0^3}$	$\frac{2a}{\sqrt{\pi}}$	$\frac{0.095m}{\beta}$	$\frac{9.2Ga}{2-\mu}$	$\frac{0.163a}{V_s}K$
Rocking	$\frac{3(1-\mu)}{8}\frac{m}{\rho r_0^3}$	$\frac{2a}{\sqrt[4]{3\pi}}$	$\frac{0.24m}{\beta}$	$\frac{4.0Ga^3}{1-\mu}$	$\frac{0.6a}{V_s}K$

[1] β is mass (inertia) ratio,[2] r_0 is equivalent radius, [3] m_v is mass (inertia), [4] K is static stiffness, [5] C is damping

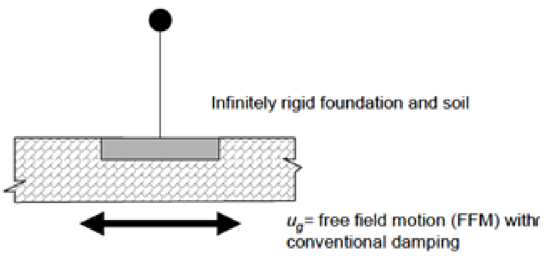

Figure 2 Rigid base model [12]

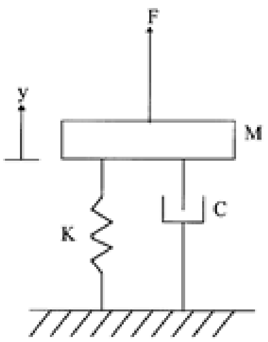

Figure 3 Basic one dimensional soil foundation model [2]

Dimensions of each square foundation are listed in Table 4, the shear wave velocity is considered about 380 m/s, which corresponds to the lower limit for soil type B according to Euro code 8. Other soil specification are presented in Table 3. Computed values of springs, dashpots coefficients and soil mass for each frame used in SSI models are presented in Table 5.

Table 3 Parameter description of foundation structure interaction system [2].

Soil parameter	Description	Assumed value
G_0	Shear modulus of soil	260 MPa
G_s	Secant shear modulus	0.1G0=26 MPa
ρ	Mass density of the soil medium	2000 kg/m^3
V_s	Shear velocity in soil	$\sqrt{G/\rho}$ = 380 m/s^2
μ	Passion ratio of the soil	0.25
q	Bearing capacity of the soil	700 kPa

Table 4 The dimension of a single footing system used in each frame.

NS[1]	3	6	9	12	15	20
$a \times h$ [2]	1.2×1.0	1.7×1.0	1.85×1.00	2.0×1.0	2.5×1.25	2.65×1.25

[1] NS is number of story,[2] a is length (square foundation) and h is depth

2.3. Seismic Loads Modelling

Seismic loads are calculated and analyzed according to Euro code 8. A response spectrum analysis is done with PGA 0.35, soil type B which could be gravel or deposit of very dense sand, importance factor of frames is II and behavior factor equal 3. The response spectrum curve based on mentioned parameters is presented in Figure 4.

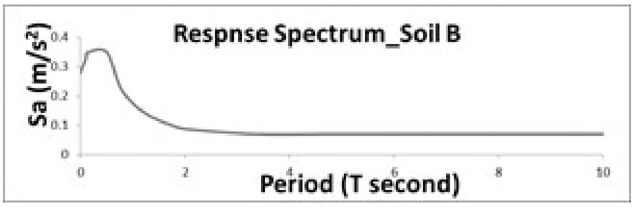

Figure 4 The response spectrums based on soil type B

Table 5 Horizontal mass, stiffness and damping coefficient for SSI

Frame name	NS3NB3 NS3NB6	NS6NB3 NS3NB6	NS9NB3 NB9NB6	NS12NB3 NS12NB6	NS15NB3 NS15NB6	NS20NB3 NS20NB6
m_v	12.57	21.73	27.63	39.68	77.50	100.62
c_v	192.92	277.80	326.03	414.98	648.41	771.67
K_v	122200.00	146640.00	158860.00	179226.66	224033.33	244400.00
m_h	3.98	6.88	8.75	12.56	24.54	31.86
c_h	32.97	47.48	55.72	70.93	110.83	131.89
K_h	102514.28	123017.14	133268.57	150354.28	187942.85	205028.57
m_r	22.61	56.27	83.97	153.49	468.42	723.74
c_r	98.51	170.22	216.42	310.79	607.02	788.08
K_r	58500.00	101088.00	128524.50	184565.33	360479.16	468000.00

[1] v, h and r stand for vertical, horizonal and rocking respectively

Also linear and nonlinear time history analyses are performed. P-Delta effect is take into account in nonlinear time history analysis. Duzce earthquake 1999 is selected and obtained from PEER ground motion data base [13] as presented in Table 6. Each earthquake has three records, two horizontal and one vertical, but in this work, just horizontal records are applied on frame structure (see Figure 5)

Table 6 The seismic hazards used in this research.

