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An evaluation of the seismic response of symmetric steel space buildings

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Abstract. This paper evaluates the seismic response of three dimensional steel space buildings using the spread plastic hinge approach. A numerical study was carried out in which a sample steel space building was selected for pushover analysis and incremental nonlinear dynamic time history analysis. For the nonlinear analysis, three earthquake acceleration records were selected to ensure compatibility with the design spectrum defined in the Turkish Earthquake Code. The interstorey drift, capacity curve, maximum responses and dynamic pushover curves of the building were obtained. The analysis results were compared and good correlation was obtained between the idealized dynamic analyses envelopes with and static pushover curves for the selected building. As a result to more accurately account response of steel buildings, dynamic pushover envelopes can be obtained and compared with static pushover curve of the building.

Keywords: spread plastic hinge; static pushover analysis; steel building; incremental nonlinear dynamic time history analysis

1. Introduction

Steel buildings can absorb large amount of energy caused by seismic action due to it has high strength and plastic deformation capacity. Since, the weight of steel buildings is less than that of reinforced buildings; these buildings are subjected to smaller earthquake loads than reinforced concrete buildings. As a result, this type of building is ideal for regions of high seismicity, but accurately determining the nonlinear seismic response of the buildings under earthquake conditions is critical. To more accurately account for the behaviour of nonlinear material under combined bending and axial loads, structural elements have been modelled using the spread plastic hinge approach (Hall and Challa 1995). The hinge model based on a finite element model (Challa 1992) accounts for the distributed plasticity over the length and cross section of structural element. In this model in which the cross-section is divided into fibers is applied in the study to estimate the second-order inelastic response of space steel buildings instead of the usual plastic hinge approach using a specific yield surface. Fig. 1 shows the fiber modelling of a box and I steel profiles.

The hinge model was used by researchers when evaluating reinforced concrete and steel

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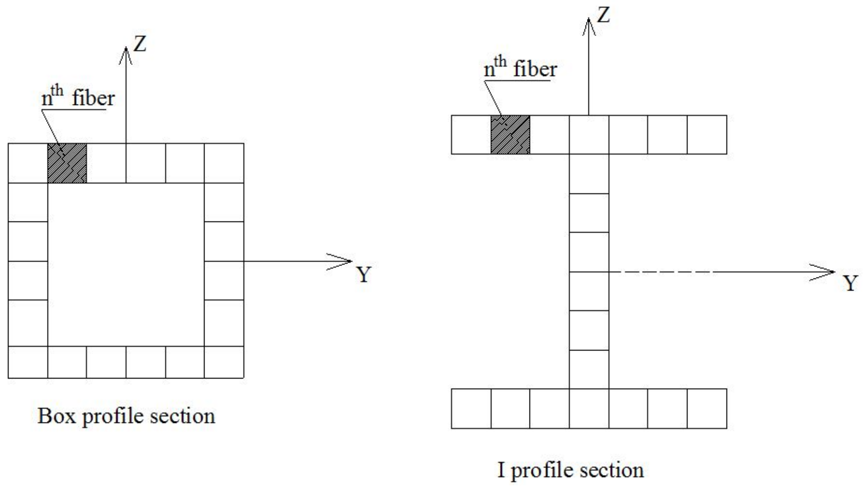

Fig. 1 Typical fiber models of the steel profiles

buildings. Mwafy and Elnashai (2001) performed nonlinear static and dynamic analysis of reinforced concrete structures using the spread plastic hinge model while Duan and Hueste (2012)evaluated the seismic behaviour of a five storey reinforced concrete building that was designed according to the requirements of the Chinese seismic code using the distributed hinge model for the analysis. Kwon and Kim (2010) assessed a reinforced concrete building, which was exposed to damage during the 2007 Pisco-Chincha Earthquake in Peru, by performing nonlinear analysis of this building using the spread hinge model. Sarno and Manfredi (2010) performed pushover and dynamic analysis for both constructed and retrofitted buildings to investigate the efficiency of buckling restrained braces and used the spread element model in nonlinear analysis, while Yön (2014) studied nonlinear behaviour of reinforced concrete buildings using this hinge model. Yön and Calayır (2014) investigated effects of confinement reinforcement and concrete strength on nonlinear behaviour of reinforced concrete buildings by using the spread hinge model. Yön and Calayır (2015) assessed the soil effect on the nonlinear response of reinforced concrete buildings. Carvalho *et al*. (2013) compared the various hinge model approaches by performing nonlinear static and dynamic analysis of a reinforced concrete structure, and Huu and Kim (2009) carried out a study investigating the practical advanced analysis of steel frames using the spread hinge approach. Huu *et al*. (2007) investigated the nonlinear analysis of space steel frames using the plastic hinge model, while Jiang *et al*. (2002) investigated the behaviour of three dimensional (3D) steel buildings using this model. Krishnan and Hall (2006) used the hinge approach for their studies, which included steel frame buildings.

This paper evaluates the seismic response of space steel buildings by performing pushover analyses and incremental nonlinear dynamic time history analyses according to different seismicity levels and local site conditions defined in the Turkish Earthquake Code (TEC 2007). For the nonlinear analyses the SeismoStruct programme which can simulate the inelastic response of structural systems subjected to static and dynamic loads was employed. The SeismoArtif programme was used to scale earthquake acceleration records to the design spectrums and the SeismoSignal software was applied to obtain the predominant periods of each selected earthquake record.

