

Evaluation of Modal Pushover–Based Scaling of One Component of Ground Motion: Tall Buildings

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Nonlinear response history analysis (RHA) is now increasingly used for performance-based seismic design of tall buildings. Required for nonlinear RHAs is a set of ground motions selected and scaled appropriately so that analysis results would be *accurate* (unbiased) and *efficient* (having relatively small dispersion). This paper evaluates accuracy and efficiency of recently developed *modal pushover–based scaling* (MPS) method to scale ground motions for tall buildings. The procedure presented explicitly considers structural strength and is based on the standard intensity measure (IM) of spectral acceleration in a form convenient for evaluating existing structures or proposed designs for new structures. Based on results presented for two actual buildings (19 and 52 stories, respectively), it is demonstrated that the MPS procedure provided a highly accurate estimate of the engineering demand parameters (EDPs), accompanied by significantly reduced record-to-record variability of the responses. In addition, the MPS procedure is shown to be superior to the scaling procedure specified in the ASCE/SEI 7-05 document. [DOI: 10.1193/1.4000091]

INTRODUCTION

For tall buildings with unusual configurations, innovative structural systems, and high performance materials, the California Building Code (CBC; ICBO 2010) and ASCE/SEI 7-05 (ASCE 2005) documents permit the use of “alternate materials and methods of construction” relative to those prescribed in their seismic requirements with the approval of the regulatory agency. For these buildings, performance-based seismic design (PBSD) concepts are being increasingly employed to ensure their safety, constructability, sustainability, and affordability. PBSD often requires nonlinear RHA to validate prescribed performance objective, which is generally *collapse prevention* for tall buildings under a very rare earthquake with a long recurrence interval, on the order of 2,475 years (Lew et al. 2008). The ground motions developed for very rare earthquakes are dominated by aleatoric (i.e., source) uncertainties because the strong ground motions recorded from large magnitude earthquakes are scarce. Thus, there is a great need to establish rational procedures for selecting and scaling records to match the target design level for a given site when there are not enough recorded motions compatible with the seismic hazard conditions.

Procedures for selecting and scaling ground motion records to be compatible with a site-specific seismic hazard are broadly described in building codes, and have been the subject of

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considerable research in recent years. Current performance-based seismic design and evaluation methodologies (e.g., ASCE/SEI 7-05) prefer intensity-based ground motion scaling where the scaling factor for a ground motion record can be chosen to minimize the difference between its elastic response spectrum and the target spectrum over a period range $0.2T_1$ to $1.5T_1$ (T_1 = fundamental vibration period of the building). Since this method does not consider explicitly the inelastic behavior of the structure, it may not be appropriate for near-fault sites where the inelastic deformation can be significantly larger than the deformation of the corresponding linear system. For such sites, scaling methods that are based on the inelastic deformation spectrum or consider the response of the first-mode inelastic single-degree-of-freedom (SDF) system become more appropriate (Bazzurro and Luco 2004, Luco and Cornell 2007, Tothong and Cornell 2008, PEER 2009).

Kalkan and Chopra (2010, 2011) utilized these concepts to develop a *modal pushover-based scaling* (MPS) procedure for selecting and scaling earthquake ground motion records in a form convenient for evaluating existing structures and proposed designs of new structures. The objective of MPS is to determine scale factors for a small number of appositely selected records so that the nonlinear RHA of a structure subjected to these scaled records is accurate (i.e., accurately estimates the median values of the engineering demand parameters, or EDPs, such as member forces, member deformations, or story drifts) and is efficient (i.e., minimizes the number of records needed to reliably obtain these EDP estimates). In contrast to the ASCE/SEI 7-05 scaling method, the MPS procedure explicitly considers structural strength obtained from the first-mode pushover curve, and determines a scaling factor for each record to match a target value of the deformation. Target deformation is defined as the median deformation of the first-mode inelastic SDF system due to a large ensemble of unscaled ground motions compatible with the site-specific seismic hazard; it may be estimated by either (1) performing nonlinear RHA of the inelastic SDF system to obtain the peak deformation due to each ground motion, and then computing the median of the resulting data set; or (2) multiplying the median peak deformation of the corresponding linear SDF system, known from the median response spectrum, by the inelastic deformation ratio, estimated from an empirical equation (e.g., Chopra and Chintanapakdee 2004).

Appropriate for first-mode dominated structures, this approach is extended for structures with significant contributions of higher modes. MPS is based on the standard intensity measure (IM) of spectral acceleration that is directly available from the USGS seismic hazard maps, where it is mapped for periods of 0.2 s and 1 s for the entire United States to facilitate construction of site-specific design spectrum, or it can be computed from the uniform hazard spectrum obtained by probabilistic or deterministic seismic hazard analysis for a given site. Thus, the MPS procedure does not require attenuation relations for any other IM.

The MPS procedure has been proven to be accurate and efficient for determining seismic demands due to one-horizontal component of ground motion for low- and medium-rise buildings, where the first-mode of vibration generally dominates their response (Kalkan and Chopra 2010, 2011). This paper investigates the accuracy and efficiency of the MPS procedure for nonlinear RHA of tall buildings where higher-mode effects generally have larger contribution to response. In addition, the accuracy and efficiency of the scaling procedure recommended in the ASCE/SEI 7-05 document is evaluated.

MODAL-PUSHOVER-BASED SCALING PROCEDURE: SUMMARY

The MPS procedure scales each record by a factor such that deformation of the first-mode inelastic SDF system—established from the first-mode pushover curve for the building—due to the scaled record matches a target value. For first-mode dominated structures, this version of the MPS procedure has been shown to be sufficient (Kalkan and Chopra, 2010, 2011). Because higher vibration modes are known to contribute significantly to the seismic response of tall buildings, the MPS procedure checks for second-mode compatibility of each record by comparing its scaled elastic spectral displacement response values at the second-mode vibration period of the structure against the target spectrum. This approach ensures that each scaled earthquake record satisfies two requirements: (1) the peak deformation of the first-mode inelastic SDF system is close enough to the target value of its inelastic deformation; and (2) the peak deformation of the second-mode elastic SDF system is not far from the target spectrum. The step-by-step description of the MPS procedure for one component of ground motion is given in Kalkan and Chopra (2010, 2011), and it is a special case of the procedure for two components of ground motion defined in a companion paper in the most recent issue of the journal (Reyes and Chopra 2012).

GROUND MOTION ENSEMBLE

A total of 21 near-fault strong earthquake ground motions were compiled from the Next Generation Attenuation project ground motion database. These motions were recorded during seismic events with moment magnitude $M \geq 6.5$ at closest fault distances $R_{rup} \leq 12$ km and belonging to NEHRP site classification C and D. The selected ground motion records and their characteristic parameters are listed in Table 1. Because the twenty-one ground motions selected were not intense enough to drive the buildings considered far into the inelastic range—an obvious requirement to test any scaling procedure—they were amplified by a factor of 3; the resulting twenty-one ground motions are treated as “unscaled” records for this investigation. Shown in Figure 1A are the 5%-damped median response spectra of the “unscaled” ground motions. The median spectrum is taken as the design spectrum (i.e., target spectrum) for purposes of evaluating the MPS and ASCE/SEI 7-05 scaling methods; also shown in this figure is the median spectrum of the ground motion ensemble as a four-way logarithmic plot, together with its idealized version (dashed-line). The idealized spectrum is divided into three period ranges: the long-period region to the right of point d, $T_n > T_d$, is called the displacement-sensitive region; the short-period region to the left of point c, $T_n < T_c$, is called the acceleration-sensitive region; and the intermediate-period region between points c and d, $T_c < T_n < T_d$, is called the velocity-sensitive region (Chopra 2007; Section 6.8). Note that the nearly constant velocity region is unusually narrow, which is typical of near-fault ground motions. In order to describe why 3× amplification needed even for strongest near-fault motions to generate enough inelastic action from the tall buildings, Figure 1C compares the maximum credible earthquake (MCE) and design spectrum for Downtown LA (as an example) on the basis of the ASCE/SEI 7-05 with the median spectrum of unscaled and 3× amplified records. Also shown in this figure are the elastic fundamental vibration periods of the structures. It is apparent that the median values of $A(T_1)$ of the original records are much lower than the design spectral acceleration. Median spectrum of 3×-amplified records barely reach to the design spectrum. This comparison clearly explains the need for upscaling original records in order to generate sufficient inelastic structural response.