Earthquakes	RSN[1]	year	M_w[2]	V_{s30}[3] m/s	PGA
Düzce	1615	1999	7.14	338	0.20666

[1]RSN is Record Sequence Number, [2]$M_w$is Moment magnitude scale, [3]V_{s30} is average value of propagation velocity of S waves in the upper 30m of the soil profile at shear strain of 10-5 or less (EC8)

(a)
(b)
(c)

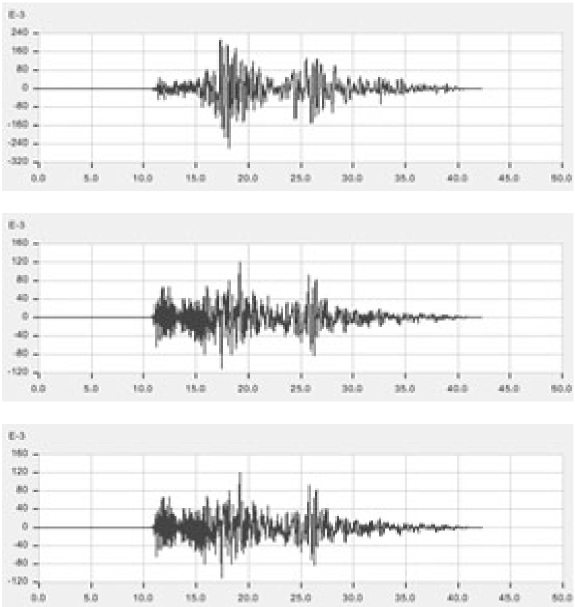

Figure 5 Duzce earthquake acceleration records vers time at RSN1615 (a) East direction (b) North direction (c) vertical direction

In developing the match of the spectrum of a set of time histories to the design spectrum, a first step is to scale each record to the level of the design spectrum. Scaling factors for selected seismic hazard also obtained from PEER ground motion database. Records are scaled to designed response spectrum using single period methods in linear time history analysis. The natural periods of frames are used as period value for scaling, so each frame has a two scaling factors one for rigid base and the other for SSI [7]. Procedure for getting scale factors are from PEER ground motion data base are Inserting target response spectrum and search limits for seismic hazard, after that Search records , Selecting scaling method and obtaining search result and scale factors . The computed scaling factors are listed in Table 7.

Table 7 Scale factor (SF) calculated by the single period method.

Frame name	SF_{rigid}	SF_{SSI}	% diff[1]
NS3xNB6	0.62	0.61	59.60
NS3xNB3	0.61	0.97	-0.70
NS6xNB6	0.87	0.78	-11.63
NS6xNB3	0.76	0.67	-10.99
NS9xNB6	0.56	0.73	61.00
NS9xNB3	0.60	0.97	31.11
NS12xNB3	1.35	1.40	4.26
NS12xNB6	1.35	1.39	3.17
NS15xNB6	1.69	2.03	24.91
NS15xNB3	1.71	2.14	20.26
NS20xNB6	2.99	3.30	-3.03
NS20xNB3	3.21	3.12	10.16

[1]$[(SF_{SSI} - SF_{rigid})/SF_{rigid}] \times 100$

Records in nonlinear time history analysis are scaled according to mean squared error of the differences methods In mean squared error method a quantitative measure of the overall fit of the spectrum of a time history record to a design or target spectrum is the mean squared error of the differences (summed over a discrete set of periods) between the spectral accelerations of the record and the design spectrum, computed using logarithms of spectral acceleration and period. Thus a scaling factor can be determined for each record that minimizes the mean squared error over a user-defined period range of significance [7]. In present study, the range of period for each frame is the natural periods of frames. Scaling factor and period range are presented in Table 8.

Table 8 Scale factor (SF) calculated by the mean squared error method.

