



## Response of Steel Moment and Braced Frames Subjected to Near-Source Pulse-Like Ground Motions by Including Soil-Structure Interaction Effects

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Received 20 December 2016; Accepted 27 January 2017

### Abstract

Most seismic regulations are usually associated with fixed-base structures, assuming that elimination of this phenomenon leads to conservative results and engineers are not obliged to use near-fault earthquakes. This study investigates the effect of soil–structure interaction on the inelastic response of MDOF steel structures by using well known Cone method. In order to achieve this, three dimensional multi-storey steel structures with moment and braced frame are analysed using non-linear time history method under the action of 40 near-fault records. Seismic response parameters, such as base shear, performance of structures, ductility demand and displacement demand ratios of structures subjected to different frequency-contents of near-fault records including pulse type and high-frequency components are investigated. The results elucidate that the flexibility of soil strongly affects the seismic response of steel frames. Soil–structure interaction can increase seismic demands of structures. Also, soil has approximately increasing and mitigating effects on structural responses subjected to the pulse type and high frequency components. A threshold period exists below which can highly change the ductility demand for short period structures subjected to near-fault records.

*Keywords:* Near-Fault Earthquakes; Soil-Structure Interaction; Cone Model; Steel Structure; Ductility Demand.

### 1. Introduction

Ground shaking near an active fault has some distinctive characteristics. Short–duration impulsive motion is one of them. This pulse is clearly evident in velocity time history where the fault rupture propagates towards the site at a velocity close to shear wave velocity. In addition, Near-fault records are rich in high frequencies. In other words, both short and high frequency contents of near-fault records are strong as opposed to ordinary ground motions that only have strong low frequency content [1, 2].

A plenty number of studies on both linear and nonlinear behaviour of structures subjected to pulse type motion of near-fault earthquakes were done. However, the effect of soil flexibility on response of structures subjected to near-fault records was ignored. In addition, high-frequency part of near-fault earthquakes was overlooked. As a consequence, all research process and computer simulation models may be doomed to provide unrealistic or at least questionable results. Somerville et al studied on the particular effects of forward directivity (1997) [3]. Among the years 2002 and 2003, an equation was introduced in order to create pulse type motion of near-fault earthquakes by Papageorgiou and Mavroeidis [5, 6]. The effect of pulse type motions on response spectrum was studied by Mavroeidis et al. in 2007. They clarified that pulses are able to strongly influence on response spectrum [7]. A new method was proposed to identify pulse-type component of near-fault ground motions by Tang and Zhang (2011) [8]. Iervolino et al. (2012) studied on inelastic displacement factors under near-fault ground motions and expressed a formulation for estimating inelastic displacement of structures subjected to near-fault earthquakes [9]. In 2004, the response of multi-story framed structures subjected to far-field earthquakes and simulated pulse type motions were

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studied by Krawinkler and Alavi. They proved that near-fault ground motions can impose higher forces on structures. They also introduced a new formula to calculate pulse period [10]. Ghannad et al. studied on response of structures with rigid-base subjected to different frequency contents of pulse type motions [11]. They used moving average filtering for decomposing frequency components of earthquakes. In 2006, it was found that the lower floors of the structures experience slighter demands than the higher ones under both forward and fling step pulses of near-fault records [12]. Ghannad et al. studied on response spectrum of near-fault ground motion and proposed an elastic pseudo-acceleration response spectrum for near-field records [13]. Lately, a study has been done on the ductility demands of structures subjected to ordinary earthquakes and pulse type motions by Sehhati et al [14]. They found that near-fault ground motions can cause lesser demands in lower floors of the building and higher floors need more ductility. Sharif and Behnamfar (2012) assessed the impacts of equivalent cosine pulse type motion on behaviour of steel moment resisting frames. They proved that the response of structure is influenced by ground motion pulse period [15]. It should be mentioned that not only all the previous studies were performed only for structures with rigid-base and there were no traces of soil effects, but also the effects of high frequency content were attracted much less attention.

Soil flexibility can highly affect dynamic properties of superstructure. Aviles and Perez-Rocha (2003, 2005) tried to modify strength reduction factors in order to be able to consider SSI effects on the nonlinear response of structures [16, 17]. Ghannad and Ahmadnia concluded that the strength reduction factor is different for structure with rigid base and soil-foundation-structure system [18]. Examining recent strong ground motions, it is found that the seismic damage sustained in a superstructure is strongly affected by the response of its foundation and the supporting ground and damage index of structure can also be affected by SSI mechanism [19-21]. Behmanesh and Khoshnoudian studied about the effects of SSI on inelastic behavior of existing buildings and concluded that neglecting soil flexibility can cause inaccuracy in the structural analysis [22]. Among all these researches only the effects of far-fault earthquakes were studied. In terms of near-fault ground motions, high frequency part of records was ignored. Lately, the effects of pulse period of artificial near-fault earthquakes on demands of structures by applying SSI were studied by Khoshnoudian and Ahmadi. Their results are similar to those obtained by Sharif and Behnamfar [15]; but, with taking the effects of soil into account. They demonstrated that response of soil-structure systems chiefly depends on the ratio of the pulse period to period of the soil-structure system [23]. However, they did not consider the high frequency component.

All previous studies are mainly based on the dynamic response of single degree of freedom (SDOF) oscillator whereas more realistic and accurate response of real structural response needs multi degree of freedom (MDOF) systems. Furthermore, the impact of higher modes participation on structural response can be affected by SSI and near-field records which cannot be captured by the SDOF oscillator and simulated pulse-type motion. Consequently, for representing the real behaviour of near-fault ground motion characteristics simulated pulse-type motion is not appropriate. The reason lies in the fact that by using this type of earthquake simplification it is not possible to scrutinize all the applied energy to the structure. Therefore, it is indispensable to study both high frequency and pulse type components instead of using only simulated pulse alone. As mentioned earlier, structural responses could be affected by soil flexibility and various quota of near-fault ground motions and it is more complicated than SSI effects on behaviour of structures subjected to ordinary ground motions.

In this study, using the entire frequency contents of near-field records these limitations have been removed. Also the effects of soil flexibility and its damping on response of structures subjected to each frequency components are investigated. For investigating the real response of structures, three dimensional MDOF structures were used. A series of 40 records which have been already decomposed by Baker, are used [24]. Since distance to the site is a necessary condition but not sufficient; it cannot be an appropriate criterion. Selecting near-fault records with forward directivity effects requires some procedures to define pulse-containing records. Up to now, the most commonly methods for identifying the pulse of forward directivity effects can be summed up in two techniques. Firstly, site must satisfy both seismologic and geometric conditions. Secondly, pulse-like shape should be evident in the velocity time history [25]. Lately, Baker employed wavelet analysis to find ground motion with forward directivity pulse [24]. In this research, the combination of all described criteria was used to select ground motion records containing directivity pulses.

The main priority of this research is to explore the effects of SSI on the inelastic response of three dimensional steel frames with two different lateral resisting systems subjected to near-fault ground motions. To fully portray the effects of near-fault earthquakes on response of structures, more than 900 nonlinear time history analysis were performed to detect the influences of near-fault ground motions, its various well known components and interacting parameters on responses of two useful lateral resisting systems of steel structures. For this purpose, inelastic behaviour of one, five and ten-story steel structures with concentrically braced frame (CBFs) and moment resisting frames (MRFs) in two different directions subjected to near-fault earthquakes considering fixed-base and the effects of soil were investigated.

## 2. Soil-Structure Interaction Modeling

Two-dimensional frame of the structures by taking into account the soil beneath them is shown in Figure 1. Soil layer parameters are represented in Table 1. For modeling soil-structure system, substructure method is used. The site in Figure 1. consists of one layer on a flexible half-space, where waves propagate vertically towards infinity, yielding radiation damping in the vertical direction, as for a site of a homogeneous half-space. Soil is replaced by a mechanical equivalent system based on the Cone model concept. Well-known Cone model has been recommended in [26-30]. This model represents circular rigid foundation known as a massless disk and mass moment of inertia  $I_0$ . Therefore, the term  $m_0$  set equal to zero and the settlement of structures on the soil has been neglected and all parts of the foundation have the same displacement. The horizontal and vertical translational, rotational, and torsional motions of the foundation are introduced as the horizontal,  $s$ , (sway), the rocking,  $r$ , and the torsional,  $T$ , DOFs, respectively. Moreover, in order to be able to calculate the soil stiffness independently of frequency, an additional internal degree of freedom,  $\phi$  is added. The term  $u_g$  and  $\phi_h$  were applied to the horizontal and rocking motion at the higher roof.  $u$  was applied to the strain deformation of the superstructure. Note that the kinematic part of SSI is ignored and only the inertial part was considered. This is due to the assumption that the rigid foundation lies down on the surface of the soil.

There are two sources of damping in the soil. The first damping mechanism of energy dissipation in the soil is known as radiation damping. In addition to the radiation damping, soil materials can also waste energy through friction. In order to consider the material damping of the soil, the correspondence principle is applied by replacing the elastic modules with the corresponding complex ones.

$$Ec \rightarrow Ec^* = \rho c p^2 (1 + 2i\zeta) \tag{1}$$

$$G \rightarrow G^* = \rho c s^2 (1 + 2i\zeta) \tag{2}$$

Strong ground motions can drive the soil medium to inelastic behavior. In order to consider inelasticity and indirectly hysteretic behavior, the soil medium can be assumed as equivalent elastic using the secant shear modulus,  $G_s$  instead of the aforementioned maximum shear modulus,  $G_0$ . This is according to ASCE/SEI 7-10 [31].

Soil layer parameters are presented in Table 1.

Table 1. Soil Layer Parameters

Shear wave velocity ( $V_s$ ) (m/s)	Poisson's ratio ( $\nu$ )	Damping ratio $\xi_0$	Layer depth	Site classification
275	0.4	0.05	infinite	D

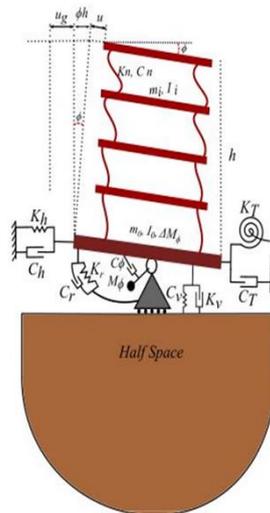


Figure 1. Flexible-base system

## 3. Methodology and Procedure of Analysis

### 3.1. Problem Parameters

The dynamic behavior of soil-structure system depends on the height and dimension of the structure, its dynamic specifications like its stiffness, the properties of the soil layer and input ground motion. It has been proved that the effect of these factors can be best described by the following non-dimensional parameters [32].