Table 1. Selected near-fault ground motions

No.	Earthquake	Year	Station	M	R_{rup} (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, Calif.	1979	EC Meloland Overpass FF	6.5	0.1	186	0.31	79.3	28.1
3	Imperial Valley, Calif.	1979	El Centro Array #7	6.5	0.6	211	0.42	80.2	41.0
4	Superstition Hills, Calif.	1987	Parachute Test Site	6.5	1.0	349	0.46	74.8	36.3
5	Loma Prieta, Calif.	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, Calif.	1994	Jensen Filter Plant	6.7	5.4	373	0.75	77.8	31.9
8	Northridge, Calif.	1994	Newhall - W Pico Canyon Rd	6.7	5.5	286	0.39	76.6	43.1
9	Northridge, Calif.	1994	Rinaldi Receiving Sta	6.7	6.5	282	0.63	109.2	28.3
10	Northridge, Calif.	1994	Sylmar - Converter Sta	6.7	5.4	251	0.75	109.4	45.8
11	Northridge, Calif.	1994	Sylmar - Converter Sta East	6.7	5.2	371	0.68	87.3	31.7
12	Northridge, Calif.	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

BUILDINGS SELECTED, MODELING AND VALIDATION

The buildings selected to evaluate the efficiency and accuracy of the MPS method are existing 19- and 52-story steel special moment resisting frame buildings representative of tall building types in California. Both buildings are instrumented, and their recorded motions during past earthquakes were utilized to validate the computer models.

19-STORY BUILDING

The building shown in Figure 2a is located in Century City, Los Angeles, designed in 1966–1967 and constructed in 1967. It has 19 stories above ground and 4 stories of parking below the ground level. The vertical load carrying system consists of 11.4 cm thick

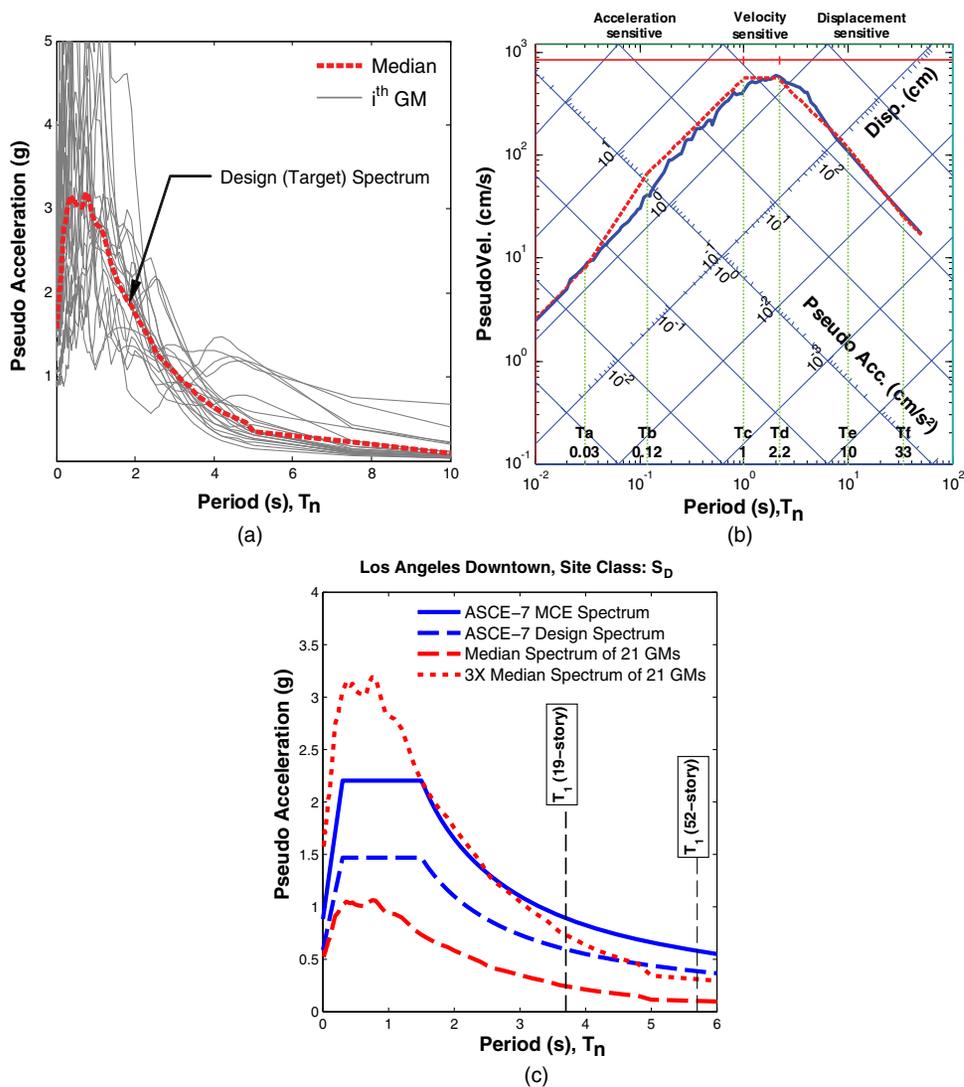


Figure 1. (a) Individual response spectra for twenty-one “unscaled” ground motions and their median response spectrum taken as the “design (target) spectrum”; (b) Median elastic response spectrum for the selected ensemble of ground motions shown by a solid line, together with its idealized version in dashed line; spectral regions are also identified; nearly constant velocity region is unusually narrow, which is typical of near-fault ground motions. (c) Comparison of ASCE-7 design and maximum credible earthquake spectra for Los Angeles downtown with median spectra of original and 3× amplified records; also marked by vertical dashed lines are the elastic fundamental period (T_1) of 19- and 52-story buildings; median spectral values of 3× amplified records at T_1 are close to the design spectral values. Damping ratio, $\zeta = 5\%$

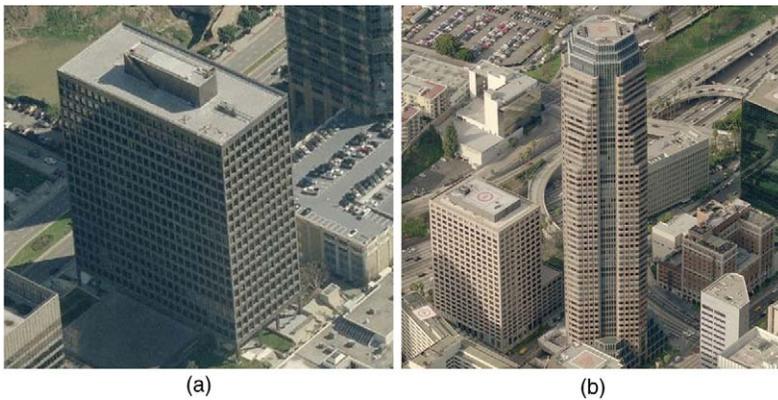


Figure 2. Overview of the (a) 19- and (b) 52-story buildings located in Los Angeles County.

reinforced concrete slabs supported on steel beams. There is no composite action between the slab and steel beams due to lack of shear studs. The lateral load resisting system consists of four steel moment frames in the longitudinal direction and five X-braced frames in the transverse direction. Moment resisting connections are used at the intersection of beams and columns. Perimeter columns are standard I-sections except at the first story where columns are built-up box-sections. The foundation system consists of 22 m long driven steel I-beam piles, capped in groups and connected by 61 cm square reinforced concrete tie beams.

The building was initially instrumented with only three sensors prior to the 1971 San Fernando earthquake, but 15 sensors were in place during the 1994 Northridge earthquake (Figure 3). During the Northridge event (its epicenter was 20 km away from the site), the recorded peak horizontal accelerations were 0.32 g at the basement, 0.53 g at the ground floor, and 0.65 g at the roof. This shaking caused moderate damage to the building in the form of buckling in some braces at upper floor levels in the transverse direction, but the perimeter moment frames were undamaged (Naeim 1997).

A 3-D computer model of the building was developed for the superstructure above the ground level, that is, first floor in Figure 3. Steel columns, beams, and braces were modeled by nonlinear beam-column element in the open source finite element platform (OpenSees 2006). To properly model the buckling response of the brace, a slight imperfection is introduced at the brace's mid-span. Centerline dimensions were used in element modeling. The building weight, including estimates of nonstructural elements such as partition walls and the mechanical equipment in the roof, was estimated to be 102,309 kN. Nodes at each floor were constrained to have the same lateral displacement to simulate rigid diaphragm behavior. The estimated floor mass and mass moment of inertia were lumped at the center of mass at each floor. Panel zone deformations and local connection fracture were not considered; therefore, the modeling of members and connections was based on the assumption of stable hysteresis loops derived from a bilinear stress-strain model with 3% strain hardening. The expected yield stress for steel members equal to 250 MPa was used. The columns were assumed to be fixed at the first floor, assumed as the base. P- Δ effects were included in the global system level.

