Assessment of ASCE-7 Ground Motion Scaling Method Using Computer Model of Instrumented High-Rise Building

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ABSTRACT

Performance-based earthquake engineering relies on accurate prediction of seismic demands from structural systems under given hazard conditions. In the past-decade, numerous approaches in predicting engineering demand parameters (EDPs) from structural systems were established. Some of these methods were implemented into the design guidelines and building codes. Among them, nonlinear static procedures (NSPs) have found a wide usage in practice for checking new designs and evaluating performance of existing structures. Today, there is a paradigm shift towards using nonlinear response history analysis (RHA) in lieu of NSPs to predict EDPs. The pre-requisite to nonlinear RHAs is a set of ground motions to be selected and scaled appropriately so that analyses results would be accurate (unbiased) and efficient (having relatively small dispersion). Considering that ground motions may show significant variability in frequency content and amplitude, small dispersion (variability) of EDPs is desired since it provides a confidence. There are currently many different methods in scaling and selecting records to be used in nonlinear RHAs. Since there are no experimental validation studies available up to date, the effectiveness of these methods can only be assessed using numerical simulations. These simulations require development of realistic computer models. In this respect, structural monitoring plays a key role in providing recorded motions on existing structures which can be used to create their well-calibrated (in terms of modal periods, damping etc.) computer modes. This paper shows how the recorded earthquake data from the 52-story high-rise building located in Los Angeles is utilized to create a realistic three-dimensional computer model. This model is also used to assess the accuracy and efficiency of the ASCE-7 ground motion scaling method in predicting the building's nonlinear response. It is concluded that the ASCE-7 method results in significantly high dispersion of EDPs despite the fact that median EDPs are well estimated.

INTRODUCTION

Current performance-based design and evaluation methodologies prefer intensity-based methods to scale ground motions over spectral matching techniques that modify the frequency content or phasing of the record to match its response spectrum to the target spectrum. In contrast, intensity-based scaling methods preserve the original non-stationary content and only modify its amplitude. The primary objective of intensity-based scaling methods is to provide scale factors for a small number of ground motion records so that nonlinear RHA of the structure for these scaled records is accurate, i.e., it provides an accurate estimate in the median value of the engineering demand parameters (EDPs), and is efficient, i.e., it minimizes the record-to-record variations in the EDP. Scaling ground motions to match a target value of peak ground acceleration (PGA) is the earliest approach to the problem, which produces inaccurate estimates with large dispersion in EDP values [Nau and Hall 1984; Miranda 1993; Vidic et al. 1994; Shome and Cornell 1998]. Other scalar intensity measures (IMs) such as: effective peak acceleration, Arias intensity and effective peak velocity have also been found to be inaccurate and inefficient [Kurama and Farrow 2003]. None of the preceding IMs consider any property of the structure to be analyzed.

Including a vibration property of the structure led to improved methods to scale ground motions, e.g., scaling records to a target value of the elastic spectral acceleration, $A(T_1)$ from the code-based design spectrum or PSHA-based uniform hazard spectrum at the fundamental vibration period of the structure, T1, provides improved results for structures whose response is dominated by their first-mode [Shome et al. 1998]. However, this scaling method becomes less accurate and less efficient for structures responding significantly in their higher vibration modes or far into the inelastic range [Mehanny 1999; Alavi and Krawinkler 2000; Kurama and Farrow 2003]. To consider higher mode response, a scalar IM that combines the spectral accelerations $A(T_1)$ and $A(T_2)$ at the first two periods and vector IM comprised of $A(T_1)$ and the ratio of $A(T_1)/A(T_2)$ have been developed [Bazzurro 1998; Shome and Cornell 1999]. Although this vector IM improves accuracy, it remains inefficient for near-fault records with a dominant velocity pulse [Baker and Cornell 2006].

To recognize the lengthening of the apparent period of vibration due to yielding of the structure, a scalar IM defined as a combination of $A(T_1)$ and $A(cT_1)$ where $c > 1$, has been considered [Mehanny 1999; Cordova et al. 2000]; alternatively, scaling earthquake records to minimize the difference between its elastic response spectrum and the target spectrum has been proposed [Kennedy et al. 1984; Malhotra 2003; Alavi and Krawinkler 2004; Naeim et al. 2004; Youngs et al. 2007].