2. Numerical application

In the numerical application, such as with high ductility, a 3D space symmetric steel building was selected and pushover and incremental nonlinear dynamic time history analysis was performed. The selected building has six storeys with three and four bays in the x and y direction, respectively. The storey heights of the building are 3 m and the total height of the building is 18 m. When designing the structural system St37 steel and IPE profiles were used. The properties of St37 steel are shown in Table 1. Also, the structural element dimensions of the building are given in Table 2. For the nonlinear analyses, the Menegotto-Pinto (Menegotto and Pinto 1973) model, with an isotropic strain-hardening material model and kinematic strain-hardening was used. For the incremental nonlinear dynamic analysis, the selected earthquake acceleration records were adjusted to be compatible with the design spectra according to the seismicity level and local site

Table 1 Mechanical properties of St37 steel (TS648)

	Mpa
Yield stress	235
Elasticity module	206182

Table 2 Beam and columns dimensions

Storeys	Columns	X-Direction		Y-Direction	
		Exterior-beams	İnterior-beams	Exterior-beams	İnterior-beams
1-2-3	IPE600	IPE360	IPE450	IPE300	IPE300
4-5	IPE550				
6	IPE500				

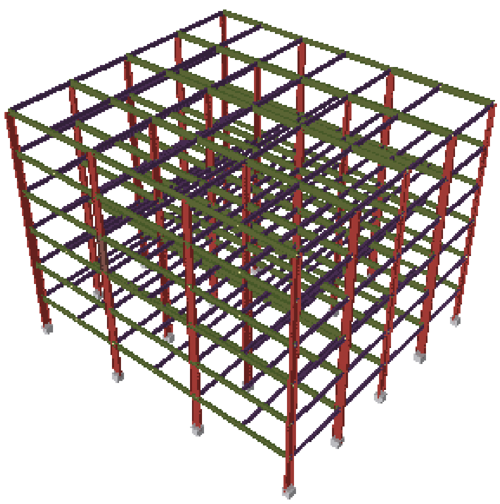

Fig. 2 Selected 3D symmetric steel building

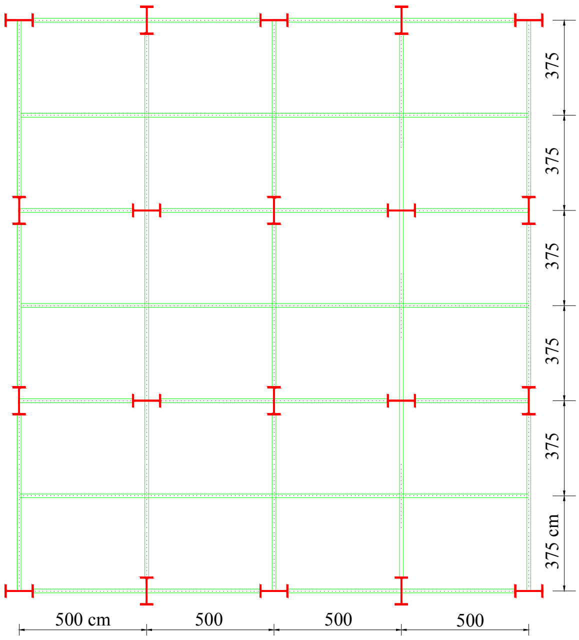

Fig. 3 Plan of the building

conditions. Interstorey drifts, capacity curves, maximum responses and dynamic pushover envelopes of the building were obtained and compared according to these adjusted records. For the onlinear analysis, the spread hinge approach was applied. To calculate the element forces and the stress–strain relationship used to monitor each section, four Gauss integration points were selected. The 3D view and plan of the building are illustrated in Figs. 2 and 3, respectively. The base of the building was assumed to be rigidly fixed and the soil compliance and damping properties were not taken into account.

2.1 Earthquake parameters and soil conditions

Properties of the selected earthquake accelerations are given in Table 3. The seismic records were obtained from the PEER Strong Motion Database and these values have been scaled in frequency to ensure compatibility with the target design spectrum according to seismic zones and local site conditions in the TEC. For the nonlinear dynamic analysis, the same seismic values were scaled and used x and y directions.

Table 3 Selected earthquake acceleration records for dynamic analysis

Earthquakes	Station	Date	Magnitude	PGA (g)
Kocaeli	Düzce	August 17, 1999	7.4	0.358
Loma Prieta	Corralitos	October 18, 1989	6.9	0.644
Imperial Valley	El Centro Array	May 19, 1940	7.0	0.313

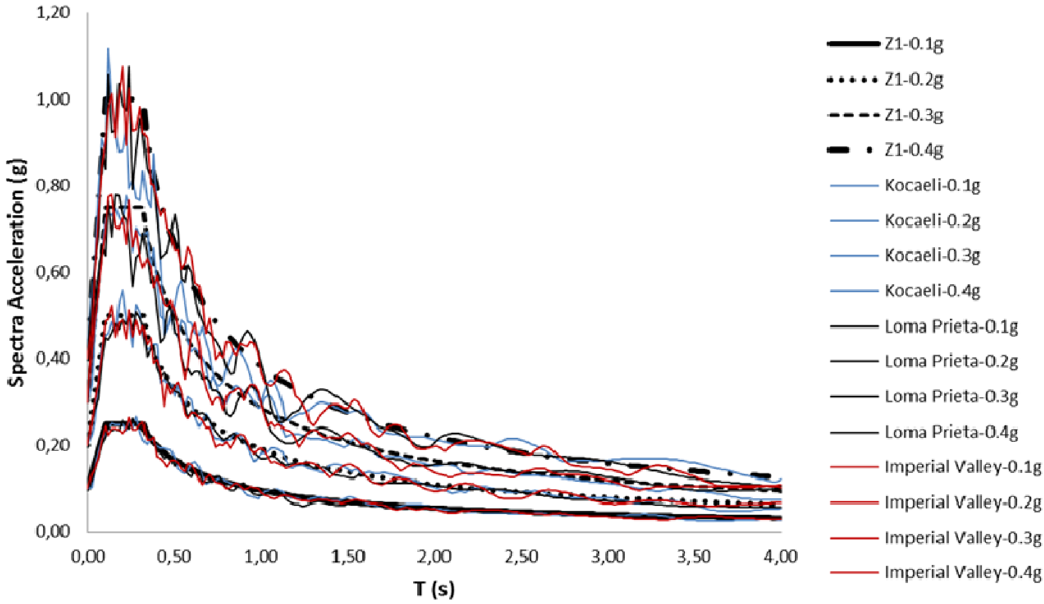

Fig. 4 Response spectra of the earthquake acceleration records scaled according to the elastic design spectrum for the Z1 soil class with ground accelerations