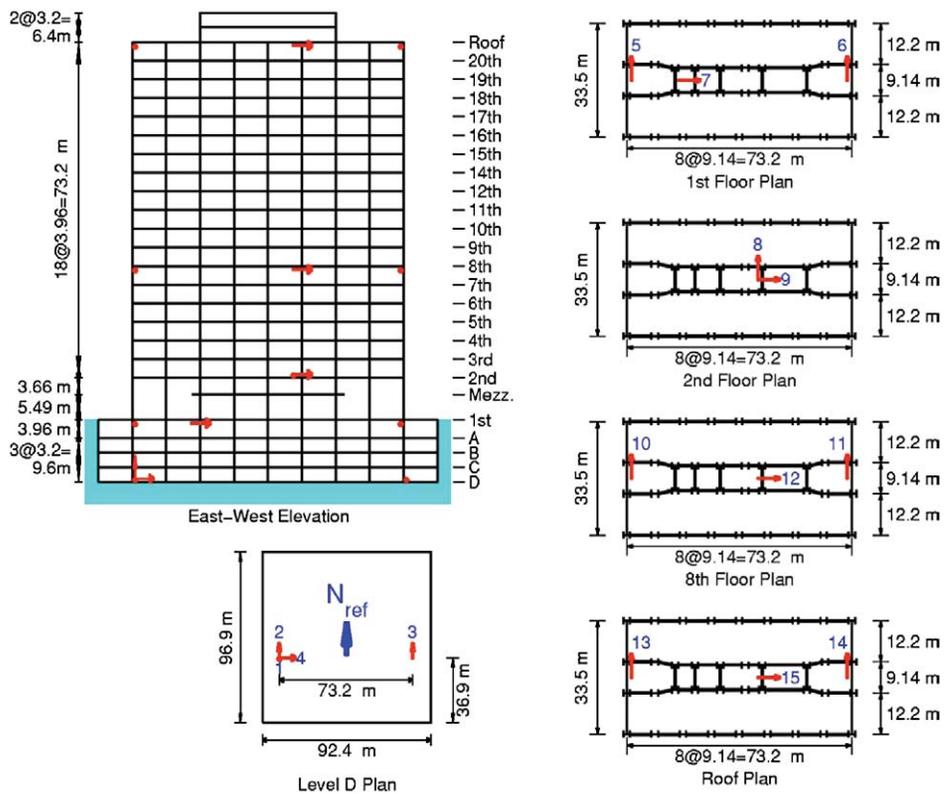


Figure 3. Instrumentation layout of the 19-story building.

The first six natural vibration periods of the building are identified by frequency domain analysis of the motion of the roof relative to base motion recorded during the Northridge event. Period computed from the computer model are close to identified values (Table 2). The two constants in Rayleigh damping were selected to provide damping ratios of 3% for the first and sixth modes. Nonlinear RHA of the building subjected to the two horizontal components of the motion recorded at the base level during the Northridge event leads to the relative displacement response in two horizontal directions at the roof, eight and second floors shown in Figure 4, where it is compared with the motions derived from records. The excellent agreement between the computed and recorded displacements indicates that the computer model is adequate.

52-STORY BUILDING

The second building selected, the eight tallest building (219 m) in downtown Los Angeles, was designed in 1988 and constructed in 1988–1990. This building comprises a 52-story steel frame office 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

Table 2. Measured and computed natural periods for 19- and 52-story buildings

	Mode	Direction	Period (s)		
			Northridge Eq.	OpenSees Model	
19-story	1	E-W	3.7	3.7	
	2	N-S + Torsion	3.4	3.4	
	3	E-W	1.4	1.4	
	4	N-S + Torsion	1.1	1.1	
	5	N-S + Torsion	0.8	0.8	
	6	E-W	0.6	0.6	
52-story	1	E-W	5.9	5.6	5.8
	2	Torsion	4.7	-	5.5
	3	N-S	5.6	5.3	5.4
	4	E-W	1.8	1.7	1.9
	5	Torsion	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	Torsion	0.9	0.9	1.0
	9	N-S	0.9	0.9	0.9

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 (Figure 2b).

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 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 consists of concrete spread footings supporting the steel columns with 13 cm thick concrete slab on grade.

The building is instrumented with twenty accelerometers to record its translational and torsional motions (Figure 5). During the Northridge earthquake (its epicenter was 30 km away from the site), 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

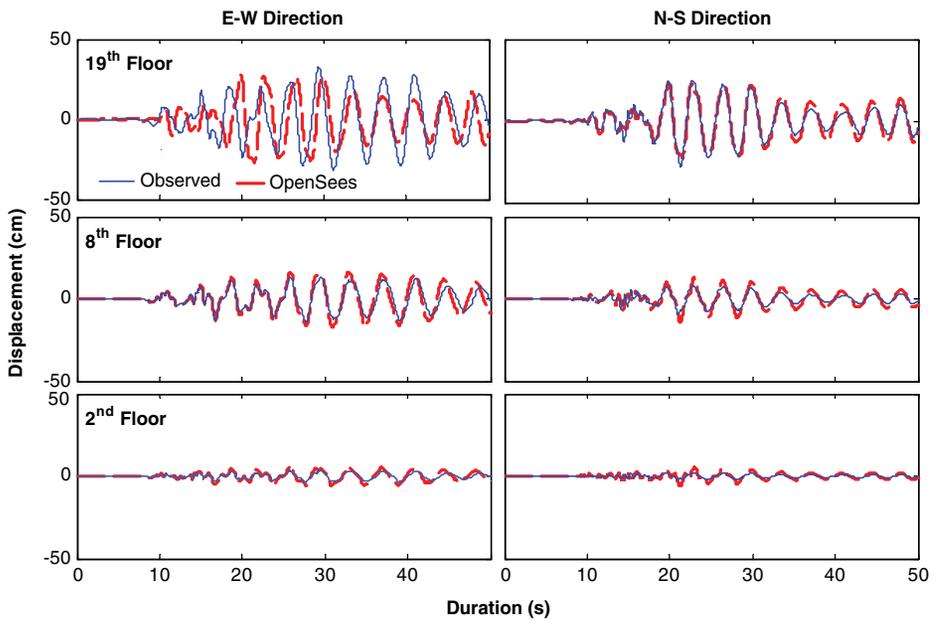


Figure 4. Comparison of observed and computed floor displacements in two horizontal directions of the 19-story building at different floor levels; recorded data is from the M6.7 1994 Northridge, California earthquake. The excellent agreement between the computed (OpenSees) and recorded displacements indicates that the computer model is adequate.

(Ventura and Ding, 2000). The latest recorded event, the 2008 Chino Hills earthquake (its epicenter is 47 km away from the site), generated a PGA of 0.06 g at the ground and 0.26 g at the roof level.

The 3-D model of the 52-story building in OpenSees included 58 separate column types and 23 different beam types. The building weight, including nonstructural elements such as partition walls and the mechanical equipment in the roof, was estimated to be 235,760 kN. The material model is based on stable hysteresis loops derived from a bilinear stress–strain model with 2% strain hardening. All steel framing including columns is ASTM A-572 (grade 50) with a nominal yielding strength of 345 MPa.

The first nine natural vibration periods of the building identified from the recorded relative roof motions with respect to base during the Northridge and Chino-Hills events. The computer model was able to match the measured periods reasonably well (Table 2). Two coefficients of Rayleigh damping were selected to provide damping ratio of 4% for the first and ninth modes. RHA of the building subjected to the three components of the motion recorded at the base level during the Northridge event leads to the relative displacement response in two horizontal directions at the roof, 35th, 22nd, and 14th floor shown in Figure 6. The excellent agreement between the computed and recorded displacements indicates that the computer model is satisfactory. Further details of computer models and validation studies are given in Kalkan and Chopra (2010).

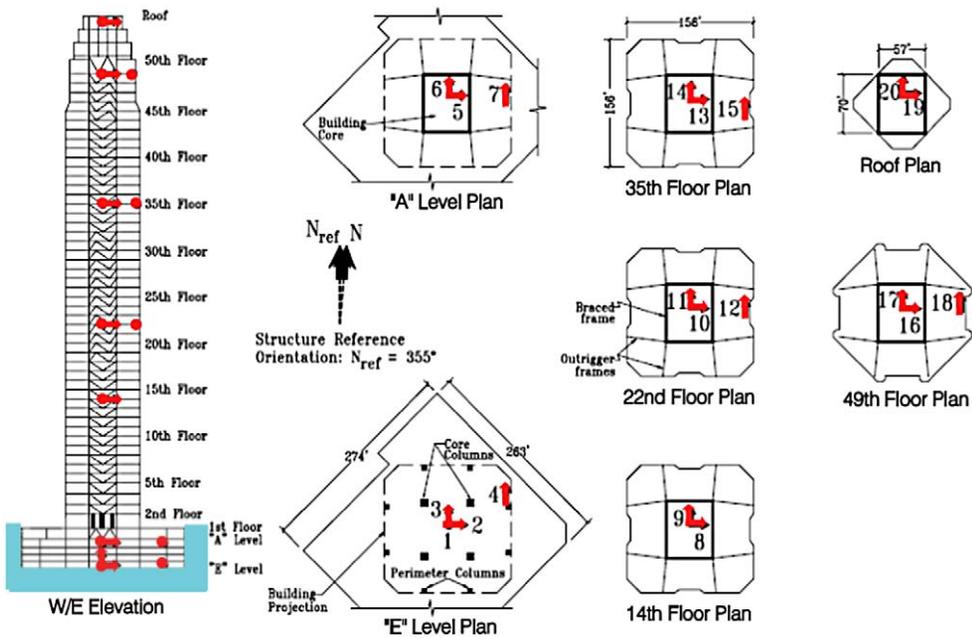


Figure 5. Instrumentation layout of the 52-story building.

