Inelastic behavior of steel special moment frames under near fault ground motions

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Abstract—**This work is conducted to investigate the effects of near fault ground motions on the peak story drift ratios and structural elements' displacements. Two sets, each consists of seven near fault ground motion records are prepared. The records of each set are scaled in order that their average spectrum approaches to the design response spectrum by following the rules provided in ASCE/SEI 07-10. One of the sets consists of records with pulses whereas the other is established with records without pulses. A ten story steel special moment frame is designed to the Load and Resistance Factor Design of the ANSI/AISC 360-10. Applied capacity based design principles for providing global ductility are along with ANSI/AISC 341-10. The loads and load combinations are compatible with ASCE/SEI 7-10. The results indicate that although the peak values do not change much, the dispersion of the drift ratios and the element displacements are considerably higher for the set of pulse type records.**

Keywords—**selecting and scaling of earthquake records, steel special moment frame, near fault ground motions, pulse type records**

I. Introduction

Seismic performance assessment of building structures with the use of nonlinear dynamic analysis is becoming more prevalent with the advancements in computer technology and with the accumulated data resources concerning the ground motion records and modeling the structural elements' behavior. However, the significant challenge to overcome for both practicing engineers and researchers is to establish the ground motion sets to be used. Unfortunately, there is currently no consensus in the earthquake engineering community on how to appropriately select and scale earthquake ground motions for the establishment of these sets [1]. There are a number of studies conducted in search for how to establish the record sets properly for seismic performance evaluation of structures [1-8] next to many other valuable work.

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Searching with this purpose by excluding the sensitivity of the structural response to the variation of the ground motion parameters is impossible. Thus, some of the studies have their emphasis on structural response whereas some of them deal with the earthquake characteristics more deeply.

Despite the above considerations, nonlinear dynamic analyses are carried out generally according to the limited rules provided by the codes in practice. Thus, according to the authors the outcomes of the rules provided by the codes should also be studied thoroughly. This study is such an attempt to evaluate the structural response of a model special moment frame. .

II. Basic Properties and Design of the Model Frame

An office building which consists of two perimeter special steel moment frames (SMF) in one direction and two perimeter special concentrically braced steel frames (SCBF) in the opposite direction in order to resist earthquake loads is considered (Fig. 1). Actually, one of the perimeter steel special moment frames is investigated as the model frame in this work. Load and Resistance Factor Design of the ANSI/AISC 360-10 is taken into consideration while designing the model frame [9]. Applied capacity based design principles for providing global ductility are based on ANSI/AISC 341-10 [10]. The loads and load combinations are based on ASCE/SEI 7-10 [11]. In the model ASTM A992 steel is used as material for both columns and beams with 445 MPa (65 ksi) and 345 MPa (50 ksi), tensile and yield stresses, respectively. Dead load for the normal stories is 3.83 kN/m^2 (80 psf) , including the weight of the steel structural elements. Live load is 3.11 kN/m^2 (65 psf) for the normal stories (the weight of the partition walls are included). Dead load and live load are considered as 3.11 KN/m^2 (65 psf) and 2.87kN/m^2 (60 psf) for the roof floor. The site class is considered as C on calculations. The mapped maximum considered earthquake spectral response acceleration values at short and 1 second periods are 1.5g and 0.6g.

The design process is mostly dominated by drift limits and strong column-weak beam considerations. Equivalent lateral load procedure of ASCE/SEI 7-10 is used to determine the distribution of lateral loads to the floor levels. The fundamental period of the model frame is found as 2.38s. All the story columns from ground level to 26 m (85 f) level are assigned as W14x193 and from that level to the top level story columns are assigned as W14x176.

The whole beams of the 1st-7th stories are W18x106 whereas the remaining are W18x71. Gravity columns are assigned as HSS 16x16x5/8 in order to be used for P-Delta modeling under the effect of "1.4D+1.6L" combination.