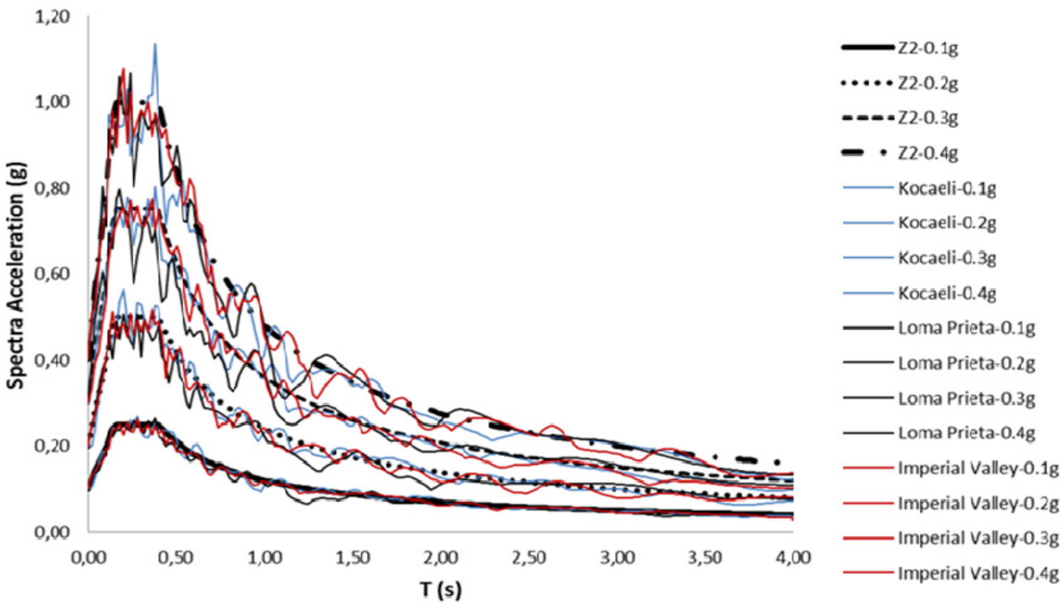

Fig. 5 Response spectra of the earthquake acceleration records scaled according to the elastic design spectrum for the Z2 soil class with ground accelerations

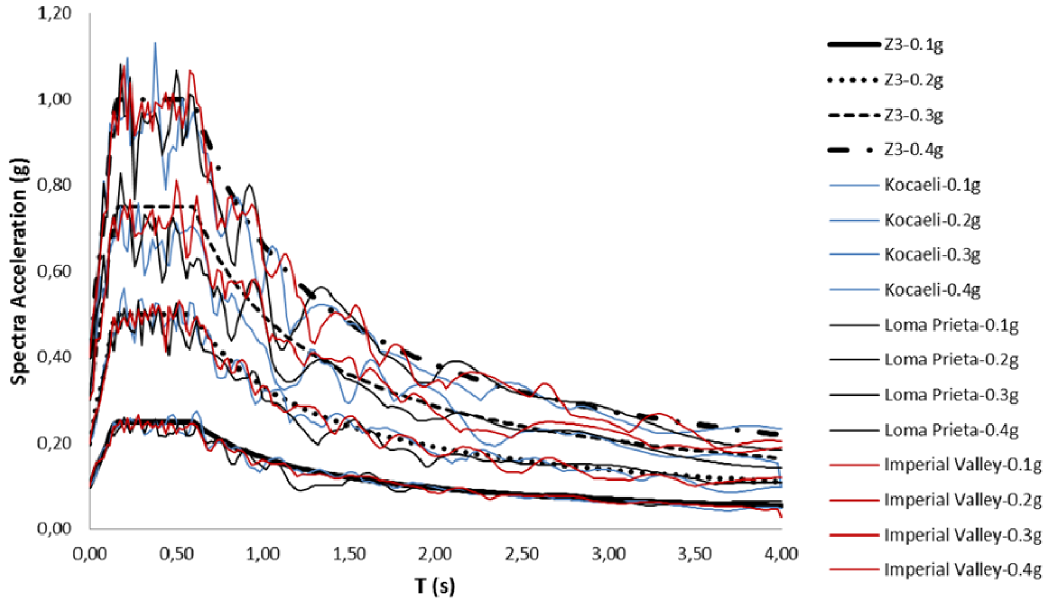

Fig. 6 Response spectra of the earthquake acceleration records scaled according to the elastic design spectrum for the Z3 soil class with ground accelerations