NS	Period 3 bay		Period 6 bay		SF
	rigid	SSI	rigid	SSI	
3	0.648	0.723	0.727	0.851	0.69
6	1.193	1.263	1.275	1.345	0.80
9	1.570	1.622	1.680	1.760	0.66
12	1.983	1.987	2.113	2.152	1.35
15	2.383	2.398	2.596	2.517	1.86
20	2.886	2.967	3.063	3.285	3.29

3. Results and Discussion

ETABS Pro is used for SSI analysis of frames which is modeled by 2D links. Links have spring and dashpot coefficients. Fundamental period and base shear of frame structures are evaluated according to response spectrum and linear/nonlinear time history analysis. In all cases of analysis, the soil type

B is used. Fundamental period and base shear are calculated for rigid base and SSI.

Figures 6 shows fundamental period of 3 and 6 bays frames respectively. It is clearly noticed that the Fundamental period of all frames have been increased when SSI model is considered in the analysis, Table 9 present the fundamental period of studied frames and increasing ratio of natural period of frames when SSI analyses are considered. It is clear that the increasing ratio of natural period is about 10% for all frames constructed on soil type B except frame NS3NB3 which is 17.17%.

(a)

(b)

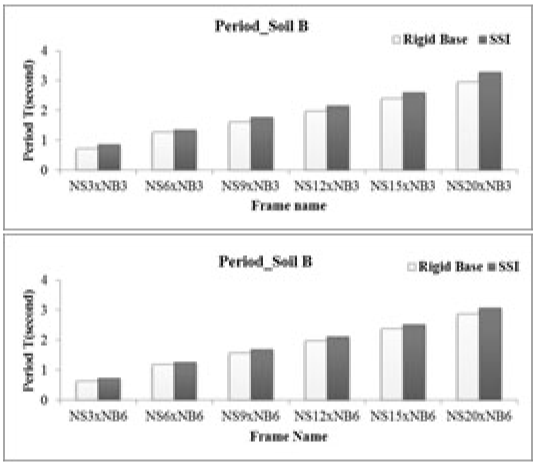

Figure 6 Fundamental period of frames (a) 3 bays (b) 6 bays

Table 9 Nature period of structure for the rigid base and SS.

Frame name	T rigid base	T_{SSI}	$\frac{T_{SSI} - T_{rigid}}{T_{rigid}} \times 100$
NS3xNB3	0.723	0.851	17.70%
NS3xNB6	0.648	0.727	12.19%
NS6xNB3	1.275	1.346	5.57%
NS6xNB6	1.193	1.263	5.87%
NS9xNB3	1.622	1.760	8.51%
NS9xNB6	1.570	1.680	7.01%
NS12xNB3	1.983	2.152	8.54%
NS12xNB6	1.987	2.113	6.34%
NS15xNB3	2.398	2.596	8.26%
NS15xNB6	2.383	2.517	5.62%
NS20xNB3	2.967	3.285	10.72%
NS20xNB6	2.886	3.063	6.13%

Figures 7-9 present the base shear for 3 and 6 bays frames. The base shear is expected to decrease when SSI is considered [14], but depending on the results of present study the effect of SSI on the base shear changes according to analysis type and frames specification (height and size). In response spectrum analysis, the base shear evaluated using SSI model is always smaller than rigid base model. The comparison of the base shear evaluated using rigid base and SSI models are presented in Table 10.

In linear time history analysis still the values of base shear are closed for two type of base, but the base shear calculated according to SSI modeling for high rise frames with 15 and 20 stories is higher than rigid base model as presented in Table 10.

In nonlinear analysis still base shear for rigid base is higher than SSI for frames with 12 stories or lesser. And the base shear for frame structure with 15 and 20 stories are higher for SSI models. The difference between base shear values in rigid base and SSI modeling is going to be bigger especially for low rise frames. The rigid base lead to over estimation of base shear as shown in Table10.