FIRST-MODE SDF-SYSTEM PARAMETERS

The force-deformation relation for the first-mode SDF system for each building is determined from the base shear–roof displacement relation defined by the first-mode pushover curve, as described by the Step 5 of the MPS procedure. For both nominally symmetric buildings, only the E-W direction is considered in the pushover analysis (Figures 3 and 5). The resulting force-deformation relations for the first-mode SDF system are shown in Figure 7. For the 19-story building, a bilinear hysteretic force-deformation relation is found to be adequate, while for the 52-story building, the hysteretic force-deformation relation is idealized by the peak-oriented model (Ibarra et al. 2005) with the monotonic curve idealized as tri-linear.

EVALUATION OF MPS PROCEDURE

The accuracy of the MPS procedure was evaluated by comparing the median (for log-normally distributed variables, it is defined also as the geometric mean) value of an EDP due to a set of seven randomly selected scaled ground motions against the benchmark value, defined as the median value of the EDP due to the twenty-one “unscaled” ground motions compiled from multiple events. This large ensemble of ground motion records is assumed to provide unbiased estimates of “true” median response. A scaling procedure is considered to be efficient if the dispersion of an EDP due to the set of seven scaled ground motions is small—small dispersion in the demands from the individual records in an ensemble enhance the engineering confidence on the design outcome. Results are presented for three sets of seven ground motions that were randomly selected from the twenty-one ground motions.

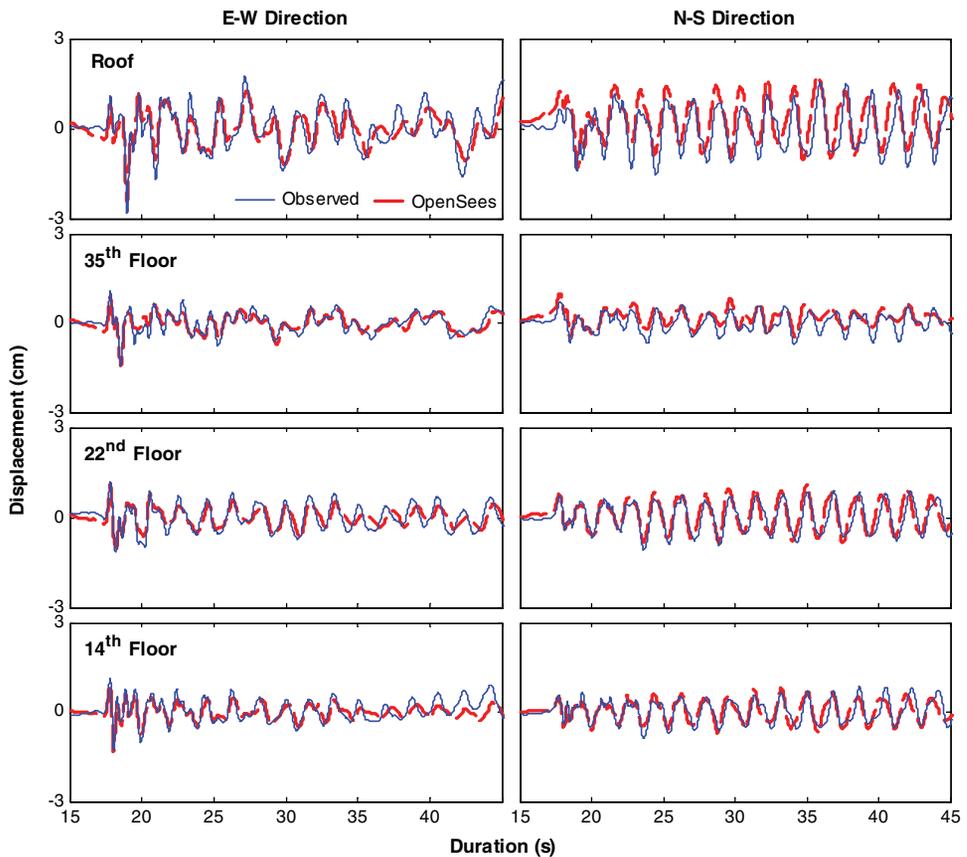


Figure 6. Comparison of recorded and computed floor displacements in two horizontal directions of the 52-story building at different floor levels; recorded data is from the M5.4 2008 Chino-Hills, California earthquake. The excellent agreement between the computed (OpenSees) and recorded displacements indicates that the computer model is satisfactory.

The records in each set were selected from at least three different earthquakes to avoid any dominant influence of a single event on the ground motion set.

BENCHMARK RESULTS

Figure 8 shows the benchmark EDPs for the two buildings: maximum values of floor displacements (normalized by building height), story drift ratios (story drift ÷ story height), and plastic rotations of the beams and columns. Results from individual records are also included to demonstrate the large dispersion or record-to-record variability. The peak values of story drift ratios range from 1.2% to 12.5% for the 19-story building, and 0.4% to 5.8% for the 52-story building. Almost all of the excitations drive both buildings well into the inelastic range as shown in Figure 9, where the roof displacement values due to the twenty-one “unscaled” ground motions are identified on the first-mode pushover curve where their corresponding FS_1/L_1

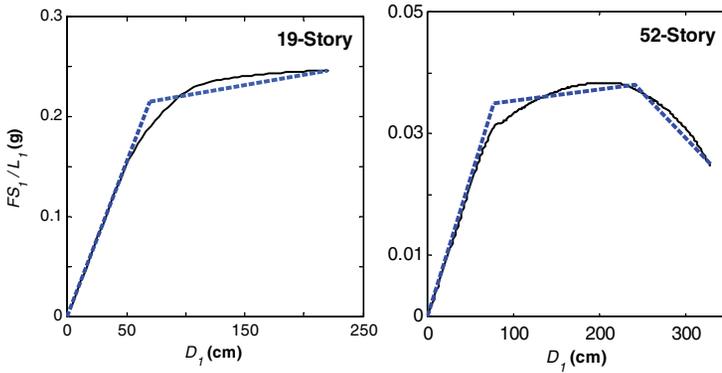


Figure 7. Comparison of first-mode pushover curve (solid line) and its idealized model (dashed line) for the 19- and 52-story buildings. For the 19-story building, a bilinear hysteretic force-deformation relation is found to be adequate, while for the 52-story building, the hysteretic force-deformation relation is idealized by the peak-oriented model with the monotonic curve idealized as tri-linear.

values read directly from the capacity curve; also shown is the median value. The median deformation exceeds the yield deformation by factors of 3.3 and 2.6, respectively for the 19- and 52-story buildings. In this plot, displacement values were computed only while the FS_1/L_1 values are their corresponding values read from the capacity curves.

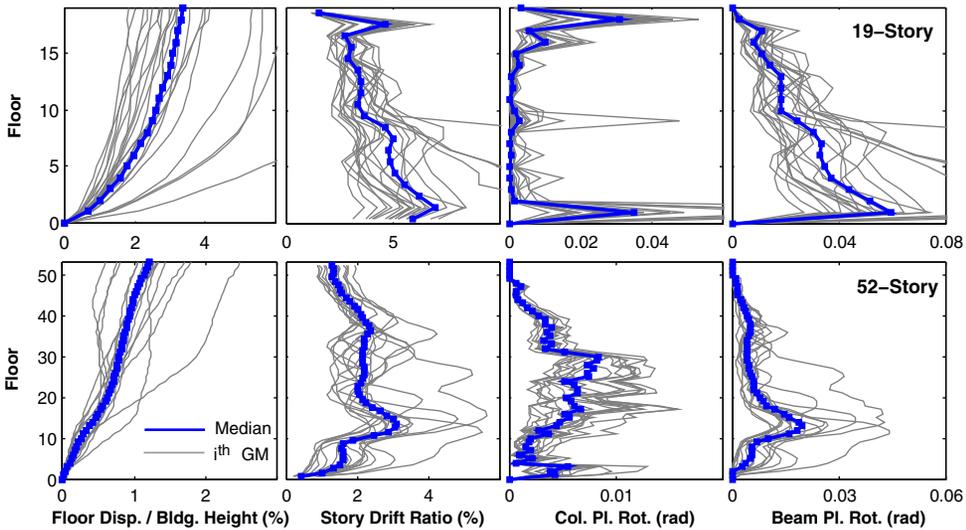


Figure 8. Median values of EDPs determined by nonlinear RHA of 19- (top row) and 52-story (bottom row) buildings for twenty-one “unscaled” ground motions; results for individual ground motions are also included to show significant record-to-record variability.