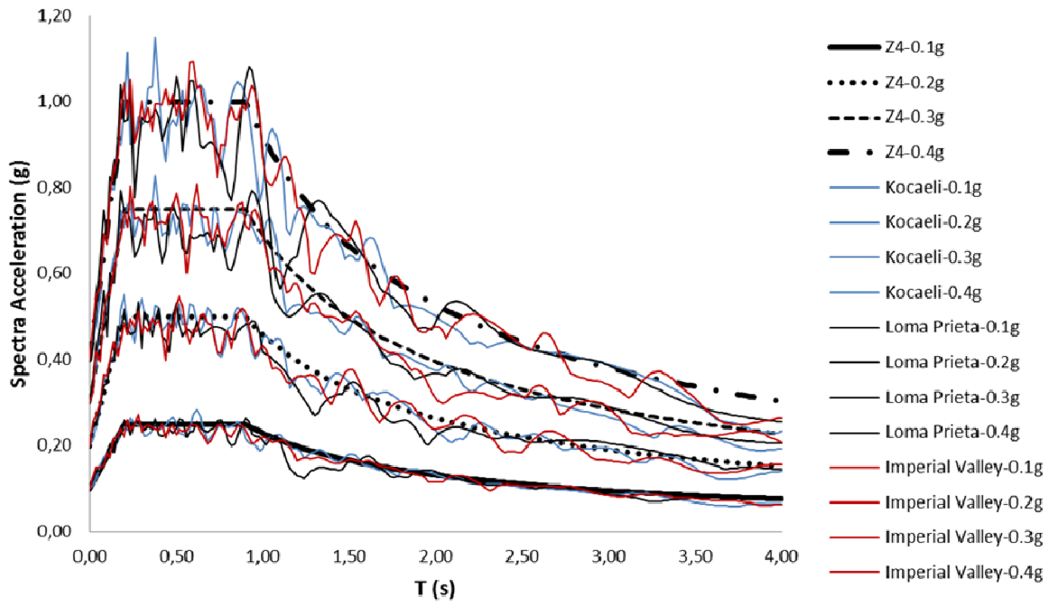

Fig. 7 Response spectra of the earthquake acceleration records scaled according to the elastic design spectrum for the Z4 soil class with ground accelerations

The TEC defines four soil classes from Z1 to Z4 and four ground acceleration from 0.1 g to 0.4 g for buildings. The design spectra were determined by multiplying the elastic spectrum with the ground acceleration and the building importance factor was calculated from the local site conditions are given in Figs. 4-7, ensuring that the building response will have been taken into consideration.

2.2 Results and discussion

The interstorey drift of the building for various ground accelerations and soil classes are illustrated in Figs. 8-10 in the *x* direction. The ratio of the interstorey drift typically increases from Z1 to Z4 for the same ground acceleration. However, the increase in these ratios differs according to the earthquake characteristics. For the *x* direction, interstorey drift ratios for Z4-0.4 g, Z3-0.4 g, Z4-0.3 g and Z3-0.3 g exceed the effective interstorey drift ratio, which was determined to be 2% in the TEC for scaled Kocaeli earthquake. For Z4-0.4 g, the interstory drift ratio exceeds 3.5%. For scaled LomaPreita earthquake, the interstorey drift ratios of Z4-0.4 g, Z3-0.4 g and Z4-0.3 g exceed the 2% limit ratio. For this scaled earthquake, the most intersorey drift ratio is around 4.0%. For scaled Imperial Valley, interstorey drift ratios for Z4-0.4 g and Z3-0.4 g exceed the 2% limit ratio earthquake. The most interstorey drift ratio occurs in case of Z4-0.4 g and this ratio is around 3%.
The interstorey drifts of the building are given for various ground accelerations and soil classes as shown in Figs. 11-13 in the *y* direction. For *y* direction, interstory drift ratios for Z4-0.4 g, Z3-0.4 g,

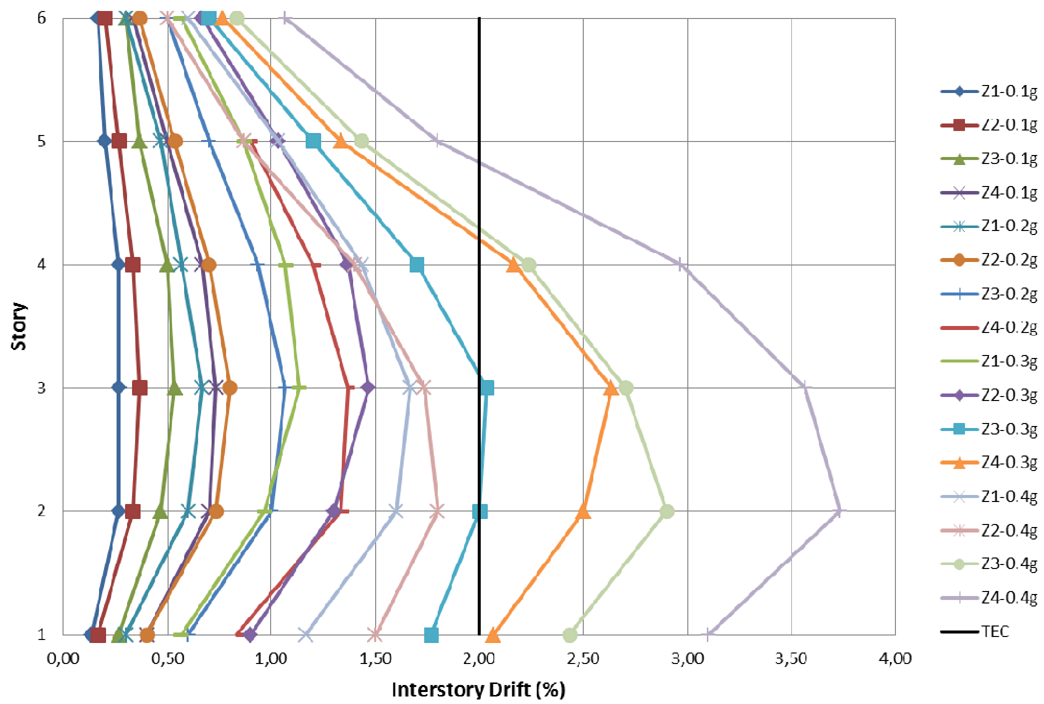

Fig. 8 Interstorey drifts obtained from nonlinear dynamic time history analysis using scaled Kocaeli earthquake in the *x* direction