- The ratio of the stiffness of the structure to that of the soil which called dimensionless frequency.

$$\bar{S} = \frac{\omega_s h}{c_s} \quad (3)$$

Where  $\omega_s$  is the circular frequency of the rigid base structure and  $c_s$  is the shear-wave velocity of the soil.

- Aspect ratio of the building

$$\frac{h}{r} \quad (4)$$

Where  $r$  is the equivalent radius of the foundation. This is an index that represents structures' slenderness ratio.

- Poisson's ratio of soil equal to 0.4. Damping and stiffness of the soil is affected by Poisson's ratio.

### 3.2. Method of Analysis

This paper examines 24 three dimensional MDOF structure. The studied architectural plan of models is shown in Figure 2. Structures have one, five and ten floors. The story height, dead and live loads are presumed 3 m, 580  $kg/m^2$  and 200  $kg/m^2$  as for typical buildings, respectively. Two useful lateral resisting systems including special concentric braced frame in X direction and special moment frame in Y direction were used. Structures are residential in very high risk zones on soil type D and were loaded and designed according to National building regulations of Iran [33, 34].

Soft soil was chosen for this research, on the ground that the greatest effect of SSI is on this kind of soil. Plastic hinges specifications are according to FEMA 356 [35]. Models have the capability to be analyzed directly in the time domain for nonlinear dynamic analysis. For this purpose, PERFORM 3D [36] and SAP 2000 [37] softwares were used. The study is done for the target ductility value of the rigid-base state equal to 6 ( $\mu_{fix} = 6$ ). For this purpose, the yield strength demand of the rigid-base structure is calculated by iteration within a 1% error for each ground motions.

In MDOF system, it needs to calculate the ductility demand for each story; the greatest value of each story's ductility is the main ductility demand of the MDOF system. In order to consider the damping of the superstructure, Rayleigh damping was used. Total number of 40 near-fault records by using wavelet analysis was selected, which will be explained in the next subsection. Almost 405 nonlinear time history analyses for 40 near-fault records are performed on structures with various heights, lateral resisting systems and thus non-dimensional frequencies and aspect ratios. Models were analyzed once with fixed-base and for the second time by considering SSI effect by modeling dashpots and springs under the models. The effect of SSI on period change, damping ratios, base shear, structural performance, structural members damage ratio index, maximum displacement of stories, inter story drift ratios (IDR) were studied. Also, to evaluate the ductility demand of soil-structure models subjected to near-fault records by considering SSI effects, a collection of 850 soil-structure systems including MDOF models with different key parameters and fixed-base periods from 0.05 to 3 second were investigated for each input ground motions. Furthermore, around 960 nonlinear time history analyses were performed for considering the effect of different frequency contents on response of structures. Due to size limitation of papers, results were not presented in detail for each records separately, and they were limited to the average of all records.

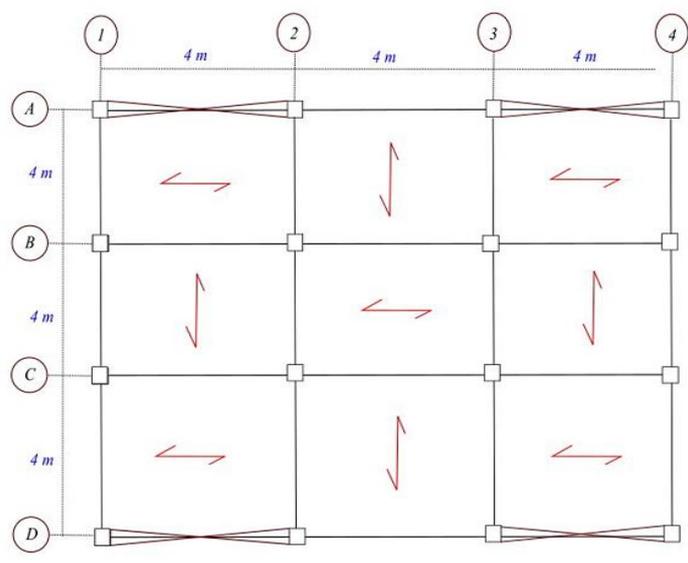


Figure 2. Planes of structures

#### 4. Seismic Input

Due to the sensitivity of the structural response to input earthquake, especially in nonlinear dynamic analysis, a set of 40 near-fault earthquakes have been used which have been recorded at a distance less than 10 km from the fault rupture. A complete list of the original records and their specifications such as pulse period ( $T_p$ ), peak ground velocity; magnitude and epicentral distance are presented in Table 2. Also for considering the effects of different frequency contents of near-fault earthquakes on response of structures, these 40 records have been studied by Baker [24] for wavelet analysis. To achieve this goal, MATLAB [38] code is developed to use for wavelet analysis and decomposing different frequency contents.

In order to select near-fault records, two different methods were used. First, collecting data for near-fault earthquakes required a simple technique for determining which records contain a pulse and which do not. Methods used to date can be summarized by two general requirements. The first is that the site must have the geometric and seismologic requirements for forward directivity phenomenon. For instance, the distance from the epicentre of the earthquake to the site must be less than 20 km. On the other hand, by observing the velocity time history, pulse-like shape can easily be identified [25].

In near-fault records that contain a directivity pulse demonstrate a peak velocity two times the value of any peak velocity with respect to the other one. Although other criteria were considered, but for the initial selection of pulse-like records these methods were used for their simplicity and ease of calculation. In the next step another method is utilized to distinguish directivity pulses which require some further discussions. In this method, the square of the velocities at each time step throughout the record must be calculated. The pulse will then show up as a single or several large spikes. There are two advantages in this method. First, it only reports the absolute values of the velocity. Next is the exaggeration of relatively high velocity pulses. Disparities between the pulse and the rest of the record are more pronounced and more easily distinguished visually. An example of the velocity squared time history is shown in Figure 4. for the Erzincan, Erzincan Receiving Station record. The limiting criteria would then be that the peak velocity squared value must be four times the value of any other velocity squared value without the pulse. Note that this is identical to saying that the peak absolute velocity is twice the absolute velocity without the pulse [25].

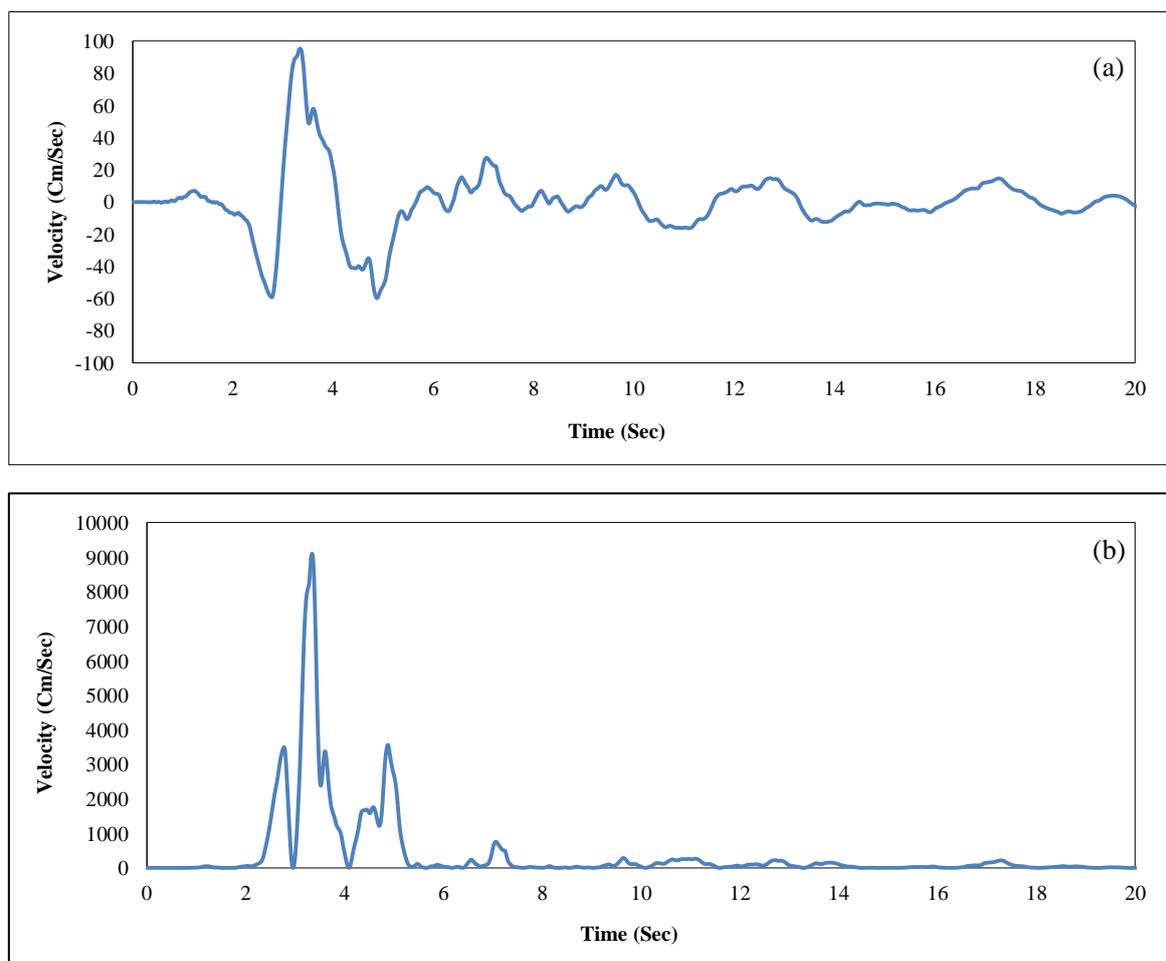


Figure 3. Velocity squared time history for the Erzincan Receiving Station, Erzincan, Turkey Earthquake (a) Velocity time history (b) Velocity squared time history

As a third method wavelet analysis has been used in order to identify pulse like records. Recently a new method was developed to extract the largest velocity pulse from an original near-fault record by Baker [24]. He used Daubechies wavelet of order four as the mother wavelet to identify the dominant pulse of the record. In this research the size of the extracted pulse was used to develop quantitative criteria for classifying a ground motion as pulse-like. To identify pulse-like records in fault-normal ground motions potentially caused by directivity effects, two additional criteria were applied: the absolute amplitude of the velocity pulse is large relative to the remainder of the record, and the pulse arrives at the beginning of the strong ground motion. These criteria are known as pulse indicator,  $PGV_{min}$  and late-arriving pulses.