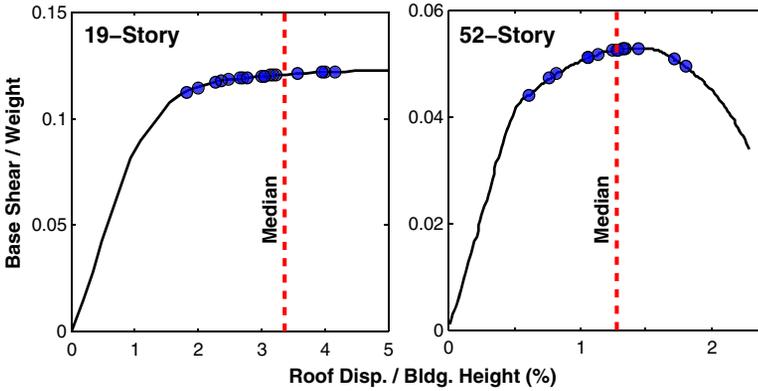


Figure 9. Roof displacements determined by nonlinear RHA of the 19- and 52-story buildings for twenty-one ground motions identified on first-mode pushover curves. The median deformation exceeds the yield deformation by factors of 3.3 and 2.6, respectively for the 19- and 52-story buildings.

TARGET VALUE OF INELASTIC DEFORMATION

In evaluations of the MPS, “exact” values of first-mode target inelastic spectral displacement (i.e., \bar{D}_1^I) were assumed to be unknown and they were estimated using an empirical C_R equation (Chopra and Chintanapakdee, 2004). This estimate is compared in Figure 10 against the “exact” value of \bar{D}_1^I (dashed horizontal line) determined by

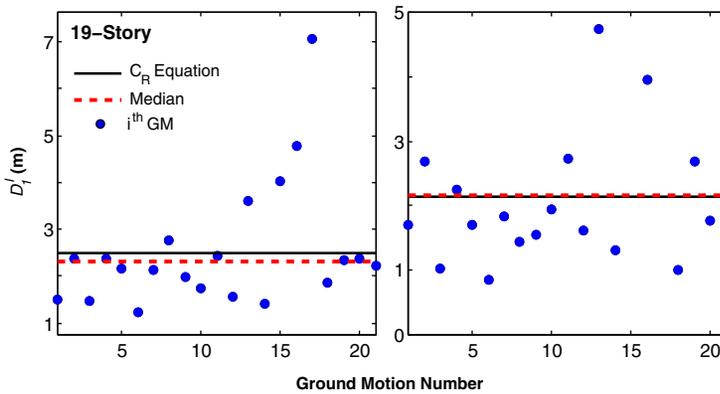


Figure 10. Peak deformation D_1^I values of the first-mode inelastic SDF system for twenty-one “unscaled” ground motions for the 19- and 52-story buildings; “exact” target value of deformation \bar{D}_1^I is identified by horizontal dashed line; horizontal continuous line indicates target value of deformation \bar{D}_1^I established by the C_R equation. The empirical equation for C_R overestimates “exact” value of \bar{D}_1^I by only 8% for the 19-story building, and under estimates “exact” value of \bar{D}_1^I by only 1% for the 52-story building.

Table 3. Scale factors (including initial 3× multiplier) for 19- and 52-story buildings and for three sets of seven ground motions according to the MPS and ASCE/SEI 7-05 methods

No.	Earthquake Name	Recording Station	GM Set	Scale Factor			
				MPS		ASCE/SEI 7-05	
				19-story	52-story	19-story	52-story
1	Superstition Hills, Calif.	Parachute Test Site	1	4.8	2.2	3.3	1.7
2	Northridge, Calif.	Jensen Filter Plant	1	4.3	5.5	2.5	1.8
3	Northridge, Calif.	Sylmar - Converter Sta East	1	3.9	2.9	3.7	2.8
4	Kobe, Japan	Takatori	1	6.6	3.6	2.5	1.4
5	Chi-Chi, Taiwan	TCU065	1	2.0	0.8	3.2	1.9
6	Chi-Chi, Taiwan	TCU102	1	3.3	1.3	4.8	2.4
7	Kocaeli, Turkey	Yarimca	1	3.5	2.3	6.4	3.5
1	Erzincan, Turkey	Erzincan	2	6.6	4.9	3.8	2.4
2	Imperial Valley, Calif.	EC Meloland Overpass FF	2	3.5	3.3	5.5	2.8
3	Kobe, Japan	Port Island	2	2.7	3.3	3.6	2.0
4	Northridge, Calif.	Sylmar - Converter Sta	2	3.0	2.0	2.5	2.0
5	Tabas, Iran	Tabas	2	2.4	0.7	3.7	3.0
6	Chi-Chi, Taiwan	TCU052	2	1.5	0.5	3.3	1.7
7	Chi-Chi, Taiwan	TCU084	2	4.5	4.1	1.4	1.2
1	Duzce, Turkey	Duzce	3	3.0	2.4	4.0	3.1
2	Imperial Valley, Calif.	El Centro Array #7	3	3.1	2.3	3.8	2.9
3	Loma Prieta, Calif.	LGPC	3	4.6	2.9	2.7	2.0
4	Northridge, Calif.	Rinaldi Receiving Sta	3	3.4	3.6	2.2	1.9
5	Northridge, Calif.	Sylmar - Olive View Med FF	3	4.4	3.2	3.3	2.1
6	Chi-Chi, Taiwan	TCU068	3	1.2	0.4	3.4	1.9
7	Northridge, Calif.	Newhall - W Pico Canyon Rd	3	3.3	2.4	3.5	2.1

nonlinear RHAs of the first-mode inelastic SDF system for twenty-one “unscaled” records; values from individual records are also included to show its record-to-record variability. The empirical equation for C_R overestimates “exact” value of \bar{D}_1^I by only 8% for the 19-story building, and under estimates “exact” value of \bar{D}_1^I by only 1% for the 52-story building.

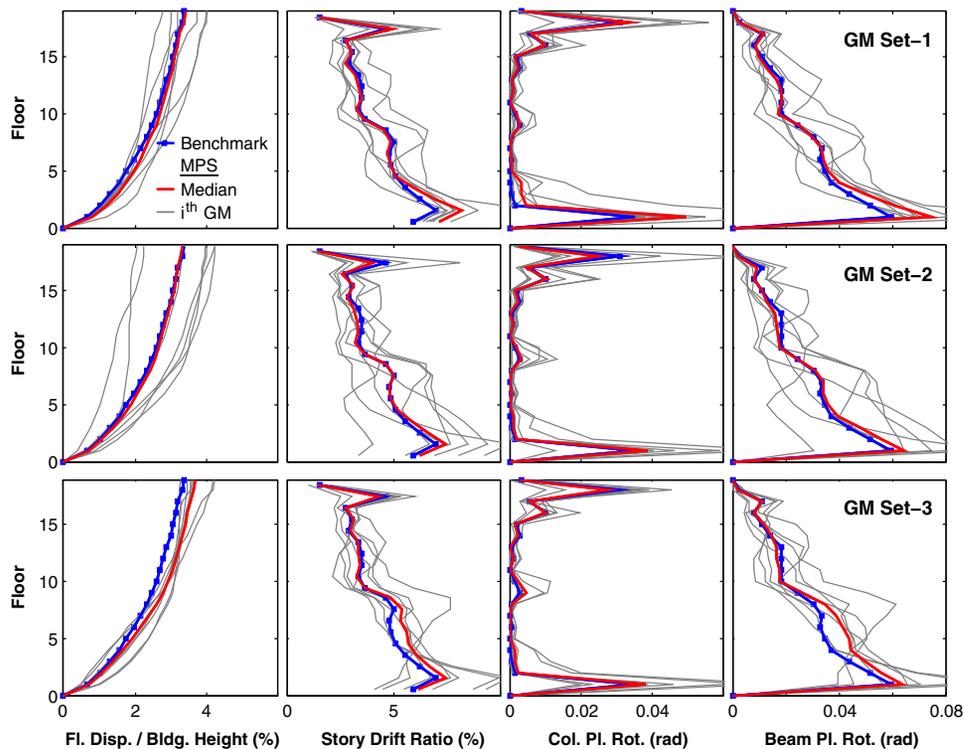


Figure 11. Comparison of median EDPs based on the MPS procedure with benchmark EDPs for the 19-story building; individual results for each of the seven scaled ground motions are also presented.