In addition to different scaling methodologies, International Building Code (IBC) [ICBO 2006] and California Building Code (CBC) [ICBO 2007] require that earthquake records be scaled according to the ASCE-7 provisions [ASCE 2005].

Since there are no experimental validation studies available up to date, the effectiveness of these methods can only be assessed using numerical simulations. These simulations require development of realistic computer models. In this respect, structural monitoring plays a key role in providing recorded motions on existing structures which can be used to create their well-calibrated (in terms of modal periods, modal shapes, modal damping etc.) computer models. This paper shows how the recorded earthquake data from the 52-story high-rise building located in Los

Angeles is utilized to create a realistic three-dimensional computer model. This model is also used to assess the accuracy and efficiency of the ASCE-7 ground motion scaling method in predicting the building's nonlinear response.

ASCE-7 GROUND MOTION SCALING METHOD

The procedures and criteria in the 2006 IBC and 2007 CBC for the selection and scaling of ground motions for use in nonlinear RHA of structures are based on the ASCE-7 provisions [ASCE 2005]. According to ASCE-7, earthquake records should be selected from events of magnitudes, fault distance and source mechanisms that comply with the maximum considered earthquake. If the required number of appropriate records is not available, appropriate simulated ground motions may be included to make up the total number required.

For two-dimensional analysis of symmetric-plan buildings, ASCE-7 requires intensity-based scaling of ground motion records using appropriate scale factors so that the average value of the 5 percent-damped response spectra for the set of scaled records is not less than the design response spectrum over the period range from 0.2T1 to 1.5T1. The design value of an engineering demand parameter (EDP)—member forces, member deformations or story drifts—is taken as the average value of the EDP over seven (or more) ground motions, or its maximum value over all ground motions, if the system is analyzed for fewer than seven ground motions.

The ASCE-7 scaling procedure does not insure a unique scaling factor for each record; obviously, various combinations of scaling factors can be defined to insure that the average spectrum of scaled records remains above the design spectrum (or amplified spectrum incase of 3-D analyses) over the specified period range. Because it is desirable to scale each record by the smallest possible factor, an algorithm is developed and used in applying the code-scaling procedure, this algorithm is available at Kalkan and Chopra (2009).

BUILDING DESCRIPTION

The subject building is one of the tallest buildings in downtown Los Angeles designed in 1988 and constructed in 1988–90, this building comprises a tower and five levels of basement as underground parking. The floor plans of the tower are not perfectly square; the tip of every corner is clipped and the middle third of each side is notched. In groups of about five stories above the 36th story, the corners of the floors are clipped further to provide a setback (Fig. 1).

The structural system of the building is composed of a braced-core, twelve columns (eight on the perimeter and four in the core), and eight 91.4 cm deep outrigger beams at each floor connecting the inner and outer columns. The core, which is about 17 m by 21 m, is concentrically braced between the level-A (the level below the ground-level) and the 50th story. Moment resisting connections are used at the intersection of beams and columns. The outrigger beams, about 12 m long, link the four core columns to the eight perimeter columns to form a ductile moment resisting frame. The outrigger beams are laterally braced to prevent lateral torsional buckling and are effectively connected to the floor diaphragm by shear studs to transmit the horizontal shear force to the frame. Perimeter columns are standard I-sections, while the core columns are built-up sections with square cross section at the lower floor and crucifix section at the upper floors. The interior core is concentrically braced. The building foundation is concrete spread footings (2.74 m to 3.35 m thick) supporting the steel columns with 13 cm thick concrete slab on grade.

Figure 1. Overview of the 52-story building in downtown, Los Angeles.

The building is instrumented with twenty sensors to record its translational and torsional motions (Fig. 2). During the Northridge earthquake (its epicenter is 30 km away), the recorded values of peak horizontal accelerations were 0.15 g at the basement, 0.17 g at the level-A, and 0.41 g at the roof; no structural damage was observed. The latest recorded event, the 2008 Chino-Hills earthquake (its epicenter is 47 km away), generated a PGA of 0.06 g at the ground and 0.26 g at the roof level.