#### 4.1. Pulse Indicator

According to a definition provided by Baker [24], pulse indicator (P.I) would be computed as:

$$P.I = \frac{1}{1 + e^{-23.3+14.6(PGV\ Ratio)+20.5(energy\ Ratio)}} \quad (5)$$

The term “*PGV Ratio*” and “*energy Ratio*” refers to the ratio of the peak ground velocity (PGV) of the high frequency record and the original record’s PGV, and the energy of the high frequency record divided by the original record’s energy, respectively. Pulse indicator takes values between zero and one. Increase in the P.I values provides a strong indication that the ground motion is pulse like. The predictions are continuous, which raises the question of how to use them if a discrete classification is desired. Here, records with scores above 0.85 and below 0.20 are classified as pulses and non-pulses, respectively.

#### 4.2. Late Arriving Pulses Criteria

Pulse indicator is a necessary condition but not sufficient. The second requirement is pulse arrival time. It could be a pulse record of forward directivity once it occurs just at the beginning of ground motion. Late-arriving pulses can be recognized by computing the cumulative squared velocity of both the original record and the extracted pulse. At time  $t$ , the cumulative squared velocity (CSV) would be computed as:

$$CSV(t) = \int_0^t V^2(u) du \quad (6)$$

Where  $CSV(t)$  refers to the cumulative squared velocity at time  $t$  and  $V(u)$  is the ground-motion velocity at time  $u$ . By assessing  $CSV(t)$  function for the original ground motion and extracted pulse, the times at which each reaches  $x\%$  of its total CSV are determined. These times are denoted  $t_{x\%,orig}$  and  $t_{x\%,pulse}$ , for the original ground motion and the extracted pulse, respectively. By adjusting the percentage criteria for both high and low frequency contents of ground motions, it was determined that early arriving pulses have  $t_{20\%,orig}$  values that are greater than  $t_{10\%,pulse}$  [24].

#### 4.3. $PGV_{min}$ Criteria

The final criteria considered here is to determine a minimum for peak ground velocity of the ground motion. Since the velocity time history simplicity is in some relatively low-intensity ground motions, some pulse-like shapes become evident. Brief duration of the far-field *S*-wave pulse occurs in some low-magnitude events that have a small source area; therefore, this pulse-like shape can be observed in low magnitude earthquakes. To avoid choosing such earthquakes as near-fault ground motions, records with PGVs less than some threshold can be excluded from engineering practice for the reason that it does not have the potential to damage structures. Several threshold levels of PGV were considered, and a 30 cm/sec level was seen to eliminate nearly all small magnitude and large-distance ground motions, while retaining the damaging pulse like ground motions (which are also more likely to be caused by directivity effects) [24].

Earthquakes, which are listed in Table 2. have passed all of the proposed filter.

**Table2. List of near-fault ground motions used in this study**

Earthquake	Year	Station	$T_p$ wave-let	PGV (Cm/Sec)	$M_w$	Epi. D.
Imperial Valley-06	1979	Aeropuerto Mexicali	2.4	44.3	6.5	2.5
Imperial Valley-06	1979	Agrarias	2.3	54.3	6.5	2.6
Imperial Valley-06	1979	Brawley Airport	4.0	36.1	6.5	43.2
Imperial Valley-06	1979	Holtville Post Office	4.8	55.1	6.5	19.8
Imperial Valley-06	1979	El Centro Differential Array	5.9	59.6	6.5	27.2

Imperial Valley-06	1979	EC Meloland Overpass FF	3.3	115.0	6.5	19.4
Imperial Valley-06	1979	EC County Center FF	4.5	54.5	6.5	29.1
Imperial Valley-06	1979	El Centro Array #10	4.5	46.9	6.5	26.3
Imperial Valley-06	1979	El Centro Array #11	7.4	41.1	6.5	29.4
Imperial Valley-06	1979	El Centro Array #8	5.4	48.6	6.5	28.1
Imperial Valley-06	1979	El Centro Array #7	4.2	108.8	6.5	27.6
Imperial Valley-06	1979	El Centro Array #6	3.8	111.9	6.5	27.5
Imperial Valley-06	1979	El Centro Array #5	4.0	91.5	6.5	27.8
Imperial Valley-06	1979	El Centro Array #4	4.6	77.9	6.5	27.1
Imperial Valley-06	1979	El Centro Array #3	5.2	41.1	6.5	28.7
Coalinga-07	1983	Coalinga-14th & Elm (Old CHP)	0.4	36.1	5.2	9.6
Erzican, Turkey	1992	Erzican	2.7	95.4	6.7	9.0
Whittier Narrows-01	1987	LB - Orange Ave	1.0	32.9	6.0	20.7
Loma Prieta	1989	Gilroy Array #2	1.7	45.7	6.9	29.8
Loma Prieta	1989	Oakland - Outer Harbor Wharf	1.8	49.2	6.9	94.0
Landers	1992	Barstow	8.9	30.4	7.3	94.8
Landers	1992	Yermo Fire Station	7.5	53.2	7.3	56.0
Chi-Chi, Taiwan	1999	TCU031	6.2	59.9	7.6	67.8
Chi-Chi, Taiwan	1999	TCU038	7.0	50.9	7.6	73.1
Chi-Chi, Taiwan	1999	TCU040	6.3	53.0	7.6	69.0
Chi-Chi, Taiwan	1999	TCU065	5.7	127.7	7.6	26.7
Chi-Chi, Taiwan-03	1999	TCU076	4.0	63.7	7.6	16.0
Chi-Chi, Taiwan-06	1999	CHY101	2.8	36.3	6.3	50.0
Chi-Chi, Taiwan	1999	TCU103	8.3	62.2	7.6	99.7
Chi-Chi, Taiwan-03	1999	CHY024	3.2	33.1	6.2	25.5
Yountville	2000	Napa Fire Station #3	0.7	43.0	5.0	9.9
Northwest China-03	1997	Jiashi	1.3	37.0	6.1	19.1
Kobe, Japan	1995	Takarazuka	1.4	72.6	6.9	38.6
Kobe, Japan	1995	Takatori	1.6	169.6	6.9	13.1
Northridge-01	1994	LA - Wadsworth VA Hospital North	2.4	32.4	6.7	19.6
Northridge-01	1994	Sylmar - Converter Sta	3.5	130.3	6.7	13.1
Northridge-01	1994	Sylmar - Converter Sta East	3.5	116.6	6.7	13.6
Northridge-01	1994	Sylmar - Olive View Med FF	3.1	122.7	6.7	16.8
Superstition Hills-02	1987	Parachute Test Site	2.3	106.8	6.5	16.0
N. Palm Springs	1986	North Palm Springs	1.4	73.6	6.1	10.6

## 5. Seismic Response Evaluation of Soil–MDOF Structure System

### 5.1. Fundamental Periods and Damping Ratios of Structures

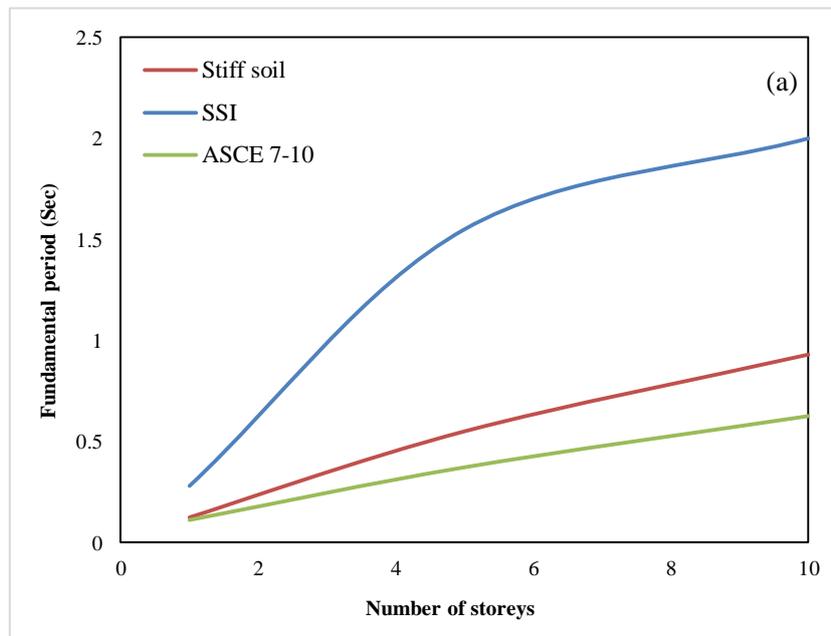
This section examines the fundamental period of vibration of the building structures under consideration. By considering the soil under the structures, the stiffness of structures reduces and damping increases. These are due to soil flexibility and new sources of damping namely radiation damping and hysteretic material damping. Intensity of SSI depends on  $\bar{S}$  and  $h/r$  ratios. By increasing  $\bar{S}$ , SSI effects on response of structure rise and structural stiffness decreases. Moreover, structure is influenced more by soil damping. Dynamic-stiffness coefficients can be seen in Table 3. according to CONAN [26] software. In this table only imaginary part of impedance function has been shown.

By assuming a constant amount for  $\bar{S}$ , increase in  $h/r$  ratio can increase period lengthening. The reason lies in the fact that high structures are more affected by overturning moment and foundation rotation. Moreover, high rise structures absorb less damping from the soil due to the fact that foundation rocking dissipates energy into the soil less efficiently. SSI affects CBFs more than MRFs because  $\bar{S}$  parameter is more severe in CBFs. From Figure 3, it can be observed that unlike the empirical formulas of seismic design codes for computing the fundamental periods of the CBFs and MRFs which always obtained greater value for MRFs than CBFs, fundamental period of the CBFs can

reach higher than MRFs especially on very soft soil. This can be justified by the fact that SSI can cause more rocking motion and internal forces on springs of substructure in CBFs. As a consequence, these formulas need to be improved, especially for CBFs located on soft soil. Computing fundamental period through codes leads to lower values. It seems that this is according to a conservative calculation of the imposed spectral accelerations and seismic design forces. In contrast, this can lead to stiffer structures with fewer displacements. In practical situation,  $\bar{S}$  parameter has numerical values between 0 to 3. Zero means solid base, like fixed base and 3 means maximum SSI effects, but sometimes it can be more than 3 when structures is located on very soft soils.