COMPARISONS AGAINST BENCHMARK RESULTS

Once \bar{D}_1^I is estimated, an appropriate scale factor for each record is determined by implementing Steps 7-8 of the MPS procedure given in Kalkan and Chopra (2010, 2011; i.e., scaling each record to have the target inelastic spectral displacement, \bar{D}_1^I , at the first-mode), and these scale factors are presented in Table 3 for each building; by visually comparing the elastic response spectra of 21 records shown in Figure 1A with the target spectrum (that is, median spectrum), it is possible to count at least 9 records with $A(T_1)$ values remaining above that of the target $A(T_1)$ for the 52-story building. For those records, it is natural to expect downscaling following the MPS procedure. Because the MPS procedure utilizes \bar{D}_1^I as a target; the inelastic target deformation value is generally below its elastic counterpart at long periods (Chopra and Chintanapakdee, 2001); it is therefore not surprising to see more records (13 out of 21) to be downscaled. The EDPs determined by nonlinear RHAs due to three sets of seven ground motions scaled according to the MPS procedure are compared against the benchmark EDPs in Figures 11 and 12, respectively for the 19- and 52-story buildings. 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

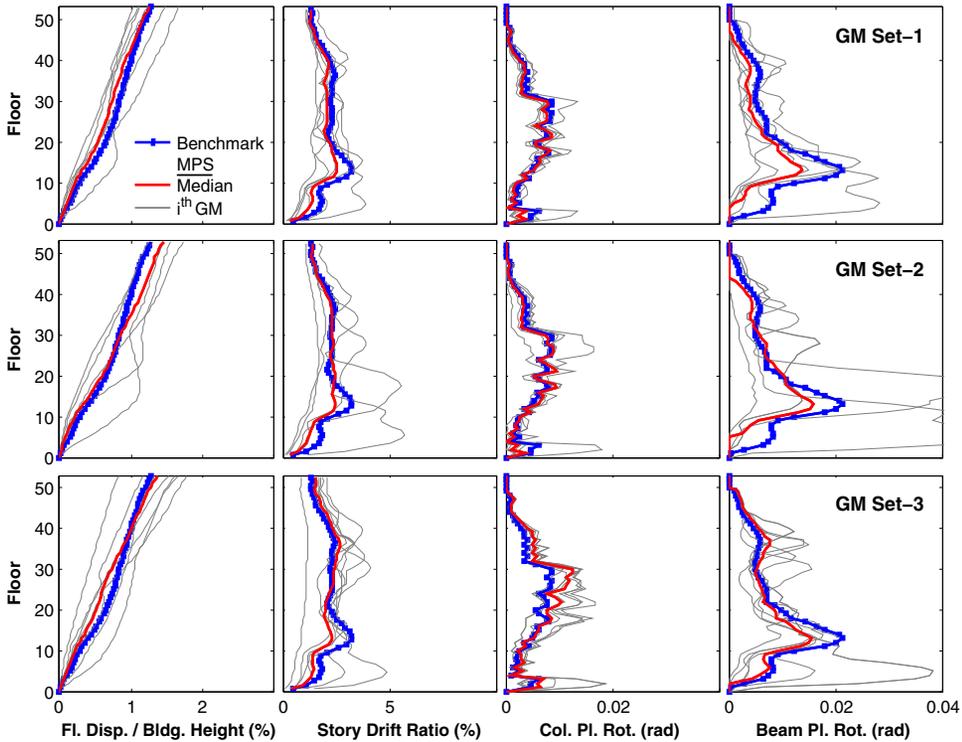


Figure 12. Comparison of median EDPs based on the MPS procedure with benchmark EDPs for the 52-story building; individual results for each of the seven scaled ground motions are also presented.

motions are close to the benchmark results. The median values of the peak floor displacements, story drift ratios and column plastic rotations are well estimated. The height-wise average errors in estimating the median values of peak floor displacements and drifts are 7% and 3%, respectively for the 19-story building, 9% and 8%, respectively for the 52-story building. These are the errors averaged over GM Sets 1–3.

The dispersion of the EDP values due to the seven scaled records is reduced as compared to the dispersion associated with the original records (Figure 8), but this reduction is less at intermediate and upper floors, suggesting that second-mode responses should also be considered in the MPS procedure.

MPS CONSIDERING THE SECOND MODE

Next, the second vibration mode is considered in selecting the most appropriate set of seven ground motions out of the twenty-one records scaled based on the first-mode response only by implementing Steps 10–13 of the MPS procedure given in Kalkan and Chopra (2010, 2011).

Table 4. Scale factors (including initial 3× multiplier) for 19- and 52-story buildings considering second mode

No.	Earthquake Name	Recording Station	Scale Factor
			19-story
1	Superstition Hills, Calif.	Parachute Test Site	4.8
2	Duzce, Turkey	Duzce	3.0
3	Erzincan, Turkey	Erzincan	6.6
4	Loma Prieta, Calif.	LGPC	4.6
5	Northridge, Calif.	Sylmar - Converter Sta	3.0
6	Chi-Chi, Taiwan	TCU102	3.3
7	Northridge, Calif.	Newhall - W Pico Canyon Rd	3.4
			52-story
1	Superstition Hills, Calif.	Parachute Test Site	3.4
2	Duzce, Turkey	Duzce	3.6
3	Imperial Valley, Calif.	El Centro Array #7	3.4
4	Imperial Valley, Calif.	EC Meloland Overpass FF	5.0
5	Northridge, Calif.	Sylmar - Converter Sta	3.1
6	Northridge, Calif.	Newhall - W Pico Canyon Rd	3.6
7	Kocaeli, Turkey	Yarimca	3.4

The seven records with the highest ranks (see Step 12) were defined as Ground Motion Set 4; this set is different for each building (see Table 4).

Figure 13 compares the median EDPs from ground motions scaled by the MPS method with the benchmark values for the two buildings. Considering the second-mode in selecting ground motions provides a more accurate estimate of the median EDPs; the height-wise average errors in estimating the median values of peak floor displacements and drift ratios are reduced to 6% and 2%, respectively for the 19-story building, 6% and 7%, respectively for the 52-story building. The height-wise average errors in beam plastic rotations are reduced from 34% (averaged over GM Sets 1–3) to 15% for GM Set 4 for the 52-story building where higher-mode contributions to response are more pronounced; however, the errors in case of the 19-story building remain essentially unchanged. For both buildings considering the second-mode in ground motion selection significantly reduces record-to-record variability (compared to the results achieved by Ground Motion Sets 1–3 as shown in Figures 11 and 12). The enhanced accuracy and efficiency are demonstrated in Figure 14, where the Δ and σ —the median ratio of the estimated story drift to its benchmark value, and dispersion of this ratio—are plotted for the four sets of ground motions. Ground Motion Set 4 is more accurate (i.e., height-wise distribution of Δ is on average closer to unity) and efficient (i.e., σ is smaller) than Ground Motion Sets 1 through 3.

COMPARING MPS AND CODE-BASED SCALING PROCEDURES

The ASCE/SEI 7-05 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

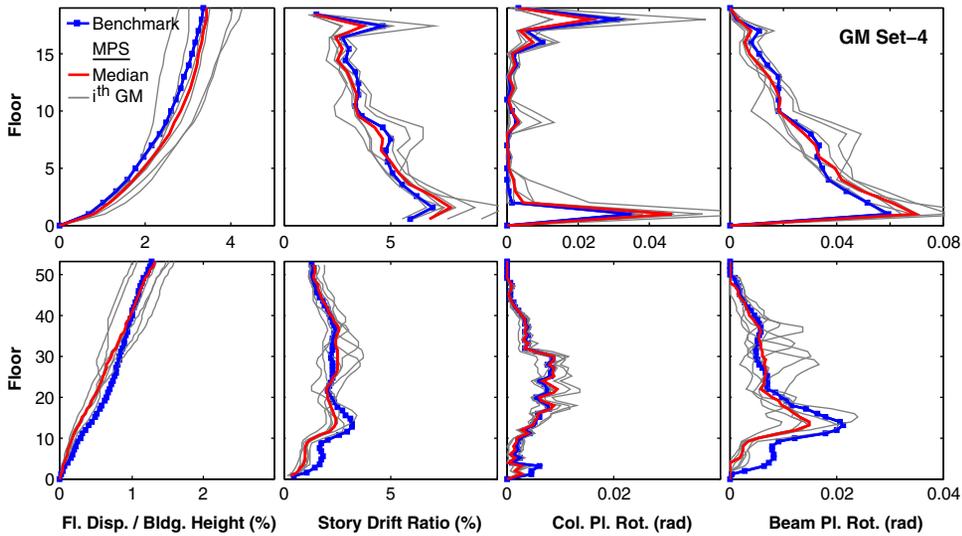


Figure 13. Comparison of median EDPs for Ground Motion Set 4 scaled according to MPS procedure with benchmark EDPs; individual results of seven scaled ground motions are also presented. Results are for the 19- (top row) and 52-story building (bottom row).

average spectrum of scaled records remains above the target spectrum over the specified period range. Because it is desirable to scale each record by the scale factor closest to 1.0, an algorithm, given in Appendix A of Kalkan and Chopra (2010), was used in applying the code-scaling procedure. The scale factors computed using this algorithm are presented in Table 3 for the 19- and 52-story buildings. The EDPs determined by nonlinear RHA of the structure due to a set of seven ground motions scaled according to the ASCE/SEI 7-05 scaling method are compared against the benchmark EDPs. Figures 15 and 16 present such comparisons for the two buildings and for the three sets of seven ground motions. The ground motions scaled according to the ASCE/SEI 7-05 scaling method overestimate the median EDPs with the height-wise average overestimation of floor displacements and story drifts by 15% and 10% respectively, for the 19-story building, 93% and 130% respectively, for the 52-story building; obviously this overestimation in the latter case is unacceptably large. Errors in beam and column plastic rotations are also considerable. Additionally, as evident by comparing Figures 11 and 12 with Figures 15 and 16, the MPS method leads to much smaller dispersion compared to the ASCE/SEI 7-05 scaling method. The large dispersion associated with the ASCE/SEI 7-05 scaling method suggests that ground motion records should not be selected randomly from a large set of records that conforms to the site-specific hazard conditions (i.e., magnitude, distance, and site geology), but their spectral shape should also be considered (Reyes and Kalkan 2012).