ANALYTICAL MODEL DEVELOPMENT

Three dimensional computer model of the building was developed. Steel columns, beams, and braces were modeled by force-based nonlinear beam-column element in the open source finite element platform, OpenSees. The building weight, including non-structural elements such as partition walls and the mechanical equipment in the roof, was estimated to be 58,000 kips. All steel framing including columns are ASTM A-572 (grade 50) with a nominal yielding strength of 345 MPa (50 ksi). The material model implemented is based on stable hysteresis loops derived from a bilinear stress-strain model with 2 percent strain hardening. The columns were assumed to be fixed at the base level. The P-Δ effects were included in the global system level.

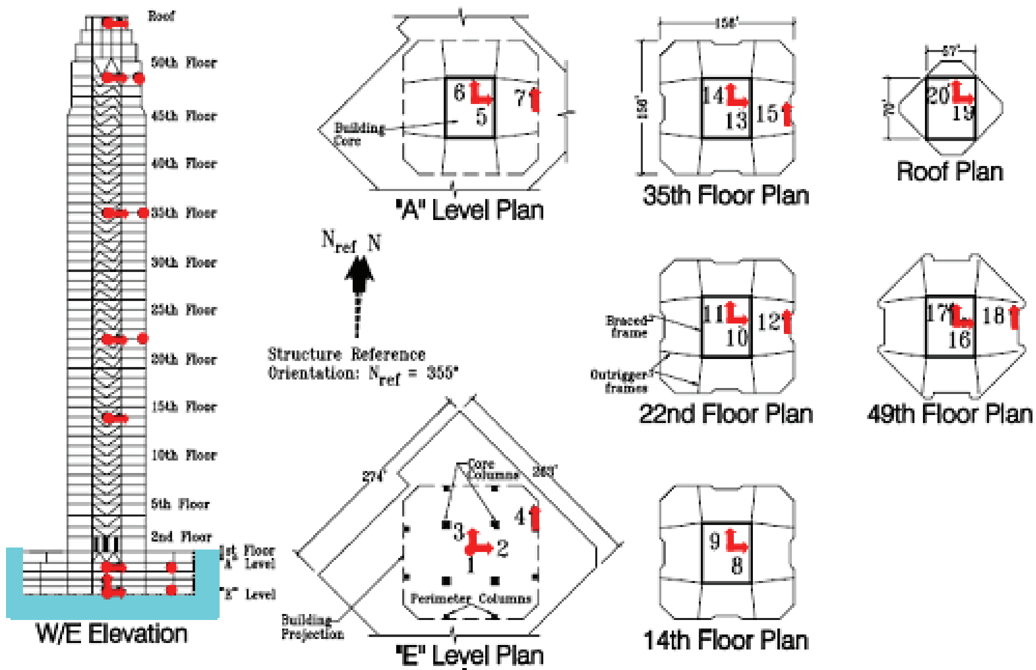

Figure 2. **Instrumentation lay-out of the 52-story building.**

As shown in Fig. 3, the first six natural vibration periods of the building are identified by frequency domain analysis of the motions at the roof relative to the first story motion recorded during the Northridge and Chino-Hills events. The computer model was able to match the measured periods (Table 1). Rayleigh damping was selected to be 4 percent of critical for the first and ninth modes. Nonlinear RHA of the building subjected to the three translational components of the motion recorded at the first story level during the Northridge event leads to the relative displacement response in two horizontal directions at the roof, eighth and second floors as shown in Fig. 4, where its compared with the motions derived from the records. The good agreement between the computed and recorded displacements indicates that the computer model is adequate for assessing the ASCE-7 ground motion scaling method.

GROUND MOTIONS SELECTED

A total of twenty one near-fault strong earthquake ground motions were compiled from the Next Generation of Attenuation project earthquake ground motion database. These motions were recorded during seismic events with moment magnitude, M $\geq$ 6.5 at closest fault distances, $R_{cl} \leq 12$ km and belonging to NEHRP site classification C and D. The selected ground motion records and their characteristic parameters are listed in Table 2.