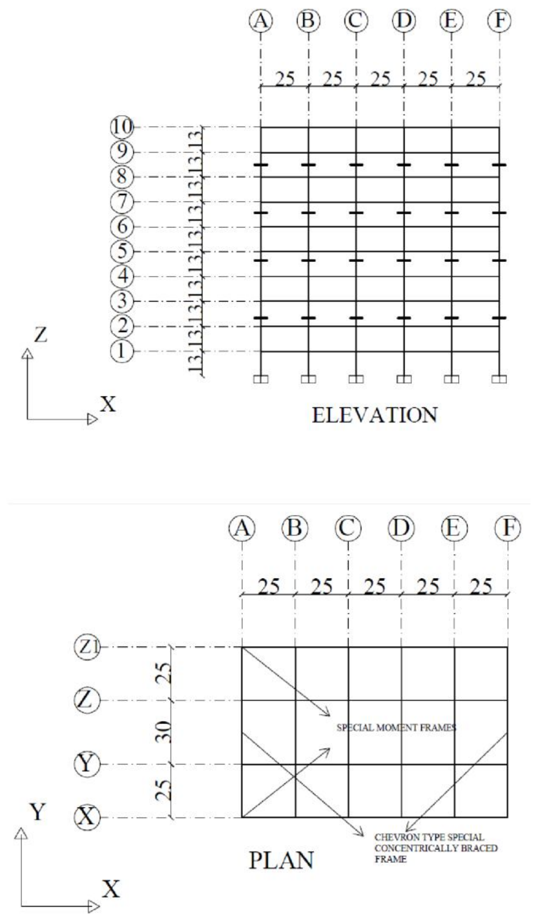

Figure 1. Basic properties of the office building (dimensions are in feet)

III. Frame Modeling for Nonlinear Analyses

In two dimensional model P-Delta effects and the gravity columns are taken into consideration but the orthogonal SCBFs' columns are neglected. All the structural elements are deformation controlled. There is an R_y factor of 1.1 for structural elements' strength calculations. It is considered as beams are not exposed to axial loads because of the rigid diaphragm considerations on the other hand columns are exposed to both axial and flexural load. Beam-to-column connections of the model frame are fully restrained. Direct modeling of the panel zones are not required per the conditions provided in ASCE/SEI:41-06 [12]. In order to determine effective spans of the beams, rigid end offsets are taken into consideration.

Beam flanges are capable of fully plastifying. Because they are restrained against lateral torsional buckling. All the cross sections of the structural elements are made of seismically compact components. Story drift ratios obtained from the analyses are not high enough to damage beam-column connections. A strain- hardening slope of 3% of the elastic slope is used for beams and columns. Because there is not a greater strain-hardening slope than justified by test data. Yield rotations of beams and columns are determined according to the (Eq.5-1) and (Eq.5-2) of ASCE/SEI:41-06[12].

Initial static analyses are carried out according to ASCE/SEI: 41-06 with the load combination of 1.1(D+0.25L) where D and L are dead and live loads. In the second step, nonlinear dynamic analyses are carried out on the deformed frame.

IV. Ground Motion Record Sets

The computer program used herein during the selection and scaling of ground motion records to approach a uniform hazard spectrum is a similar version of the one reported by PEER Strong Motion Database Center [13] however "single record" s are used instead of "record pairs". This is because it is stated in ASCE/SEI 7-10 that where two-dimensional analyses are performed, each ground motion shall consist of one horizontal acceleration history, selected from an actual recorded event. A computer program coded by Dr. Nuri Özhendekci is used during the establishment of record sets. The upper bound for the scale factors of the records is 1.75 for both sets. By the use of the program, it has been possible for the user to determine an upper bound for the scale factors in order to keep the scaled records' as close as possible to the seed records. It is important not to alter the seed records' characteristics because the main scope of this work is to study the effects of different record sets.

There are two different ground motion sets are prepared with the some hazard level of 10% possibility of exceedance in 50 years in order to study the behavior of the model frame under dynamic loads. It is expected for the whole structural elements of the model frame to satisfy Life Safety (LS) performance level. There are some distinctive characteristics which are assigned to the record sets; they are established as near fault records with velocity pulses (NF-WP) and near fault records without velocity pulses (NF-WOP). Each set has 7 scaled ground motion records compatible with the minimum number of records per ASCE/SEI 7-10. Furthermore, it is stated that the appropriate records of events should have magnitudes, fault distances and mechanisms that are consistent with the maximum considered earthquake per the same code but according to a study conducted by Iervolino and Cornell, there is little evidence to support the need for a careful site specific process of record selection by magnitude and distance [14]. Thus, a specific scenario is deliberately not adopted in this work in order to approximate the design spectrum with the best fitting MSE values and with small scale factors. Beside, the horizontal components of the whole database of the PEER Strong Motion Database Center for site class C is used during the iterations in search for the best fit [15]. The records of NF-WP have closest distances to the fault rupture between 0.5-7 km, whereas the corresponding ones are between 4.5-13.9 for the NF-WOP set and the magnitude of the whole records are between 5.5-7.62. The directivity is not considered as a classifying characteristics, so both fault normal and fault parallel components are selected randomly in order to obtain the best fitting average spectrum. The response spectra and average spectrum of each record set and design response spectrum are provided in Fig 2 and some of the basic properties of the earthquake records of these sets can be found in Tables I and II.