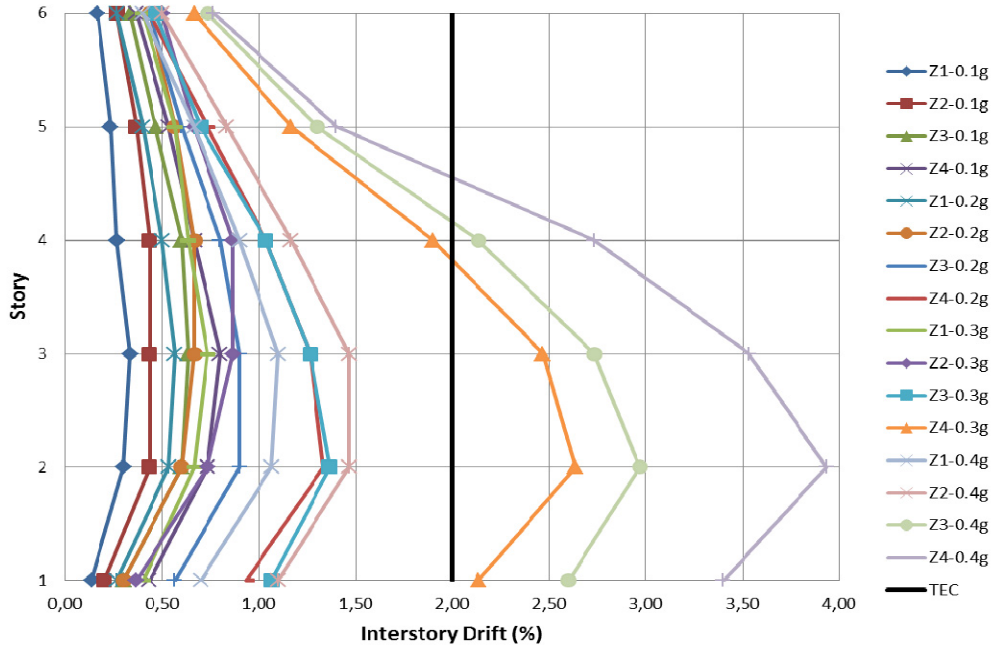

Fig. 9 Interstorey drifts obtained from nonlinear dynamic time history analysis using scaled LomaPrieta earthquake in the *x* direction

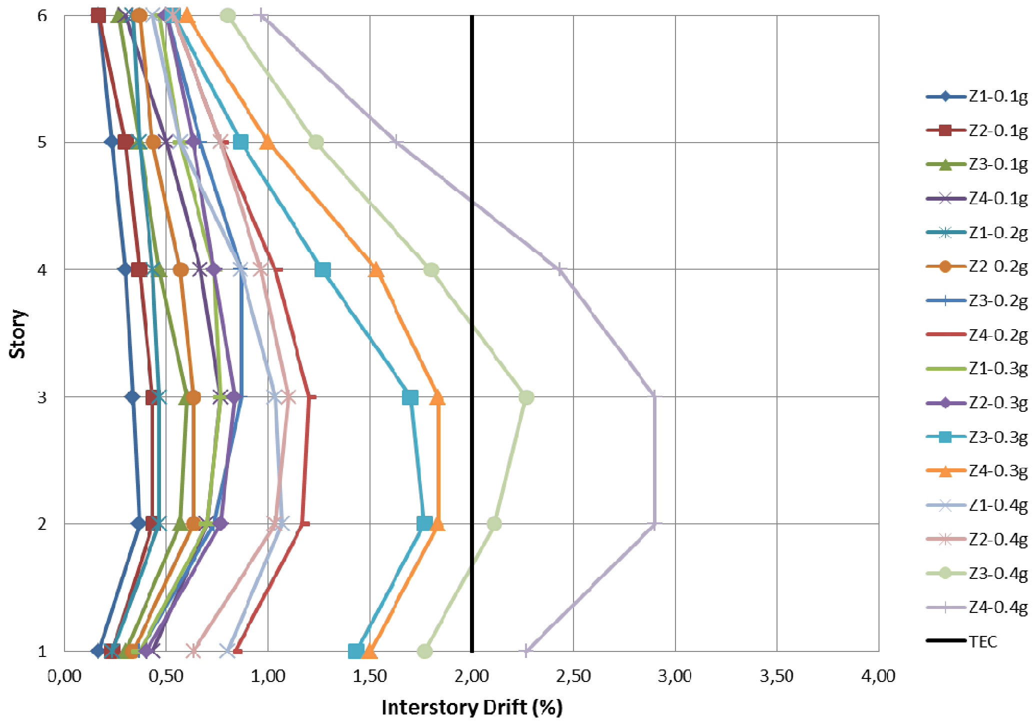

Fig. 10 Interstorey drifts obtained from nonlinear dynamic time history analysis using scaled Imperial Valley earthquake in the *x* direction