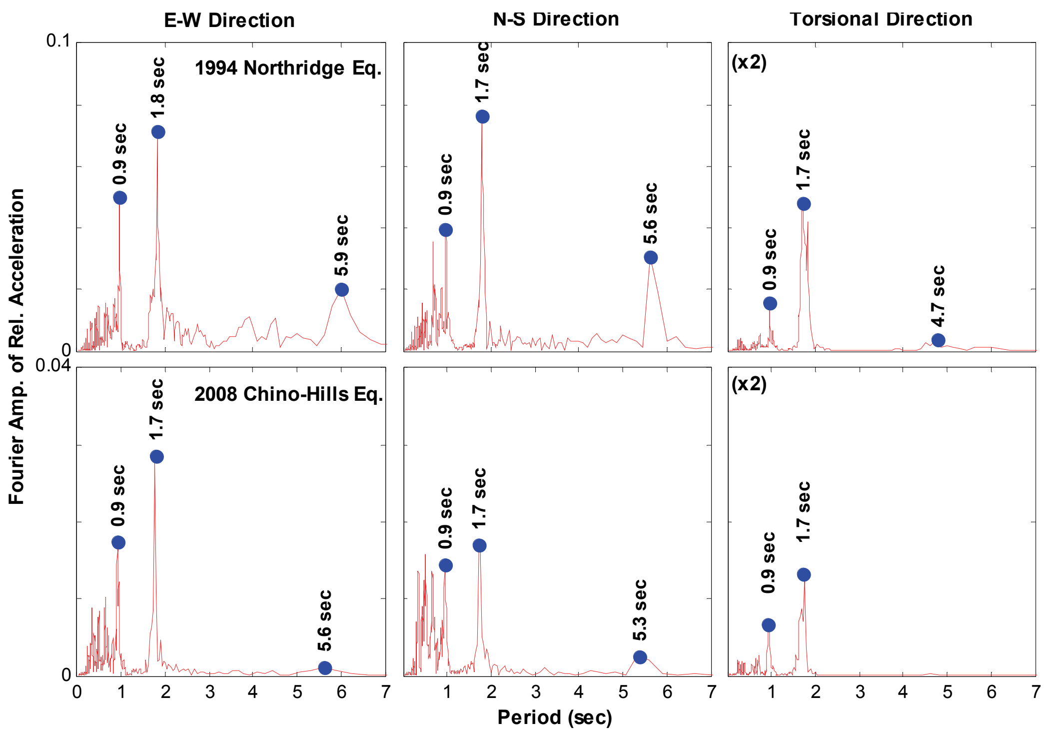

Figure 3. **Identification of natural periods for the 52-story building using recorded relative motion at the roof level (for E-W and N-S directions) and 49th floor level (for torsional direction) during the 1994 Northridge (upper-panels) and 2008 Chino-Hills (lower-panels) earthquakes.**

Table 1. Measured and computed natural periods for 52-story building

Mode No.	Direction	Period (sec)		
		1994 Northridge	2008 Chino-Hills	OpenSees
1	E-W	5.9	5.6	5.8
2	Torsional	4.7	-	5.5
3	N-S	5.6	5.3	5.4
4	E-W	1.8	1.7	1.9
5	Torsional	1.7	1.7	1.8
6	N-S	1.7	1.7	1.7
7	E-W	0.9	0.9	1.1
8	Torsional	0.9	0.9	1.0
9	N-S	0.9	0.9	0.9

EVALUATION OF ASCE-7 GROUND MOTION SCALING METHOD

A scaling procedure is considered efficient if the dispersion of EDPs due to the scaled records are small; it is accurate if the median value of the EDPs due to scaled ground motions is close to the benchmark results, defined as the median values of EDPs, determined by nonlinear RHA of the building to each of the 21 unscaled ground motions (Kalkan and Chopra 2009). An initial investigation indicated that the 21 ground motions selected are not intense enough to drive the 52-story buildings significantly into the inelastic range. Therefore, each ground motion was amplified by a factor of 2. The resulting 21 ground motions are treated as "original" motions. The median spectrum of these records is taken to be the design spectrum for purposes of evaluating the ASCE-7 ground motion scaling procedure.

The assessment is based on the comparison of the median values of EDPs determined from a set of 7 ground motions, scaled according to ASCE-7 procedure, against the benchmark results. Fig. 5 shows the benchmark EDPs; results from individual records are also included to demonstrate the large dispersion. The median value $\hat{x}$, defined as the geometric mean, and the dispersion measure, δ of n observed values of x_i are calculated from

$$\hat{x} = \exp\left[\frac{\sum_{i=1}^{n} \ln x_i}{n}\right]; \qquad \delta = \left[\frac{\sum_{i=1}^{n} (\ln x_i - \ln \hat{x})^2}{n-1}\right]^{1/2} \tag{1}$$

The EDPs selected are peak values of story drift ratio, i.e., peak relative displacement between two consecutive floors normalized by story height; floor displacements normalized by building height; column and beam plastic rotations.