**Table 3. Imaginary parts of impedance functions**

1 Story				
	Horizontal	Vertical	Rocking	Torsion
$\omega(x)$	$iC\omega(x)$	$iC\omega(x)$	$iC\omega(x)$	$iC\omega(x)$
50.45	7.00E+08	5.00E+09	5.20E+10	1.60E+10
$\omega(y)$	$iC\omega(y)$	$iC\omega(y)$	$iC\omega(y)$	$iC\omega(y)$
10.77	1.70E+08	1.25E+09	9.20E+09	3.00E+09
5 Story				
$\omega(x)$	$iC\omega(x)$	$iC\omega(x)$	$iC\omega(x)$	$iC\omega(x)$
11.46	1.65E+08	1.30E+09	9.60E+09	3.10E+09
$\omega(y)$	$iC\omega(y)$	$iC\omega(y)$	$iC\omega(y)$	$iC\omega(y)$
5.07	8.00E+07	7.10E+08	7.10E+09	1.10E+09
10 Story				
$\omega(x)$	$iC\omega(x)$	$iC\omega(x)$	$iC\omega(x)$	$iC\omega(x)$
6.77	1.00E+08	8.70E+08	7.50E+09	1.60E+09
$\omega(y)$	$iC\omega(y)$	$iC\omega(y)$	$iC\omega(y)$	$iC\omega(y)$
4.48	7.20E+07	6.50E+08	7.00E+09	1.00E+09



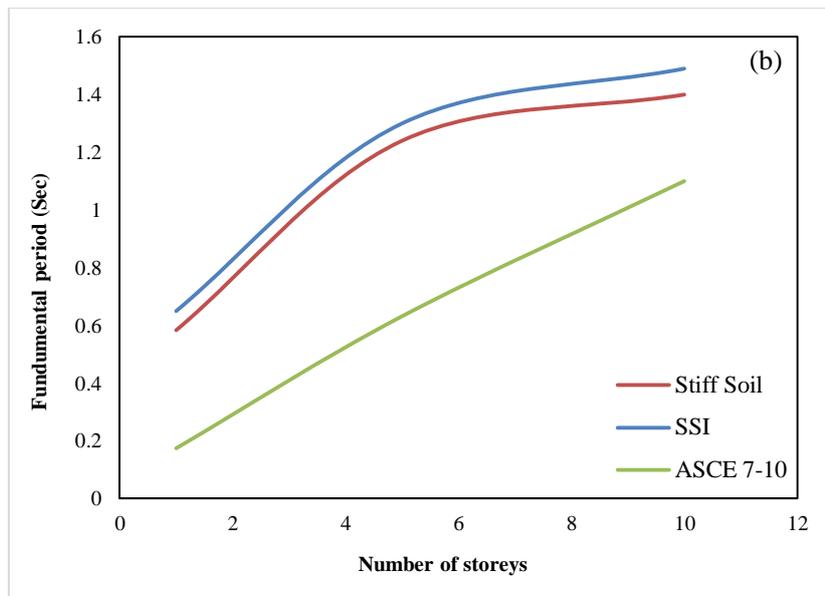
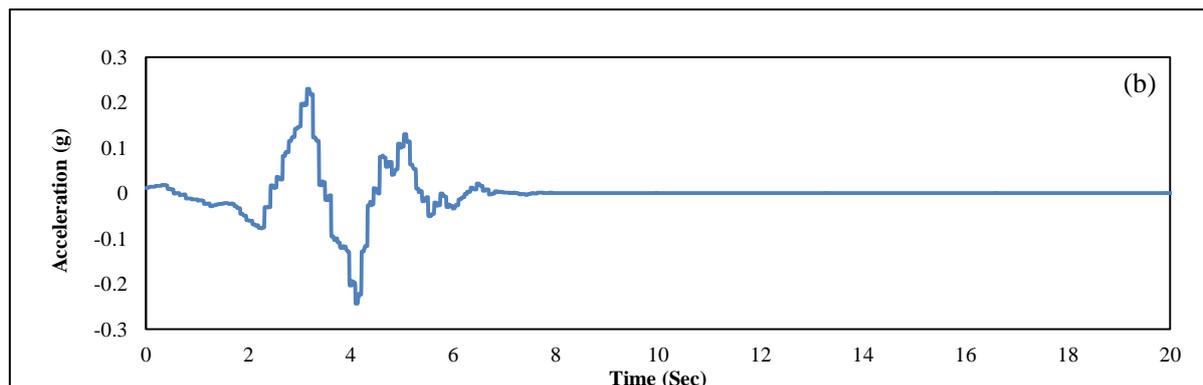
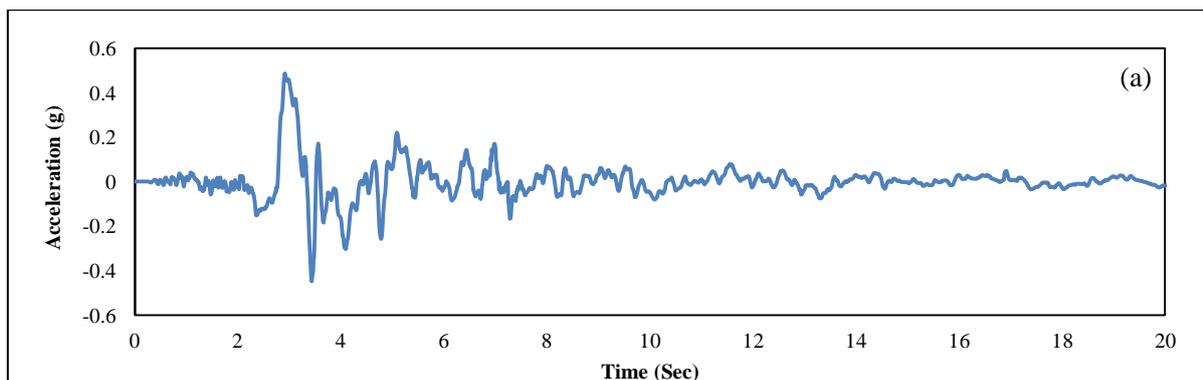
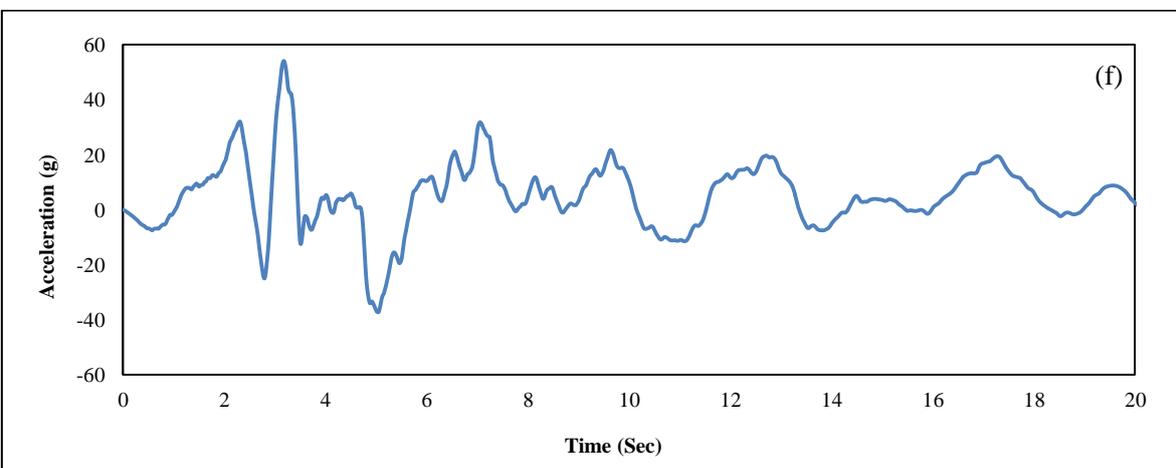
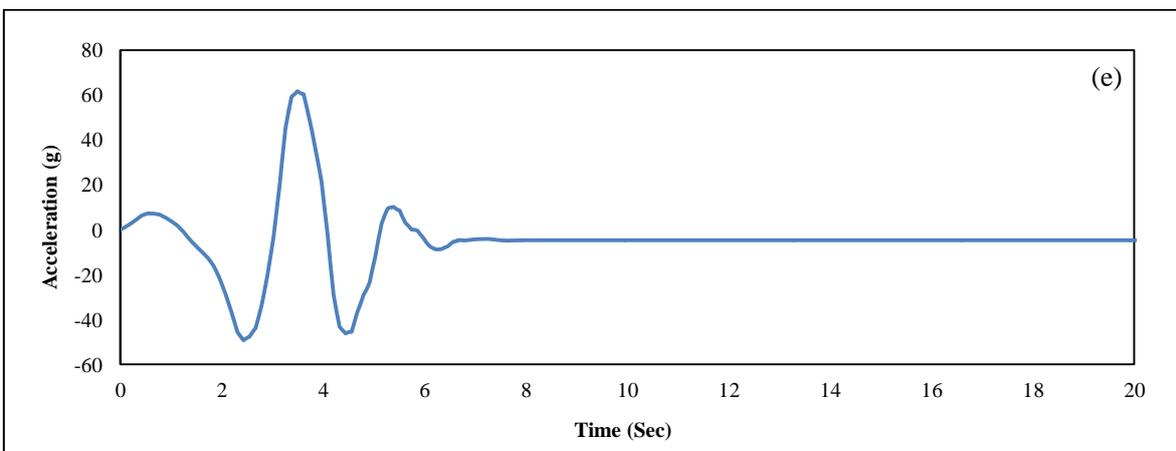
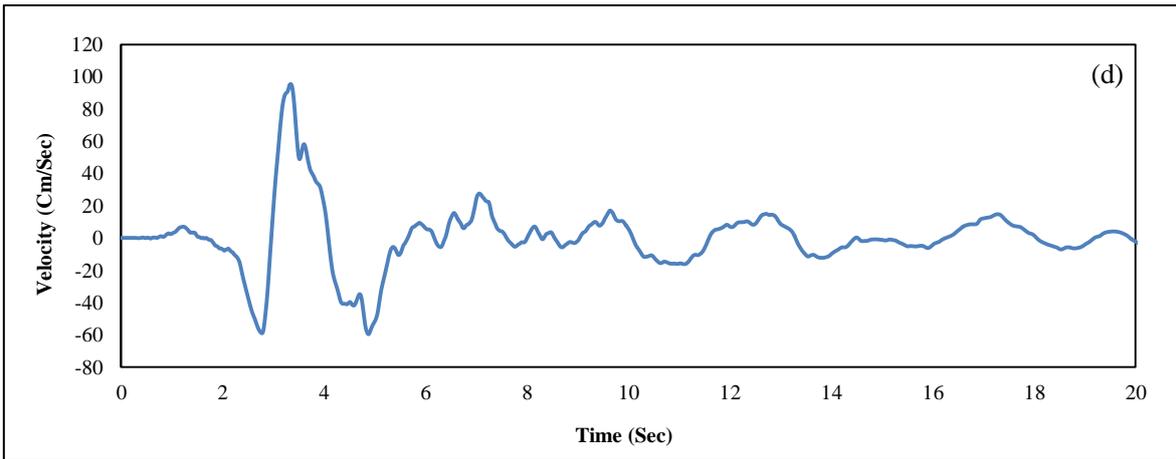
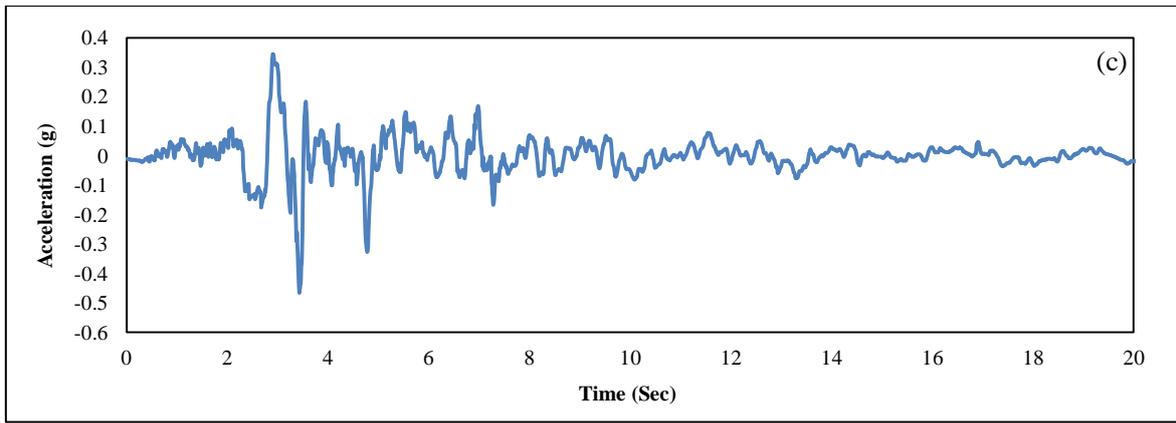