To demonstrate the importance of spectral shape of selected records on the accuracy and efficiency of the ASCE/SEI 7-05 scaling criteria, we deliberately selected a set of seven records (out of twenty-one) such that their scaled spectral accelerations, $A(T_1)$ and $A(T_2)$

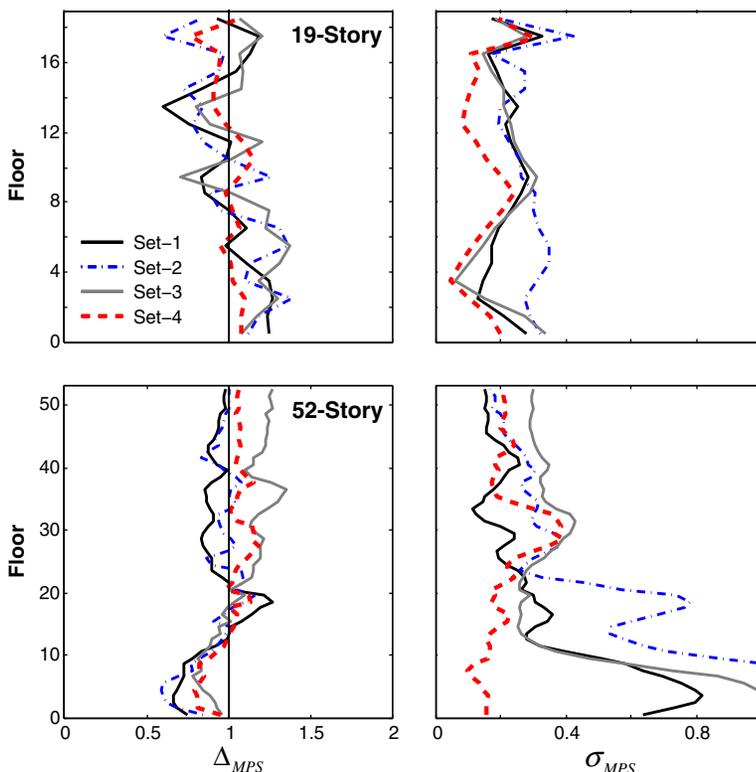


Figure 14. Median Δ_{MPS} and dispersion σ_{MPS} of story drift ratios for four ground motions sets and for 19- and 52-story buildings.

at the first two vibration periods of the building, T_1 and T_2 , are significantly higher than the design spectrum after strictly following the ASCE/SEI 7-05 scaling criteria; this final set is identified as Ground Motion Set 5. Figure 17 compares the average spectrum of scaled records in Set 5 with the design spectrum for the 19- and 52-story buildings; also shown are the individual spectra. For both buildings, the average value of $A(T_1)$ and $A(T_2)$ are significantly larger than the design spectrum values at these periods although the average spectrum is above the design spectrum between $0.2T_1$ and $1.5T_1$.

The median values of EDPs due to GM Set 5 are compared with the benchmark values in Figure 18 for the two buildings. Clearly the ASCE/SEI 7-05 scaling method grossly overestimates EDPs at almost all floors; for example, floor displacements are overestimated by as much as 100% for the 19-story building, and almost 200% for the 52-story building. The dispersion in responses due to Ground Motion Set 5 is much larger compared to Sets 1–3 when using records scaled by the ASCE/SEI 7-05 scaling method. In order to obtain acceptable estimates of EDPs that are not unreasonably conservative, the selected records must satisfy the requirement that their scaled response spectra are close to the design spectrum values at T_1 and T_2 . In contrast, the MPS method (with scaling based on “first”-mode

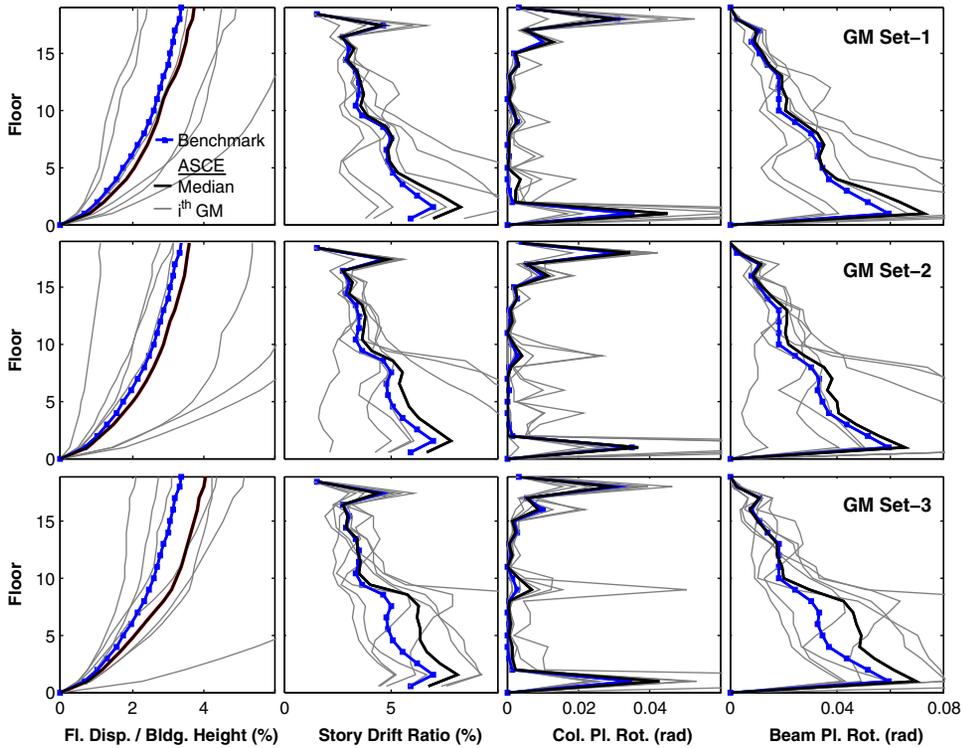


Figure 15. Comparison of median EDPs based on the ASCE/SEI 7-05 ground motion scaling procedure with benchmark EDPs for the 19-story building; individual results for each of the seven scaled ground motions are also presented.

only) is effective in scaling the records in GM Set 5 to achieve good estimates of EDPs. This is demonstrated in Figure 19 where the floor displacements and story drifts differ from the benchmark results by less than 10%; however the errors in plastic hinge rotations are larger. It is reassuring to observe that the MPS procedure is effective in scaling even records with spectral shape significantly different than the design spectrum.

CONCLUSIONS

Selected for this investigation were actual 19- and 52-story symmetric-plan buildings with their computer models calibrated against their response motions recorded during southern Californian earthquakes. This evaluation of the MPS procedure in estimating seismic demands for these two buildings has led to the following conclusions:

1. Even for the most intense near-fault ground motions, which represent a severe test, the MPS method estimates the median values of seismic demands to a good degree of accuracy (within 15% of the benchmark values) with dispersion of responses much smaller than for unscaled records in estimating seismic demands for tall buildings.

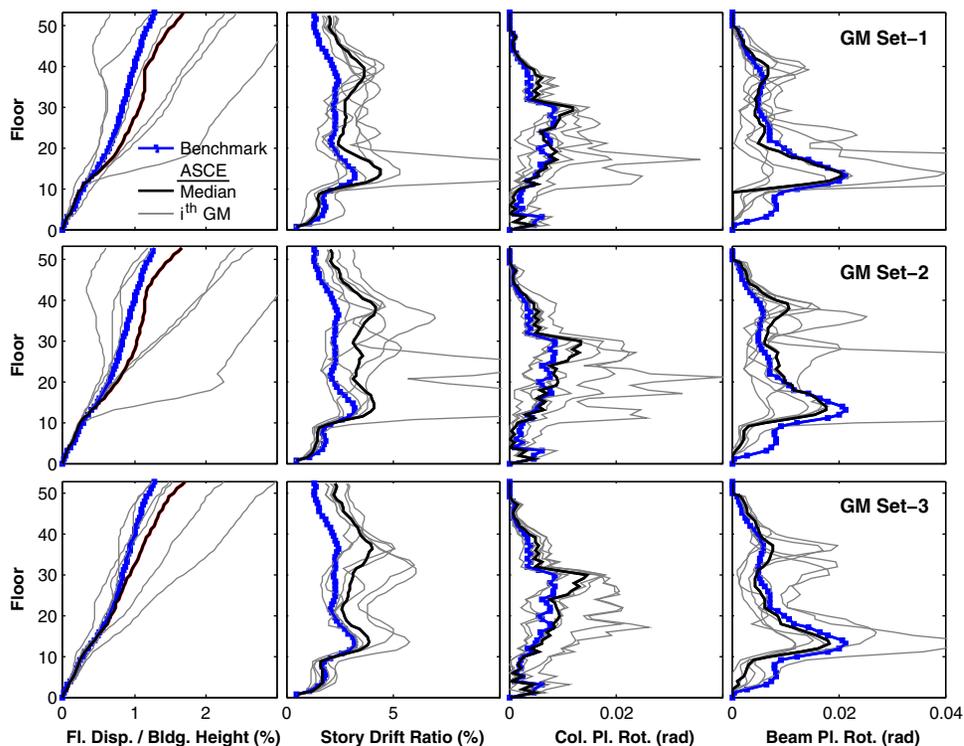


Figure 16. Comparison of median EDPs based on the ASCE/SEI 7-05 ground motion scaling procedure with benchmark EDPs for the 52-story building; individual results for each of the seven scaled ground motions are also presented.