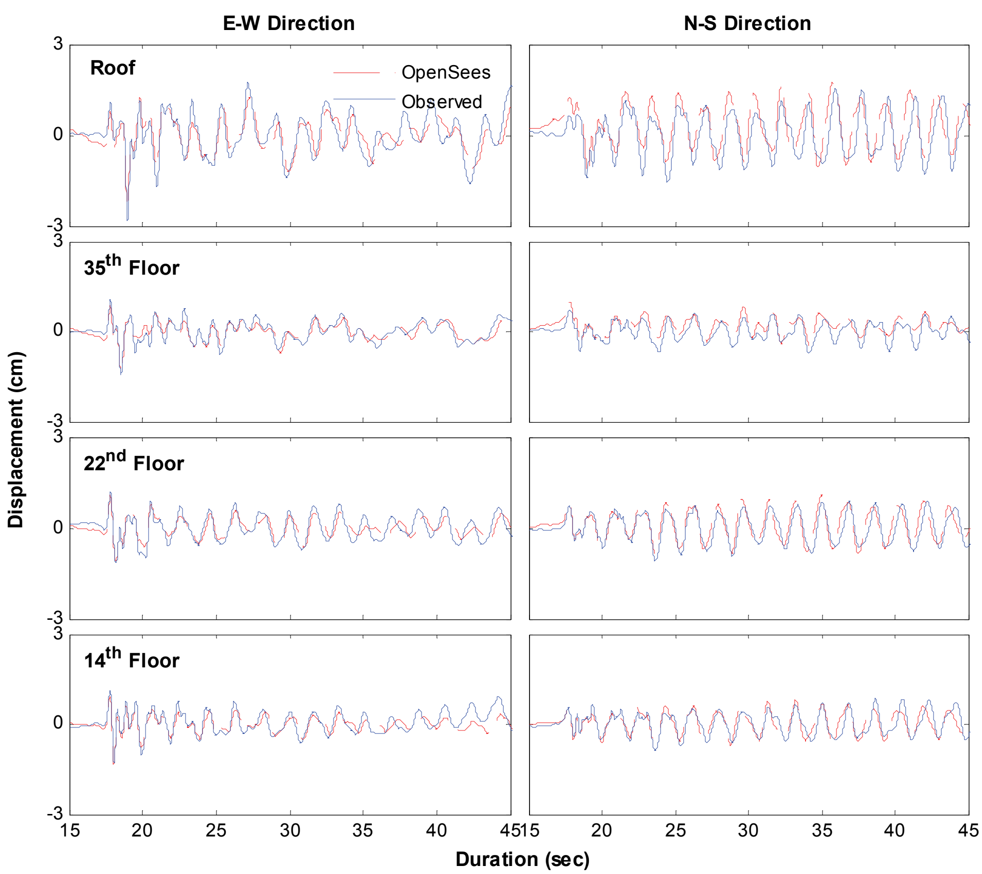

Figure 4. **Comparison of recorded and computed floor displacements in two horizontal directions of the 52-story building at different floor levels. (Recorded data is from the M5.4 2008 Chino-Hills earthquake)**