Figure 4. Fundamental period of structures (a) CBFs (b) MRFs

### 5.2. Near-Fault Ground Motion Response Spectrums

Unlike far-fault earthquake; where the elastic response spectra usually only have strong high-frequencies, near-fault records have two frequency contents. High frequency content plays an important role on response of short structures or for structural systems in which higher modes are important; for instance, structures with extreme torsional irregularity. On the other hand, low frequency content which occurs at long periods can control response of tall structures. To separate different frequency contents of pulse type ground motions, wavelet analysis code is written by using MATLAB software. The part that contains high frequency values is commonly known as R.G.M (Residual Ground Motion) or B.G.R (Back Ground Record). The part with pulse type motion is called Ex.P (Extracted Pulse) or P.T.R (Pulse Type Record). For brevity, results are only shown for Erzincan Turkey in Figure 4b and 4e illustrate the acceleration and velocity time histories which contain low frequency part of the earthquake, respectively. By contrast Figure 4.c and 4.f illustrate the acceleration time histories of high frequency section. In Figure 4.g and 4.h, acceleration and velocity response spectrums of P.T.R and B.G.R parts are presented, respectively.





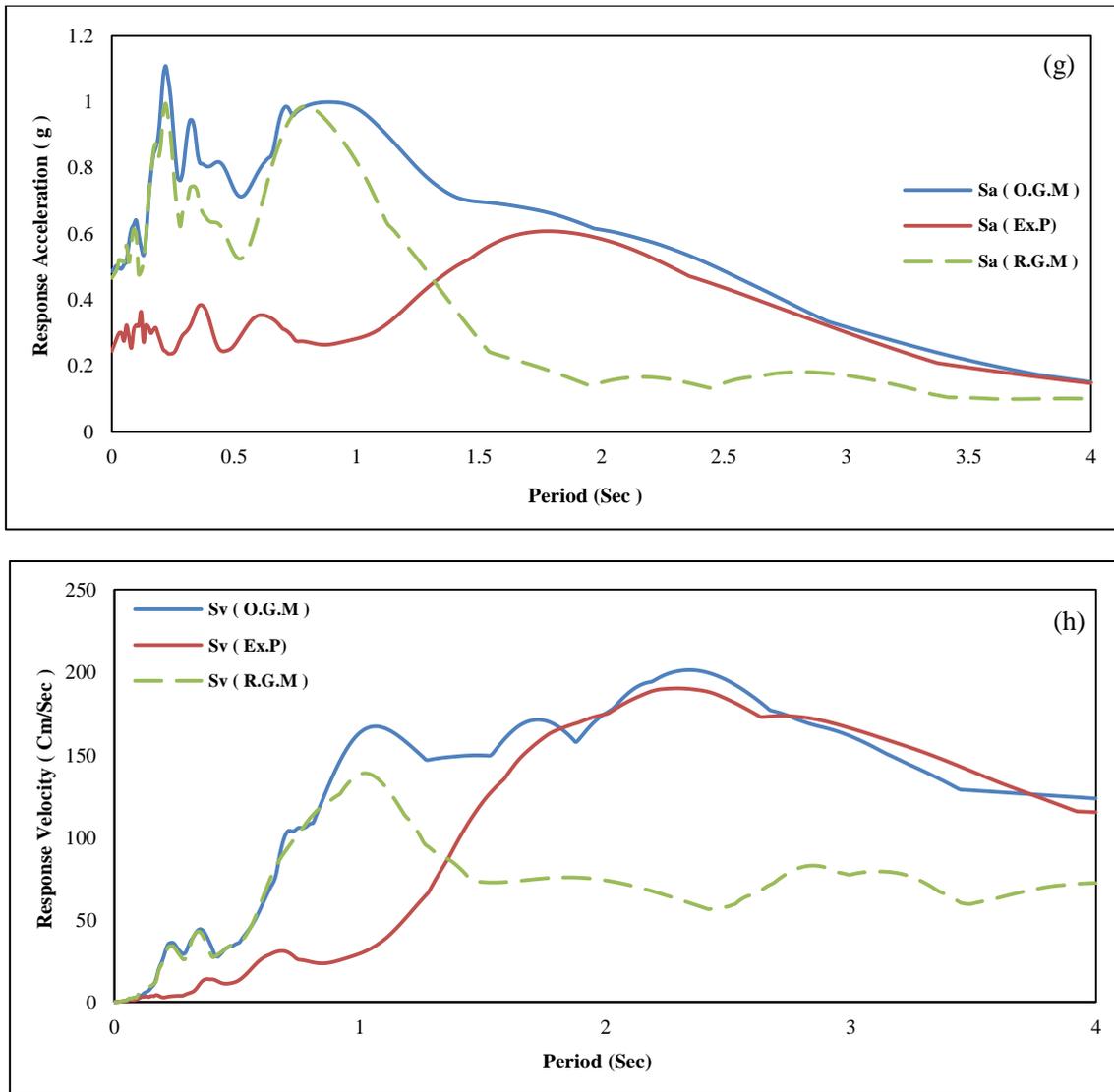


Figure 5. Erzican Turkey 1992 Time histories and Response spectrums

- (a) Original ground motion acceleration time history (b) Pulse type acceleration time history (c) Residual acceleration time history (d) Original ground motion velocity time history (e) Pulse type velocity time history (f) Residual velocity time history (g) Acceleration response spectrum with 5% damping ratio (h) Velocity response spectrum with 5% damping ratio

### 5.3. The Effect of SSI and Near-Fault Frequency Content on Increasing the Participation of Higher Modes

To assess the ability of the soil on increasing the participation of higher modes on structural response, maximum displacement of MDOF and equivalent SDOF models were computed. The ratio of these two parameters which is presented by  $\gamma$  is shown in Figure 5 for 5 and 10 story structures by assuming fixed-base and soil flexibility ( $\bar{S}=3$  and  $h/r=3$ ), respectively. The effective period and the effective damping factor for the equivalent SDOF structure are calculated by the following equations, respectively:

$$\frac{\bar{T}}{T} = \sqrt{1 + \frac{\bar{K}}{K_y} \left(1 + \frac{K_y \bar{h}^2}{K_\theta}\right)} \tag{7}$$

$$\bar{\beta} = \beta_0 + \frac{0.05}{\left(\frac{\bar{T}}{T}\right)^3} \tag{8}$$

$\beta_0$  represents foundation-damping factor.  $\bar{T}$  and  $\bar{K}$  stand for fundamental period and the stiffness of the rigid base structure, respectively.  $K_y$  and  $K_\theta$  are the lateral and the rocking stiffness of the foundation, respectively. All of these parameters are calculated according to ASCE/SEI 7-10 [31]. According to the Figure 6, soil flexibility increases the

impact of higher modes on response of structures. This increment happens for both frequency contents. Another point is that by increasing the period of pulses, high frequency content becomes more effective.