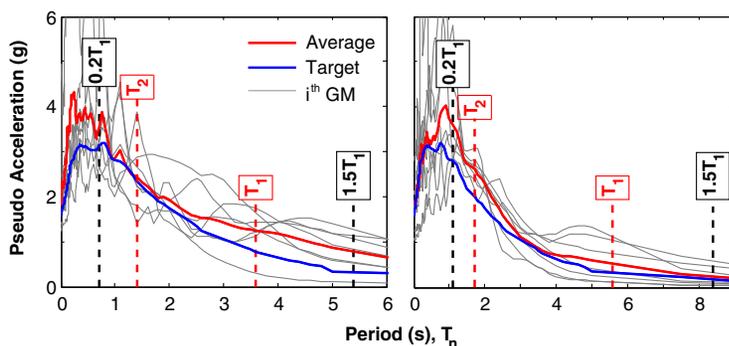


Figure 17. Comparison of design spectrum with the average spectrum of seven scaled ground motions (GM Set 5) based on the ASCE/SEI 7-05 scaling method; individual response spectra are also presented. Spectral plots are for the 19- (left), and 52-story building (right). The average values of $A(T_1)$ and $A(T_2)$ from seven scaled records are much larger than the design spectrum values at these periods.

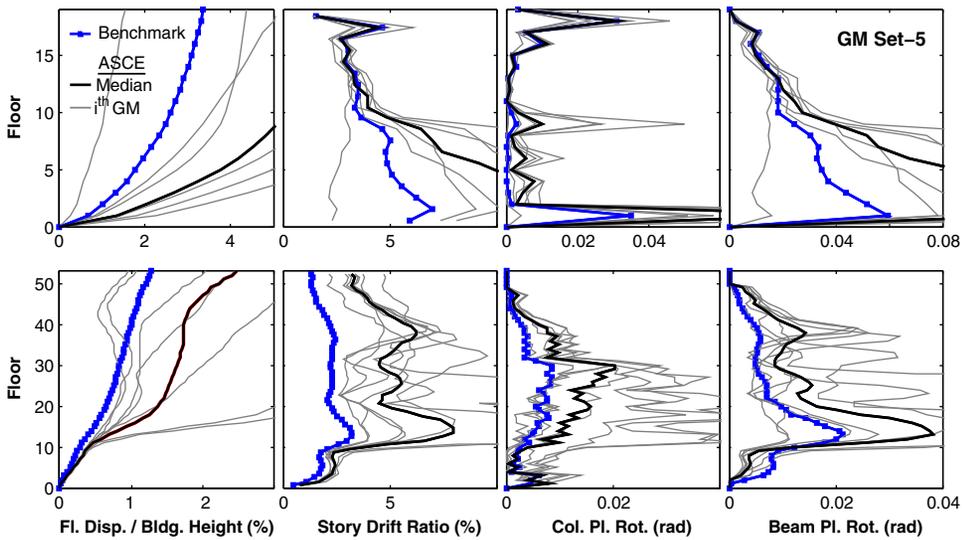


Figure 18. Comparison of median EDPs for Ground Motion Set 5 based on the ASCE/SEI 7-05 ground motion scaling procedure with benchmark EDPs for the 19-, and 52-story buildings; individual results for each of seven scaled ground motions are also presented. Results are for the 19- (top row), and 52-story building (bottom row).

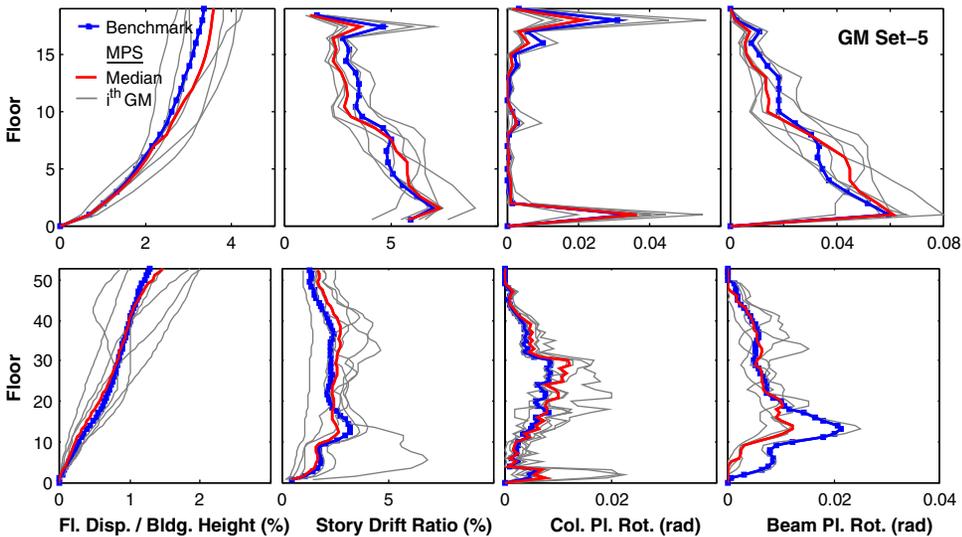


Figure 19. Comparison of EDPs for Ground Motion Set 5 scaled according to MPS procedure (considering first-mode only) with benchmark EDPs; individual results for each of seven scaled ground motions are also presented. Results are for the 19- (top row), and 52-story building (bottom row).

2. For tall buildings where higher vibration modes are known to contribute significantly to the seismic response, the MPS method requires an additional step to rank the scaled ground motions based on the closeness of the elastic deformation of second-mode elastic SDF systems to their target values. Selecting a subset of highest-ranked ground motions leads to a method that is more accurate and efficient for estimating seismic demands.
3. The MPS procedure is shown to be much superior compared to the ASCE/SEI 7-05 procedure for scaling ground motion records. This superiority is evident in two respects. First, the ground motions scaled according to the MPS procedure provide median values of EDPs that are much closer to the benchmark values than is achieved by the ASCE/SEI 7-05 procedure, which results in overly conservative predictions of EDPs. The height-wise average discrepancy of over 30% in story drift ratios (relative to the benchmark values) determined by scaling records according to the ASCE/SEI 7-05 procedure is reduced to less than 15% when records are scaled by the MPS procedure. Second, the dispersion (or record-to-record variability) in the EDPs due to seven scaled records around the median is much smaller when records are scaled by the MPS procedure compared to the ASCE/SEI 7-05 scaling method.
4. The ASCE/SEI 7-05 scaling method is seriously deficient, by resulting in structural response predictions that are grossly overestimated, if the records selected based on earthquake magnitude, distance and site geology are such that their scaled spectral accelerations at T_1 and T_2 significantly exceed the design spectrum values at these periods. However, the MPS procedure is effective in scaling even such records with spectral shape significantly different than the design spectrum.

The MPS procedure requires first-mode capacity curve computed following a pushover analysis as a priori. If the first-mode capacity curve is available, the MPS procedure can effectively be used during the preliminary design stage.

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APPENDIX: NOTATION

The following symbols are used in this paper:

- | | |
|-------|--------------------------------------------------------------------------------------------------------------------------------------|
| A | Target pseudo-spectral acceleration |
| C_R | Ratio of peak deformations of inelastic and corresponding elastic SDF systems for systems with known yield-strength reduction factor |

D_n	Peak deformation of elastic SDF system for n^{th} -mode
\bar{D}_1^I	First-mode target value of inelastic spectral displacement
M	Moment magnitude of earthquake
n	Mode sequence number
R_{rup}	Closest distance to co-seismic rupture plane
T_c	Period separating acceleration and velocity-sensitive regions of the target spectrum
T_d	Period separating velocity and displacement-sensitive regions of the target spectrum
T_n	Elastic natural vibration period
V_{S30}	Average shear-wave velocity within 30 m depth from surface
α	Ratio of post-yield and initial stiffness
ζ	Damping ratio
Δ	Geometric mean
σ	Dispersion

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