Table 2. Selected earthquake ground motions

No.	Earthquake	Year	Station	M	R_{cl} (km)	V_{S30} (m/s)	PGA (g)	PGV (cm/s)	PGD (cm)
1	Tabas, Iran	1978	Tabas	7.4	2.1	767	0.85	110.3	61.1
2	Imperial Valley	1979	EC Meloland Overpass FF	6.5	0.1	186	0.31	79.3	28.1
3	Imperial Valley	1979	El Centro Array #7	6.5	0.6	211	0.42	80.2	41.0
4	Superstition Hills	1987	Parachute Test Site	6.5	1.0	349	0.46	74.8	36.3
5	Loma Prieta	1989	LGPC	6.9	3.9	478	0.78	77.2	42.7
6	Erzincan, Turkey	1992	Erzincan	6.7	4.4	275	0.49	72.9	24.8
7	Northridge	1994	Jensen Filter Plant	6.7	5.4	373	0.75	77.8	31.9
8	Northridge	1994	Newhall - W Pico Canyon Rd	6.7	5.5	286	0.39	76.6	43.1
9	Northridge	1994	Rinaldi Receiving Sta	6.7	6.5	282	0.63	109.2	28.3
10	Northridge	1994	Sylmar - Converter Sta	6.7	5.4	251	0.75	109.4	45.8
11	Northridge	1994	Sylmar - Converter Sta East	6.7	5.2	371	0.68	87.3	31.7
12	Northridge	1994	Sylmar - Olive View Med FF	6.7	5.3	441	0.71	97.4	22.4
13	Kobe, Japan	1995	Port Island	6.9	3.3	198	0.26	62.3	29.6
14	Kobe, Japan	1995	Takatori	6.9	1.5	256	0.65	118.8	33.4
15	Kocaeli, Turkey	1999	Yarimca	7.4	4.8	297	0.31	60.5	54.7
16	Chi-Chi, Taiwan	1999	TCU052	7.6	0.7	579	0.35	131.9	183.2
17	Chi-Chi, Taiwan	1999	TCU065	7.6	0.6	306	0.68	99.5	81.8
18	Chi-Chi, Taiwan	1999	TCU068	7.6	0.3	487	0.54	206.1	336.3
19	Chi-Chi, Taiwan	1999	TCU084	7.6	11.2	553	0.79	92.7	28.8
20	Chi-Chi, Taiwan	1999	TCU102	7.6	1.5	714	0.24	93.9	65.7
21	Duzce, Turkey	1999	Duzce	7.2	6.6	276	0.42	71.0	46.3

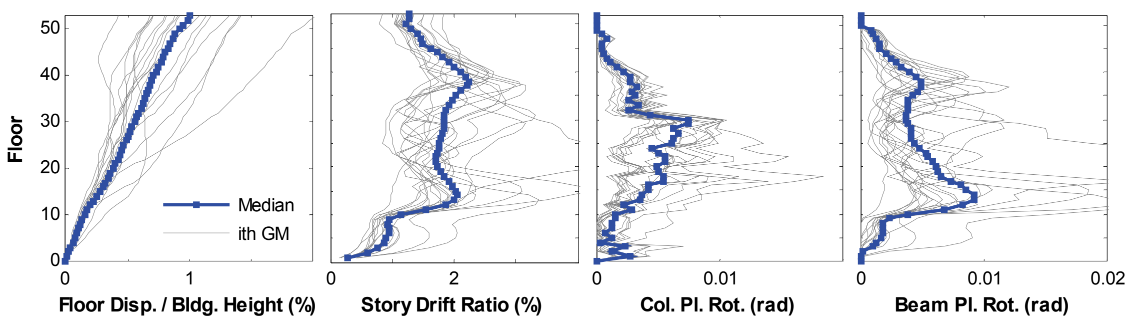

Figure 6. Median values of EDPs determined by nonlinear RHA of 52-story building for 21 ground motions; results for individual ground motions are also included.

Fig. 7 compares the EDPs determined by nonlinear RHA of the 52-story building due to the three sets of 7 ground motions. Note that each set is randomly compiled from 21 ground motions and scaled according to the ASCE-7 scaling procedure. These figures compare the median EDPs due to the three sets with the benchmark EDPs. Also included are the EDP values due to each of the scaled ground motions to show dispersion of the data. The values of EDPs due to a small (7) subset of scaled ground motions are close to the benchmark results. The median values of the peak floor displacement, story drift ratio and column plastic rotations are well-estimated (difference is less than 20 percent), whereas beam plastic rotations are slightly overestimated at intermediate floors. Whereas, the dispersion of the EDP values due to the 7 scaled records is unacceptably large for ground motion Set-1 and -

2. Studies also show that ASCE-7 scaling procedure produces exceptionally large dispersions and also over estimation of median EDPs for low and mid-rise buildings. In order to eliminate the shortcomings of the ASCE-7 scaling procedure, a new methodology for ground motion scaling based on structural dynamics theory has been recently developed and comprehensively tested (Kalkan and Chopra 2009; Sumer et al. 2009).