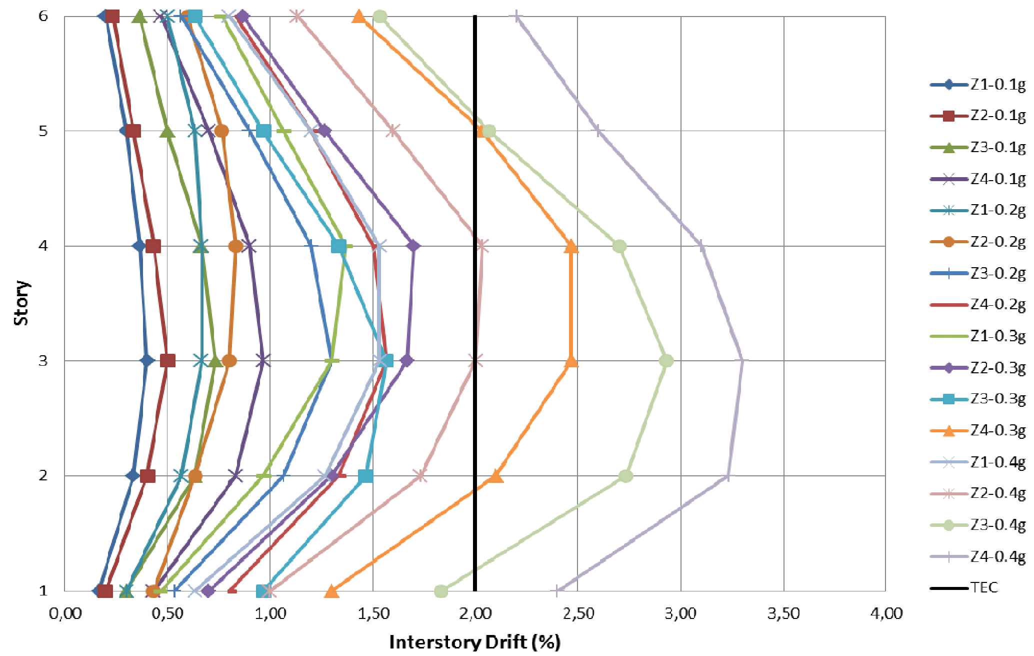

Fig. 11 Interstorey drifts obtained from nonlinear dynamic time history analysis using scaled Kocaeli earthquake in the *y* direction

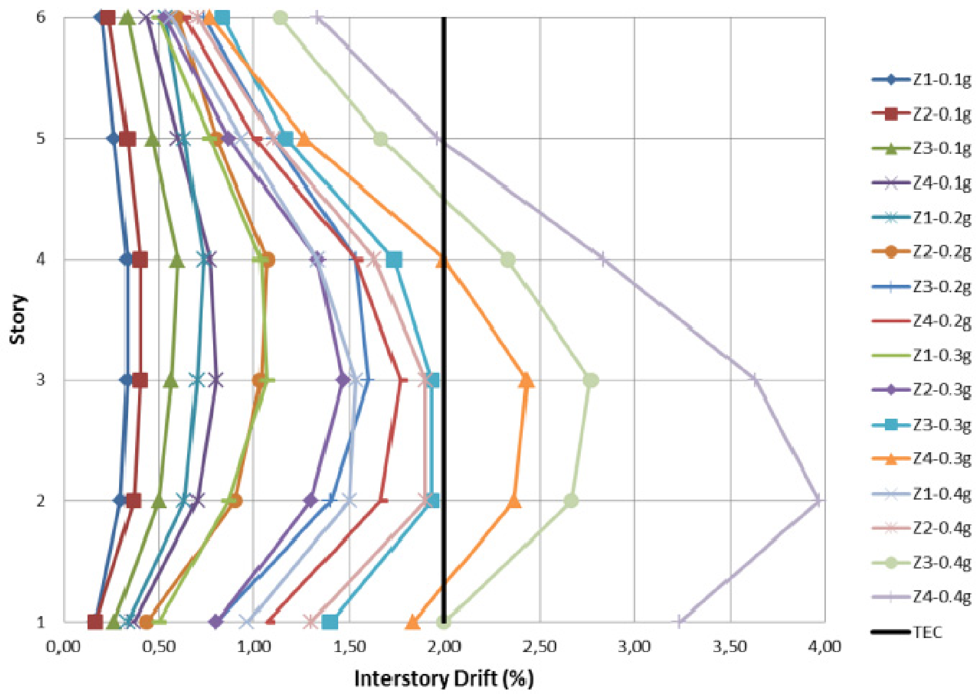

Fig. 12 Interstorey drifts obtained from nonlinear dynamic time history analysis using scaled LomaPrieta earthquake in the *y* direction

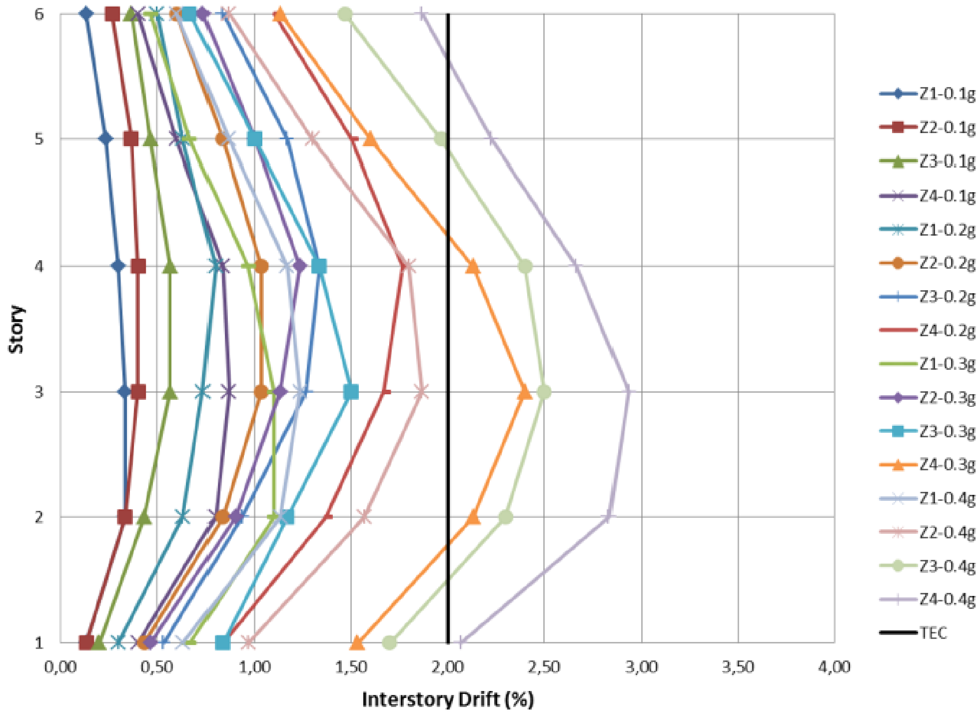

Fig. 13 Interstorey drifts obtained from nonlinear dynamic time history analysis using scaled Imperial Valley earthquake for in the *y* direction