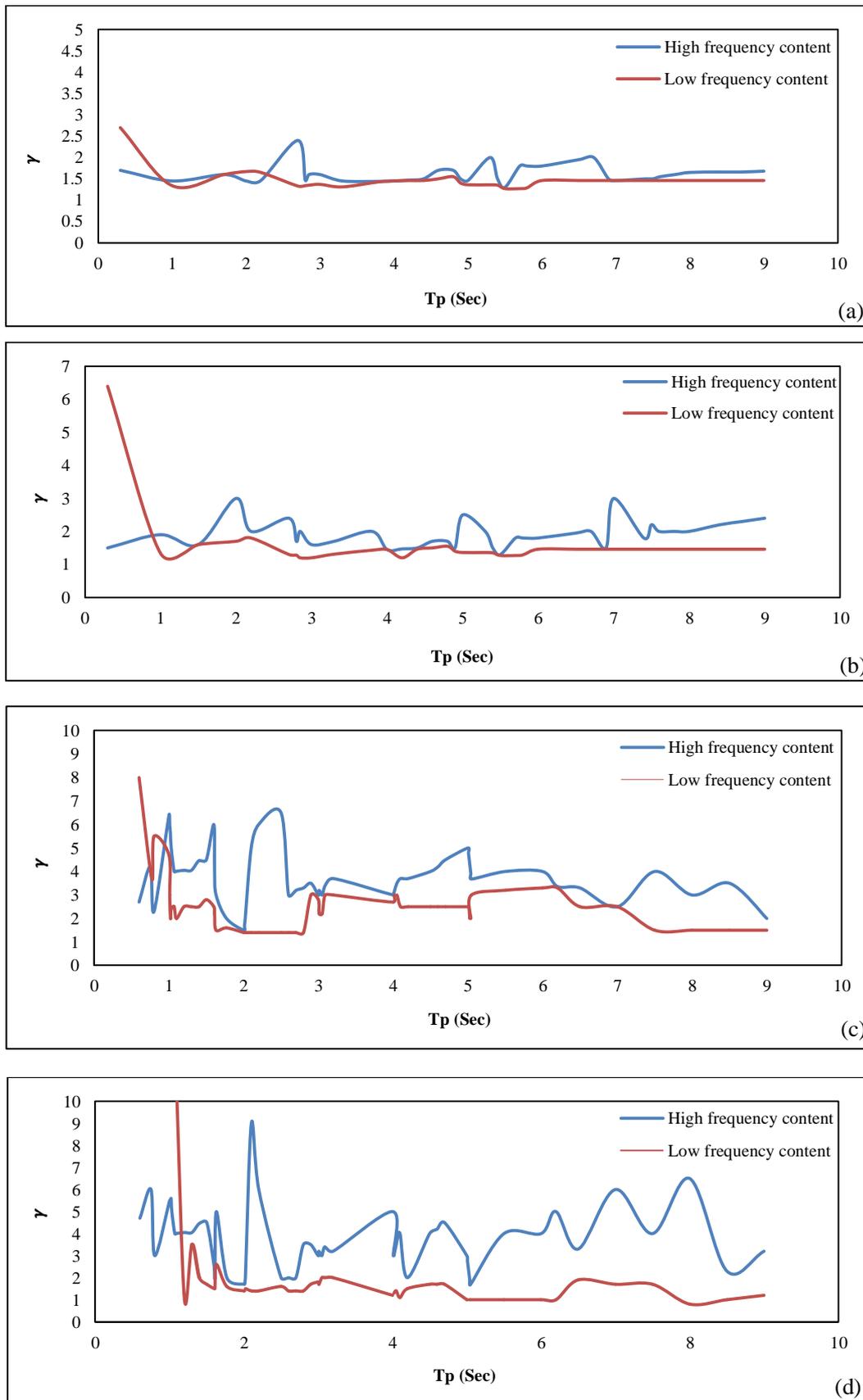


Figure 6. Maximum displacement of equivalent SDOF structures ( $\gamma$ ) (a) 5 story with fixed-base (b) 5 story with flexible base (c) 10 story with fixed-base (d) 10 story with flexible base

#### 5.4. Base Shear

The average values of base shear with fixed-base and considering SSI are illustrated in Figures 6 and 7. In most cases, SSI increases base shear especially in CBF system. As mentioned before, soil can affect the structural response in two ways. The first one is increasing the structural damping through two new sources of damping. As is obvious, increasing damping of structures can decrease response of structure in comparison with rigid-base models. The second one is soil flexibility, which can make reduction in the total stiffness of system. Unlike increasing the damping, reduce the structural stiffness can have detrimental effects on response of structures. Consequently, increase in structural damping and reduction in structural stiffness can cause a competition to determine the structural response. As noted previously, results are not presented in detail for each record separately, and they are limited to the average of all records. In some cases, SSI can decrease base shear (e.g. in 10 story structure subjected to Imperial Valley 06, Elcentro #7 SSI base shears were decreased 7% and 16% in MRFs and CBFs, respectively).

SSI effects on the base shear depend on the elastic response spectra. It seems that soil damping governs and controls the response of structure subjected to high frequency part of near-fault records, while it is the soil flexibility that controls the response of structure subjected to pulse type motion of near-fault records and increases response of structure due to the overall stiffness reduction. Proportional to the period of structures, SSI can transfer period of fixed-base structure from a point of descending in high frequency content to the point upward. In addition, it can transfer period of rigid-base structure from the frequency gap to the low frequency content, especially for structures with high values of  $\bar{S}$  and  $h/r$ . These can increase the structural response. Figure 9. illustrates the acceleration response spectra of Erzincan earthquake which is affected by the soil flexibility and damping.

The spectral acceleration of flexible-base structure,  $\bar{S}_a$  is obtained by calculating the spectrum by using the effective damping ratio,  $\beta_{eff}$ , at the corresponding elongated period,  $\bar{T}$ . Case 1 is fixed-base structure where its  $S_a$  is shown by red point. By considering soil flexibility, the stiffenes of structure reduces and case 1 shifts to case 2 according to period lengthening where its  $\bar{S}_a$  is shown by blue point. Using  $\bar{S}_a$  instead of  $S_a$  typically results higher base shear forces. On the other hand, moving from case 3 with black point  $S_a$  to case 4 with green point  $S_a$  reduces base shear demand. One of the main differences between far-fault and near-fault response spectra is existence of the frequency gap in near-fault response spectra which is specified by the green point in Figure 9.

Ghannad et.al [32] studied on elastic response spectra of SDOF models and SSI and proved that increase in structure-to-soil stiffness transfers response spectrum to the left side with lower periods and lower spectral ordinates. Furthermore, frequency gap between these two spectral peaks decreases in comparison with rigid-base systems. The reason lies in the fact that this period movement for P.T.R-dominated region is greater than B.G.R-dominated region. As a result, the elastic response of soil-structure system that the period of fixed-base structure falls in frequency gap (e.g. case 4 in figure 9), may be greater than expected base shear based on the fixed-base assumption. This effect is more significant in case of slender buildings where the radiation damping absorption is weaker in comparison with squatty buildings. Also, exciting higher modes on structural responses by considering soil flexibility can make difference in base shear value.

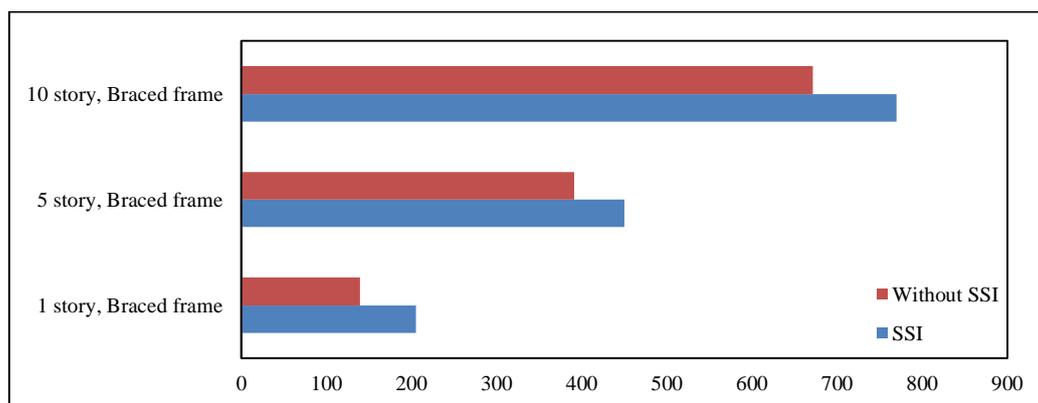


Figure 7. Base shear of braced frames

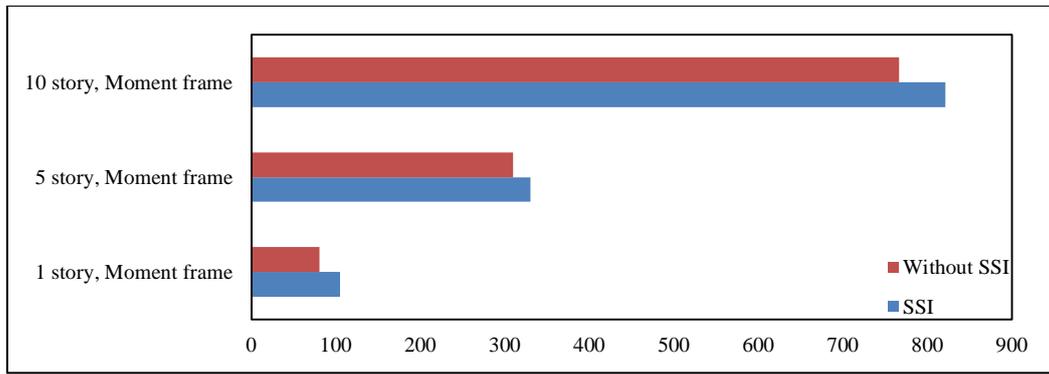


Figure 8. Base shear of moment frames

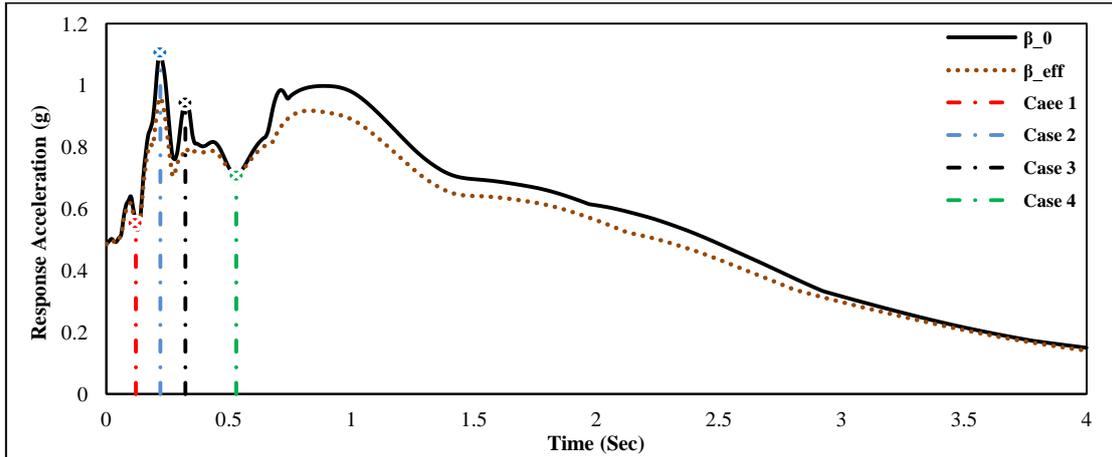


Figure 9. Inertial SSI effects on Erznican spectral acceleration (base shear) Associated with period lengthening and change in damping.