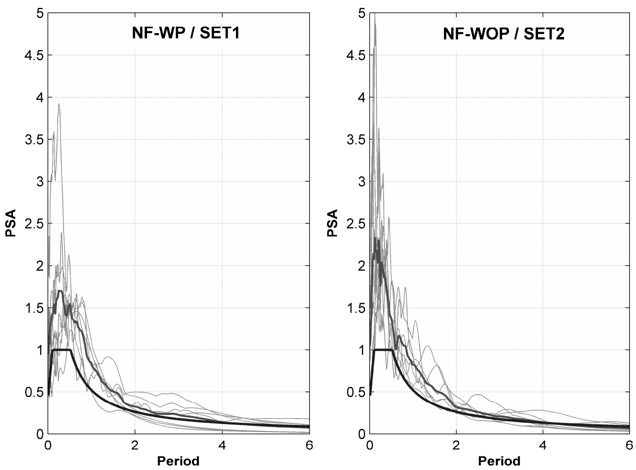

Figure 2. The response spectra, average spectrum and design response spectrum for NF-WP and NF-WOP ground motion record sets

TABLE I. SET 1 - NEAR FAULT RECORDS WITH PULSE (NF-WP)

YEAR	*EVENT*	*NGA#*	*Comp.*	*Mag.*	R_{rup} *(km)*	T_p *(s)*	*Scale*
1992	Cape Mendocino	825	FP	7.01	7	4.9	1.319
1999	Chi-Chi Taiwan	1515	FN	7.62	5.2	9.2	1.750
1989	Loma Prieta	779	FP	6.93	3.9	4.1	1.319
1994	Northridge-01	1085	FN	6.69	5,2	3.5	1.164
1985	Nahanni- Canada	496	FP	6.76	4.9	0.81	1.750
1979	Coyote Lake	150	FN	5.74	3.1	1.2	1.750
1984	Morgan Hill	451	FP	6.19	0.5	1.1	1.164

TABLE II. SET 2 - NEAR FAULT RECORDS WITHOUT PULSE (NF-WOP)

YEAR	*EVENT*	*NGA#*	*Comp.*	*Mag.*	R_{rup} *(km)*	*Scale*
1999	Chi-Chi- Taiwan	1521	FN	7.62	8.9	1.7500
1994	Northridge-01	1004	FN	6.69	8.4	1.4597
1989	Loma Prieta	802	FP	6.93	8.5	1.7500
1978	Tabas- Iran	139	FP	7.35	13.9	1.7500
1985	Nahanni- Canada	495	FN	6.76	9.6	1.7500
1976	Gazli- USSR	126	FN	6.8	5.5	1.4597
1987	Baja California	585	FN	5.5	4.5	1.7500

V. Evaluation of the Analyses Results

In Fig 3 and 4 the distribution of maximum story drift ratios along the frame height the average values of maximum drift ratios and the coefficient of variation of the drift ratios per each floor level are shown. According to this it is obvious that all of the maximum story drift ratios are smaller than 0.04 which indicates that beam to column connections are not in the range of inelastic deformation.

The peak average story drift for NF-WP is obtained of the 8th story with a value of 0.0242 also the peak value is obtained at the 4th story for the 779-FP record with a value of 0.0327. The maximum average story drift for NF-WOP is obtained at the 8th story with a value of 0.0237 and the peak value is obtained at the 4th story for the 495-FN record with a value of 0.0301. The average maximum drift ratio is observed at 8th story for the whole sets. The average drift ratios are very close for the top 3 stories. The difference between the drift ratios of 2 sets for any level is smaller than 0.005. For the bottom 3 stories the average drift ratios of NF-WP records are the lowest. For the 6 stories from the base the coefficient of variation ratios are the considerably higher that indicates a high amount of dispersion. For the 6 stories from the base NF-WOP records have considerably lower coefficient of variation values. The coefficient of variation for both sets are between 0.015 and 0.025 for top 4 stories.