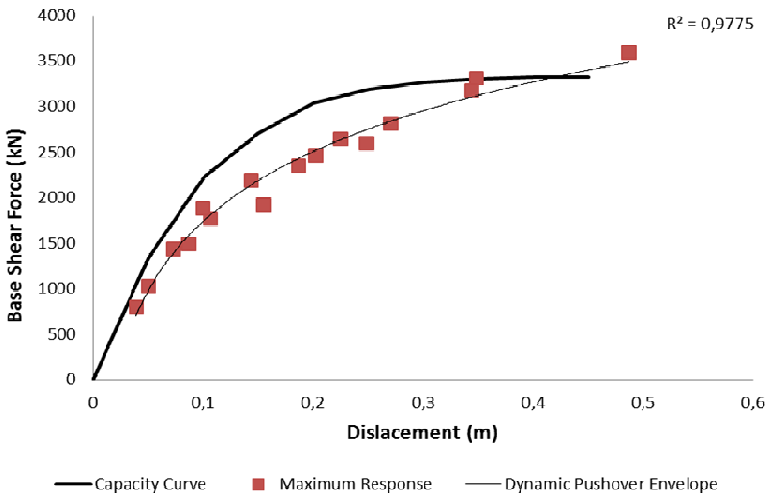

Fig. 14 Comparison of the maximum responses of scaled Kocaeli earthquake with the capacity curve of the building an dynamic pushover envelope in the *x* direction

Z4-0.3 g exceed the effective interstorey drift ratio which determined as 2% in the TEC for scaled Kocaeli, LomaPrieta and Imperial Valley earthquakes. The most interstory drift ratios occurred in case of Z4-0.4 g and these ratios were obtained as approximately 3.25%, 4.0% and 3.0% for scaled Kocaeli, LomaPrieta and Imperial Valley earthquakes, respectively. According to these figures, the ratio of interstorey drifts increase from Z1 to Z4 for the same ground acceleration. The ratios increments differ according to earthquake characteristics. But the maximum interstorey drift ratios for soft soils and high peak ground accelerations exceed the limit ratio given in the TEC.

A comparison of the capacity curve of the selected building and the maximum responses of the incremental dynamic time history analysis is presented in Figs. 14-16 in the *x* direction and Figs.

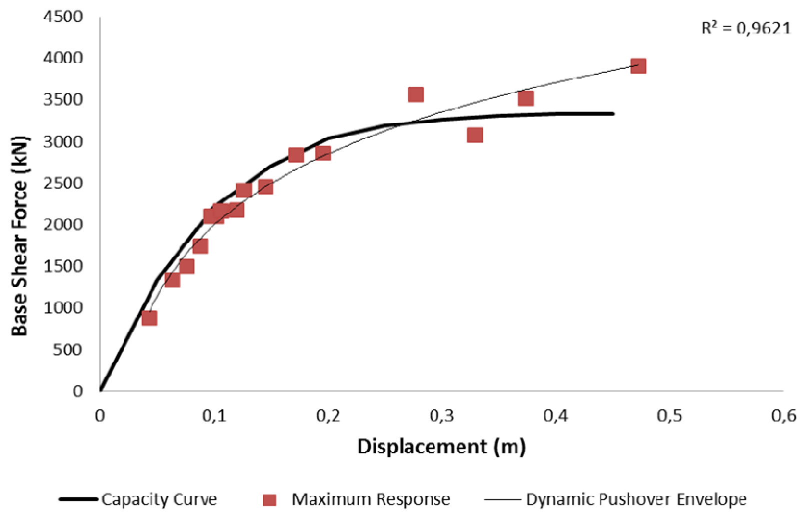

Fig. 15 Comparison of the maximum responses of scaled LomaPrieta earthquake with the capacity curve of the building and dynamic pushover envelope in the *x* direction

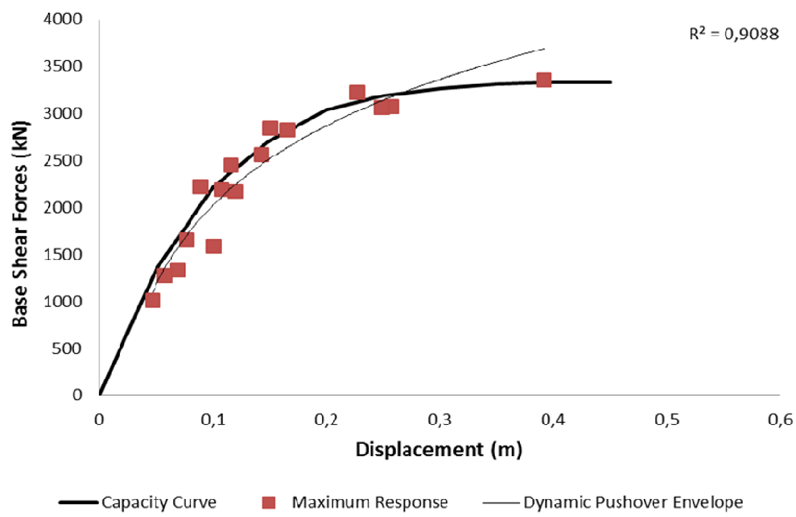

Fig. 16 Comparison of the maximum responses of scaled Imperial Valley earthquake with capacity curve of the building and dynamic pushover envelope in the *x* direction