(a)

(b)

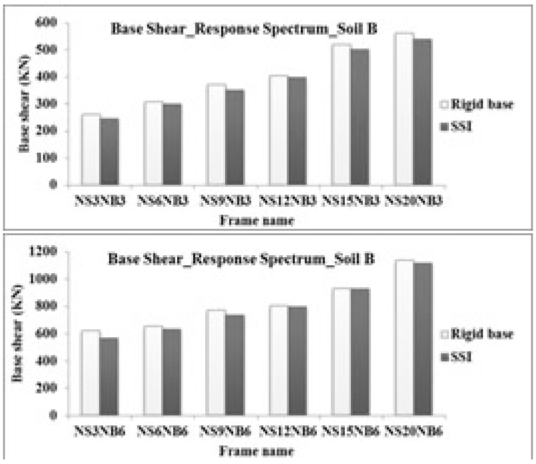

Figure 7 Base shear of frames evaluated by response spectrum analysis (a) 3 bays (b) 6 bays

(a)

(b)

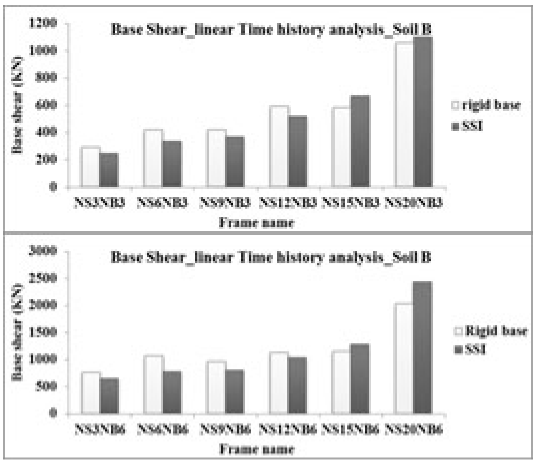

Figure 8 Base shear of frames evaluated by linear time history analysis (a) 3 bays (b) 6 bays

(a)

(b)

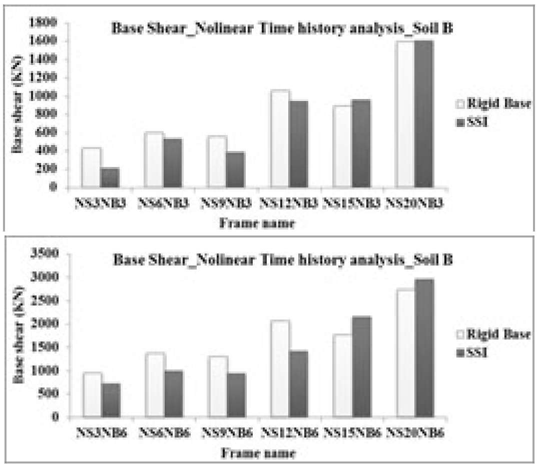

Figure 9 Base shear of frames evaluated by nonlinear time history analysis (a) 3 bays (b) 6 bays

Table 10 Comparison of base shear based on rigid base and SSI model.

Frame name	RSA[1]	LTHA[2]	NLTHA[3]
NS3xNB3	-6.5	15.70	52.82
NS3xNB6	-3.0	19.68	11.41
NS6xNB3	-5.8	11.51	31.82
NS6xNB6	-2.0	12.38	10.88
NS9xNB3	-3.6	-14.22	-6.95
NS9xNB6	-4.2	-3.97	-0.45
NS12xNB3	-9.6	15.44	23.96
NS12xNB6	-3.7	27.31	27.35
NS15xNB3	-4.3	16.41	28.10
NS15xNB6	-1.0	8.95	31.34
NS20xNB3	-0.8	-11.38	-21.52
NS20xNB6	-2.0	-20.00	-8.22

[1]Base shear ratio according to response spectrum analysis $[(F_{SSI} - F_{rigid})/F_{rigid}] \times 100$ where F is base shear in kN
[2]Base shear ratio according to linear time history analysis $[(F_{SSI} - F_{rigid})/F_{rigid}] \times 100$
[1]Base shear ratio according to nonlinear time history analysis $[(F_{SSI} - F_{rigid})/F_{rigid}] \times 100$

4. Conclusion

In response spectrum analyses, SSI gives lower base shear compare to rigid base for all frames. Similarly, in linear and nonlinear time history analysis, SSI computes lower base shear when frame height is 12 stories or lesser. Then base shear for frames with 15 and 20 stories are higher, also the differences between values of base shear are large especially for low rise frames when nonlinear analysis is done. The differences in the fundamental period between rigid base and SSI models are small. However SSI analyses give slightly higher fundamental period. Also the effect of SSI is clear when nonlinear time history analysis is considered. As mentioned by Marjanović and Petronijivic [15].the effect of SSI modeling in stiff soil is small and increase when the soil is being weaker. Defining a new model of bases using SSI changes the response of structure to seismic loads which means it shall be changing the factors of static and dynamic methods for calculating seismic response of structure in codes and those factors starts to introduce in some codes like FEMA 440 [12,16].

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