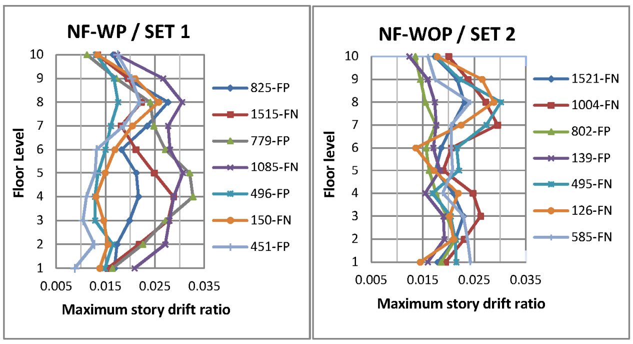

Figure 3. Maximum story drift ratios for the both record sets

In order to investigate the displacement of structural elements, the peak displacements of left end of the left beam of each story and the bottom end of the left corner column for each story are evaluated. It is found out that the tendency is very similar for beam rotation and drift ratio distributions along the frame height (Fig.4(a), Fig.5(a)). However, the dispersion is relatively high for story drift ratios when compared to beam rotations. The maximum story drift ratio reached is 0.03263 rad. at the 7th floor level for NF-WOP set, whereas the peak value is 0.02887 rad. at the 4th floor level for the NF-WOP set. The minimum story drift ratio reached is 0.007362 rad. at the 10th floor level for NF-WP set, whereas the minimum value is 0.007601 rad. at the 10th floor level for the NF-WOP set. Though the maximum story drift is the highest for NF-WP, the maximum beam rotation is the highest for NF-WOP. The column rotation is generally considerably low (below 0.003 rad.) expect for the bottom stories. For NF-WP set nearly all of the records caused high rotations at the bottom ends, however only one record caused high column end rotation for the NF-WOP set (Fig.4(b), Fig.5(b)).

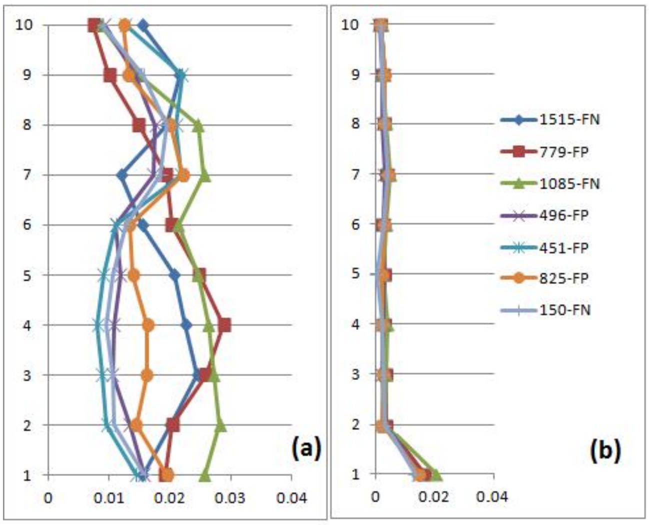

Figure 4. (a) Left end rotation for the left beam, (b) Bottom end rotation for the left corner column of NF_WP Set

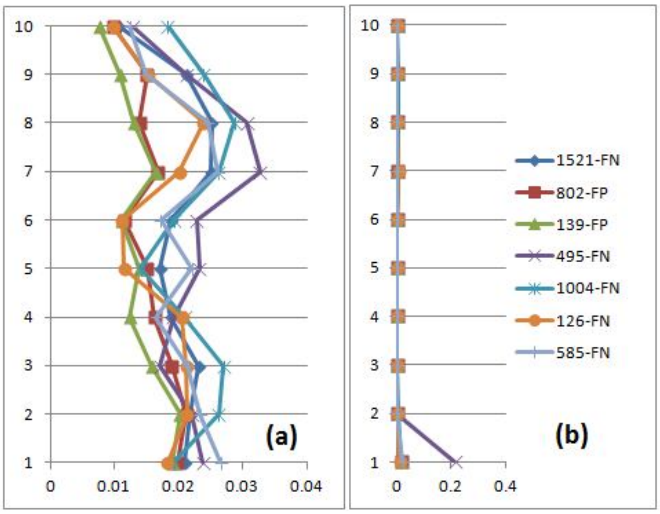

Figure 5. (a) Left end rotation for the left beam, (b) Bottom end rotation for the left corner column of NF_WOP Set

VI. Conclusions

Two record sets both consist of only the near fault ground motion records are prepared, the whole records of one of which have velocity pulses whereas the other does not. The records are selected and scaled to approach the uniform hazard spectrum defined by the seismic design code. The basic purpose was to evaluate the structural response parameters with the use of these sets and to compare the results with each other. The amplitude of the story drift ratios and structural elements' end rotations are generally close to each other for the both sets, however the dispersion of these values are a little higher for the record set with pulses. Though the difference in the dispersion between two sets is higher at lower stories this can be attributed to the stiffness and strength distribution of the model frame's elements along the elevation and the frequency content of the used records. Thus, the focus should be given to the amount of the dispersion rather than the location of it.

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