**5.5. Displacement Demand of Structures Under Different Frequency Content of Near-Fault Ground Motions**

In this part, attempts are made to investigate the effect of SSI on response of structures subjected to P.T.R and B.G.R components. Maximum displacement demands of 5 and 10 story structures with  $\bar{S} = 3$  and  $h/r = 1$  and 3 under different frequency contents of near-fault ground motions which have been decomposed by MATLAB program and the effects of SSI on them have been studied.

Figure 10. represents maximum displacement ratios under P.T.R and B.G.R components,  $(\frac{Ex.P(0)}{RGM(0)})$  and  $(\frac{Ex.P(a_0)}{RGM(a_0)})$ , for both rigid-base and flexible-base systems versus pulse period, respectively. Khoshnoudian et.al studied on response of nonlinear structures subjected to different frequency contents of near-fault ground motions [39]. In their research, they used two-dimensional shear buildings. In comparison with previous research, results for MDOF structures show same patterns.

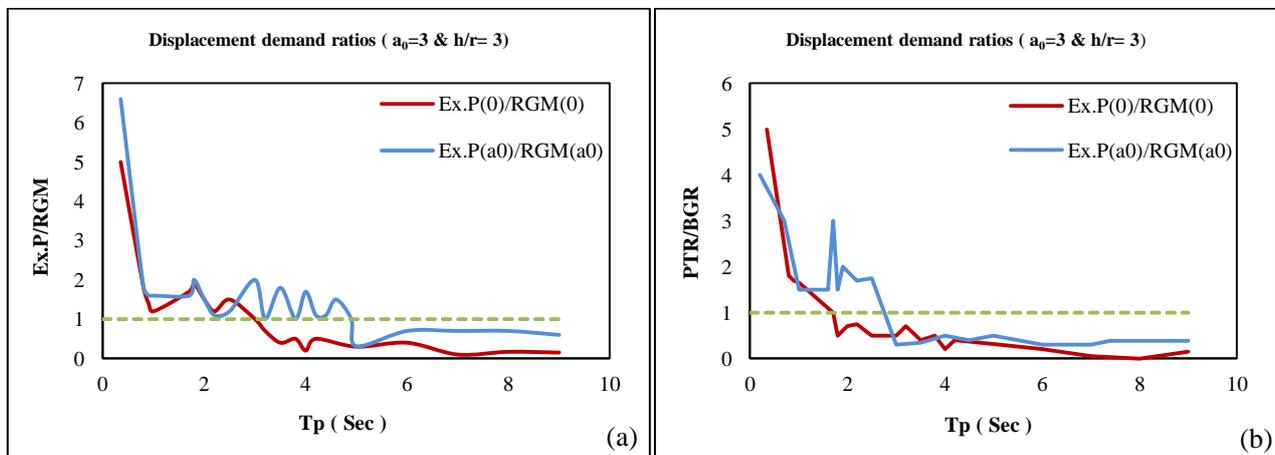


Figure 10. Maximum roof top displacement ratios of structures under P.T.R and B.G.R components with fixed base and flexible base systems (a) Five story model, (b) ten Story model

The green line means that displacements caused by P.T.R and B.G.R components are equal. Points which are higher than the green line are influenced more by P.T.R quota as opposed to the lower points which are affected by B.G.R quota. With increasing in pulse period, a descending pattern occurs and B.G.R quota on structural response becomes larger. Soil flexibility can move the intersection point to the higher periods. This is due to the fact that SSI decreases structural stiffness and elongates period of the structure. As a consequence, structure is affected by more pulses. Displacement demand ratios  $\frac{PTR(a_0)}{PTR(0)}$ ,  $\frac{BGR(a_0)}{BGR(0)}$  are illustrated in Figures 11 to 14.

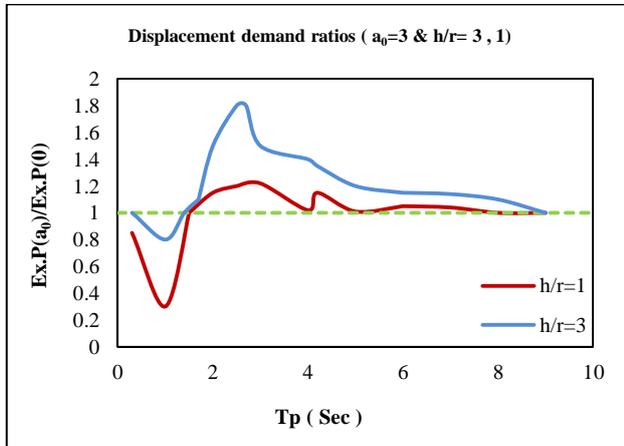


Figure 11. Effects of SSI on response of 5 story structure Under P.T.R component.

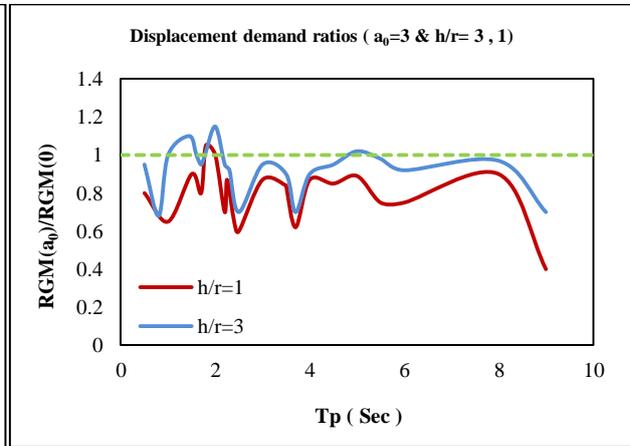


Figure 12. Effects of SSI on response of 5 story structure Under B.G.R component

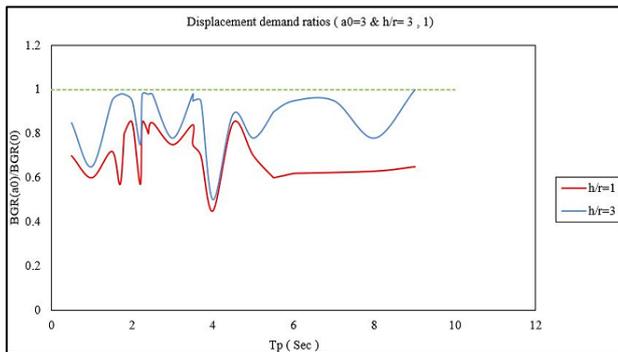


Figure 13. Effects of SSI on response of 10 story structure Under P.T.R component.

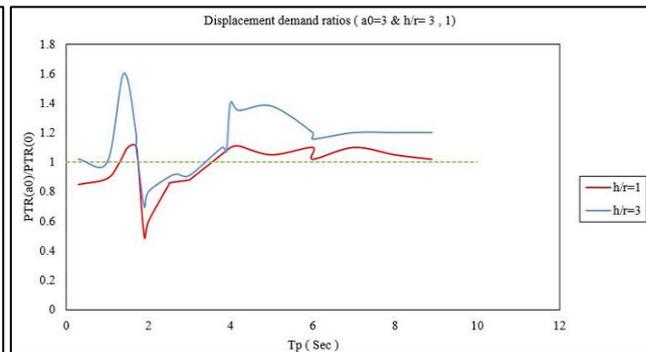


Figure 14. Effects of SSI on response of 10 story structure Under B.G.R component

For all cases, the slender structures experiences higher displacement as opposed to the squatty ones. This is due to the less soil damping absorption by the slender buildings. Another point is that when structures are subjected to P.T.R component displacement demand ratios are more than one in most periods. By contrast, displacement demand ratios related to B.G.R component are less than one. It means that, SSI decreases story demand when structures are under B.G.R component because of hysteric and wave dampings of the soil. On the other hand, when structures are under P.T.R component, story demand is increases due to the period elongation and stiffness reduction of structures.

### 5.6. Performance of Structures

Evaluation criteria of structure performance in non-linear analysis, is performance status of lateral resisting members in their behavior curves. For studying the effects of SSI on performance of members, total number of plastic hinges formed in different performance levels by assuming fixed-base and SSI effects are illustrated in Figure 15 to 17. Also, for better understanding about the capability of SSI in performance level of structural members, DR\* index was used. Figure 18 to 20. illustrate DR\* Index.