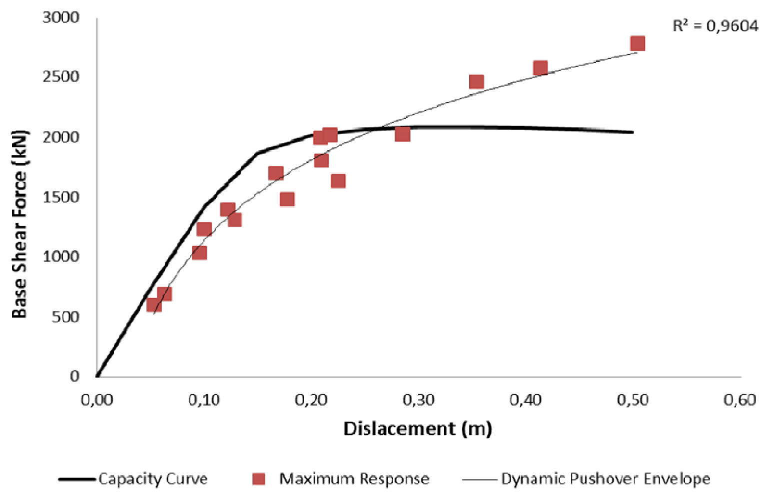

Fig. 17 Comparison of the maximum responses of scaled Kocaeli earthquake with the capacity curve of the building and dynamic pushover envelope in the *y* direction

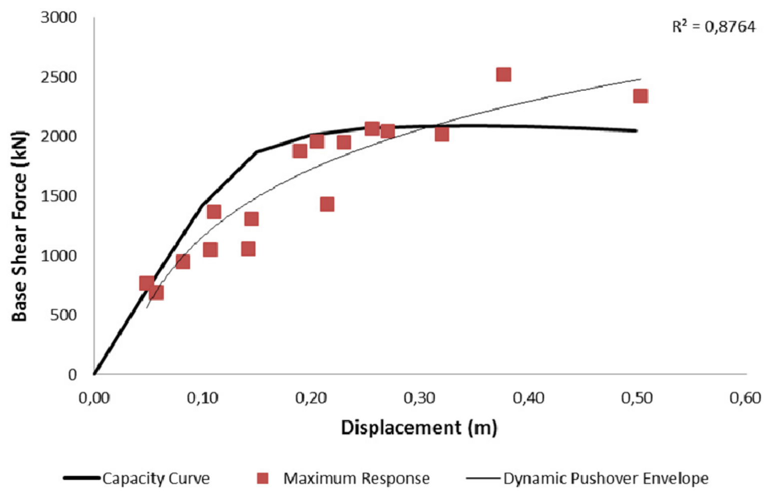

Fig. 18 Comparison of the maximum responses of scaled LomaPrieta earthquake with the capacity curve of the building and dynamic pushover envelope in the *y* direction

17-19 in the *y* direction, respectively. Also, the dynamic pushover envelopes are given in these figures. However, these responses for stiff soils and low peak ground accelerations are too close orunder the capacity curve of the building for all scaled earthquakes. However, the maximum responses for soft soils and high peak ground accelerations exceed the capacity of the building. The actual response of the selected building is shown in these figures. Seismic actions follow the same trend and shape of the pushover curve; the correlation coefficient values, are approximately 0.90 for *x* direction. In addition to this, for *y* direction seismic actions obtained from the scaled earthquakes follow the same trend and shape to that of the pushover curve, similar to that in the *x* direction. The correlation coefficient values are above or close to 0.90.

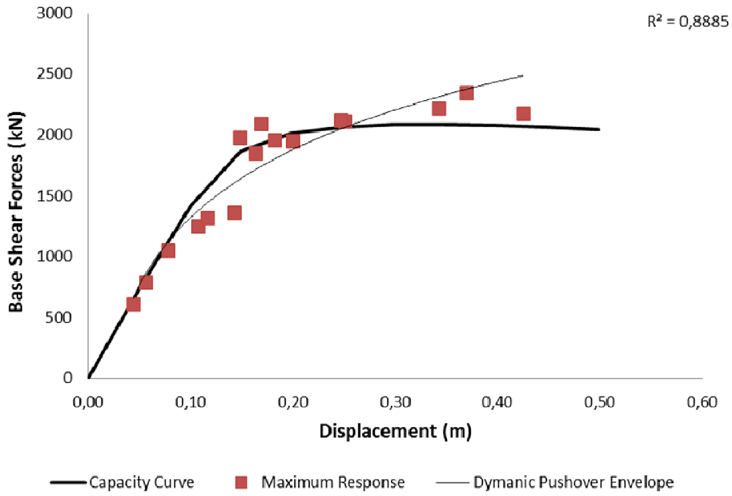

Fig. 19 Comparison of the maximum responses of scaled Imperial Valley earthquake with the capacity curve of the building and dynamic pushover envelope in the *y* direction

3. Conclusions

This paper evaluates the seismic response of a three dimensional steel space building using the spread plastic hinge approach. For numerical study, high ductility three dimensional steel space building was selected. Pushover analyses and incremental nonlinear dynamic time history analyses were performed of the building. For the nonlinear analysis three selected earthquake acceleration records were adjusted to ensure they were compatible with the design spectrum defined in Turkish Earthquake Code (TEC) by considering the various ground accelerations and local soil conditions. The results highlight the interstory drift, capacity curve, maximum responses and dynamic pushover curves of the building.

- The seismicity levels can be more critical than soft soil classes for interstorey drift ratios. In addition to this, in the case of lower amplitude ground motions, the soil classes can be more significant than the seismicity levels for the selected building model.
- Good correlation was obtained between the idealized dynamic analyses envelopes with and static pushover curves for the selected building. Maximum responses obtained from stiff soils and low peak ground accelerations are too close and under the capacity curve of building for selected earthquakes. However, the maximum response for soft soils and high peak ground accelerations exceed the capacity of building.

According to the findings, seismic zones and local soil conditions were determined to affect the nonlinear response of steel buildings, considerably. To determinate the accurate behaviour of steel buildings, the dynamic pushover envelopes can be obtained and compared with static pushover curve. Thus, the seismic response of structural system should be determined.

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