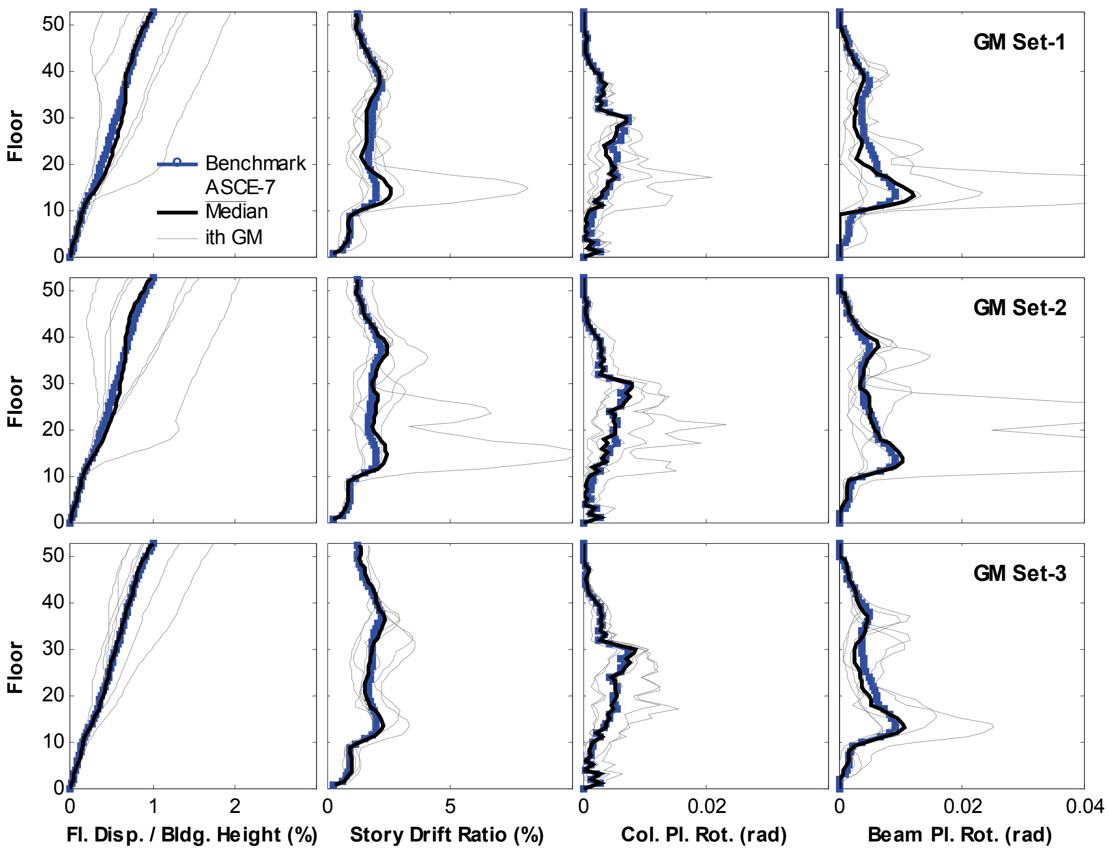

Figure 7. **Comparison of median EDPs based on the ASCE-7 ground motion scaling procedure with benchmark EDPs for the 52-story building; individual results for each of the seven scaled ground motions are also presented.**

CONCLUSIONS

The median values of engineering demand parameters (EDPs)—floor displacement, story drifts, and plastic rotations—due to three sets of 7 ground motions scaled by the ASCE-7 computed by nonlinear RHA of the 52-story building and compared against the benchmark values of EDPs, determined by nonlinear RHA due to the 21 unscaled records. The results show that dispersions due to scaled records are significantly high. The ASCE-7 scaling procedure does not insure a unique scaling factor for each record, and has a tendency to overestimate median values of EDPs, although its estimations are within 20 percent accuracy for the high-rise building studied herein. The comprehensive evaluations conducted considering different structural systems show significant overestimation of median EDPs and unacceptably large dispersions due to the ASCE-7 scaling procedure (Kalkan and Chopra 2009; Sumer et al. 2009). These findings collectively indicate that the ASCE-7 ground motion scaling procedure needs to be modified. Possible solution is to incorporate structural strength and inelastic intensity measure into scaling procedure.

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