\* DR: Damage Ratio

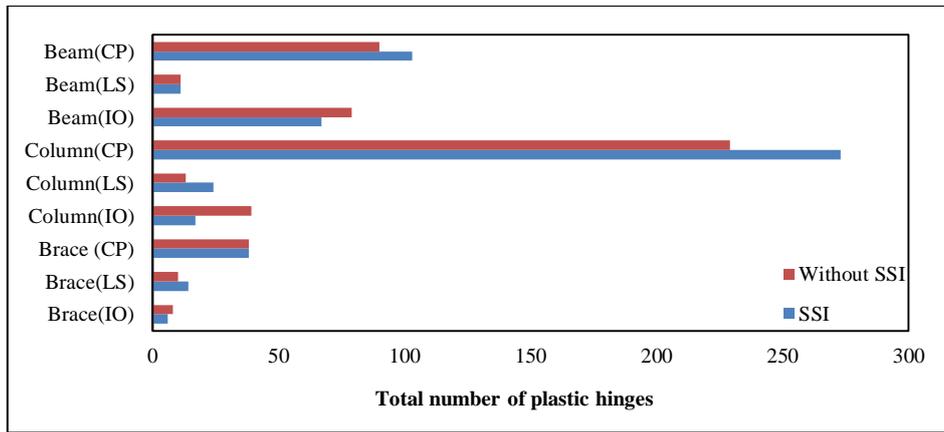


Figure 15. Total number of plastic hinges in members of 1 story structure

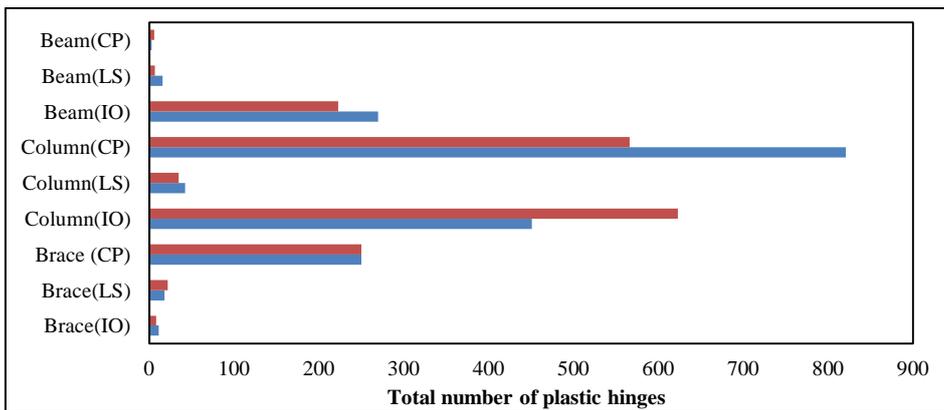


Figure 16. Total number of plastic hinges in members of 5 story structure

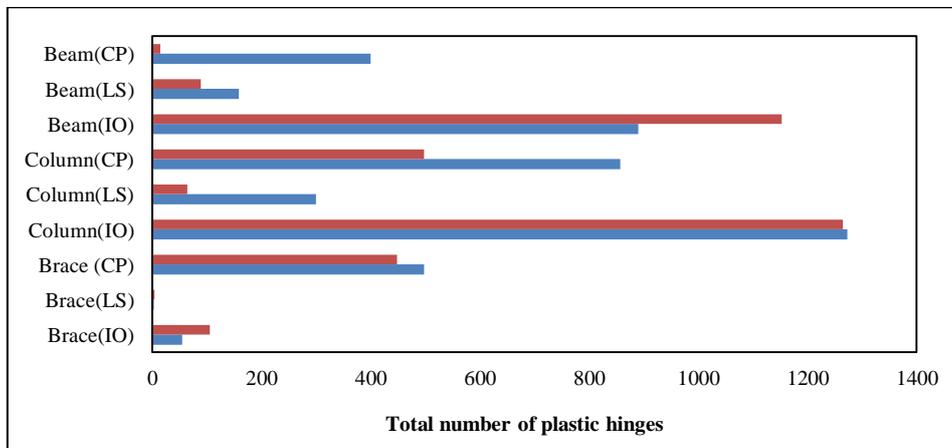


Figure 17. Total number of plastic hinges in members of 10 story structure



Figure 18. DR index of 1 story structure

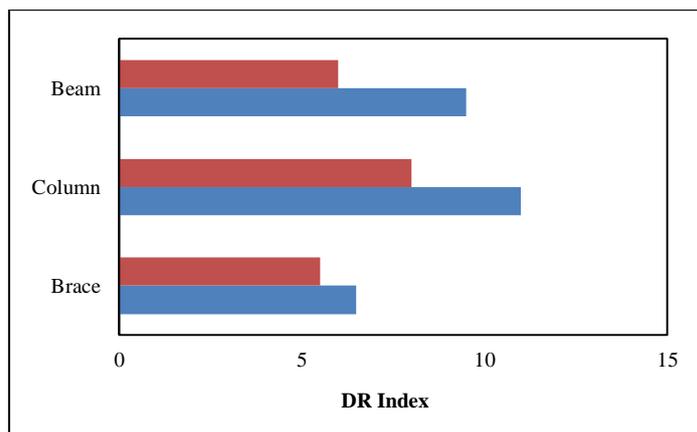


Figure 19. DR index of 5 story structure

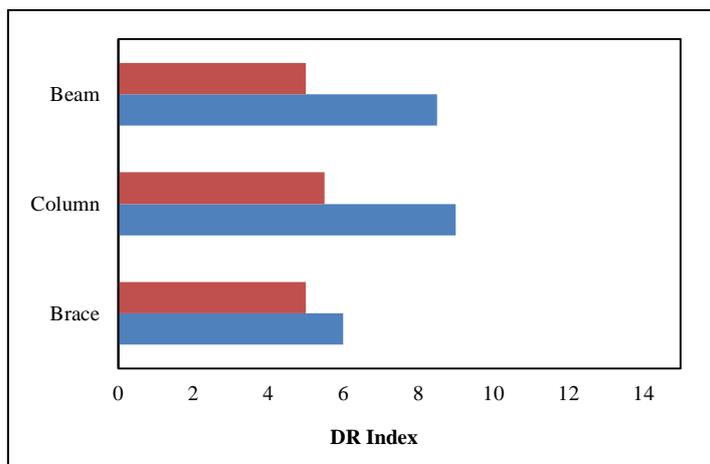


Figure 20. DR index of 10 story structure

Figures 15 to 20. show that SSI can cause the structures experience more non-linear behaviour. The most severe effect of soil is on beams and columns, respectively. By increasing aspect ratio, non-linear behaviour of beams and columns become more intense. This is due to the increment of base shear, the second order effects and aspect ratio and lower radiation damping absorption by structures. In other words, by increasing the height of the structure, the second order effects become more severe, especially in lower columns and also structure absorbs less soil damping. This means reduction of columns stiffness and more lateral load absorption by beams. Another reason is the effect of inertial SSI on ductility demands of structures at specific strength of the structure. For further investigation about this issue, it needs to scrutinize more about the behaviour of structures on soft bed. For this reason, ductility demands of structures with different fixed-base period on soft soil are compared with the target ductility of fixed-base structures. Such a comparison is made in Figure 21. for Imperial Valley-06 ground motion, recorded at Aeropuerto Mexicali station at ductility level  $\mu = 6$ . The results are illustrated for different values of  $\bar{S}$  and  $h/r$ . The abscissa is the first-mode period of the fixed-base structure,  $T_{fixed-base}$ , and the vertical axis is ductility demand.

Analysis for other ground motions led to the similar results. There is a threshold period which can be explored by the  $\frac{T}{T_p}$  ratio. In the case of  $\frac{T}{T_p} < 1$  ductility of the structure by considering soil flexibility is more than fixed-base structures. The trend shows reverse results for structures with natural periods longer than pulse period. The situation is getting worse for slender structures and larger values of  $\bar{S}$ . For earthquakes with higher pulse period the threshold period shifts to the right. As a conclusion, the effect of SSI in terms of structural ductility can be beneficial for long period structures; on the other hand, ignoring SSI leads to illogical results for short period structures

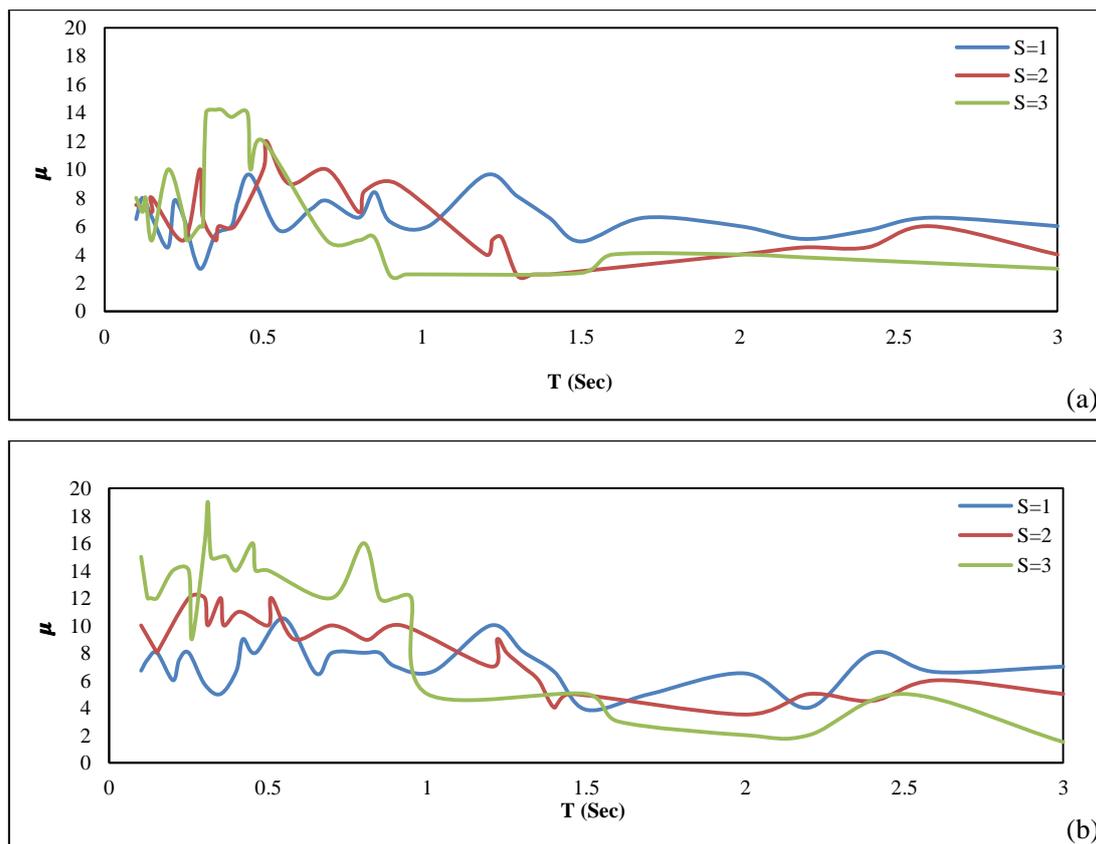


Figure 21. The inertial SSI effect on ductility demand of structures subjected to Imperial Valley-06 ground motion, recorded at Aeropuerto Mexicali station. (a)  $\frac{h}{r} = 1$ , (b)  $\frac{h}{r} = 3$

## 6. Conclusion

In this paper the effect of soil-structure interaction on non-linear behavior of multi-story steel structures subjected to near-fault ground motions is investigated. Cone method was used for modeling and analysis soil environment. Wavelet analysis was used to decompose frequency contents of near-fault ground motions. Comparison of obtained results for fixed-base structures and soil-structure systems indicates that soil flexibility can increase the impact of higher modes on response of structures subjected to both P.T.R and B.G.R components. Also, neglecting soil flexibility can cause irreparable results on base shear. Soil flexibility can impose sever non-linear behavior in structures, especially in beams. Also soil damping can have beneficial effects for low rise structures while soil flexibility can cause detrimental effect on structures with high values of aspect-ratios. Soft soil increases the ductility demand of structures with period less than pulse period and can be beneficial for structures with period more than a threshold period.

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