Modal Pushover-Based Scaling of Two Components of Ground Motion Records for Nonlinear RHA of Buildings

Juan C. Reyes,[a] and Anil K. Chopra,[b] M.EERI

The modal-pushover-based-scaling (MPS) procedure, currently restricted to scale one component of ground motion records, is extended herein to scale two horizontal components. The MPS procedure is presented in a general form, valid for three-dimensional analysis of multistory buildings, including those that are unsymmetric in plan. The accuracy and efficiency of the MPS procedure is evaluated here by applying it to an existing 9-story building, symmetric in plan. The computer model developed for the building is validated against motions of the building recorded during the Chino-Hills earthquake (2008); a related paper will evaluate this procedure applied to unsymmetric buildings. It is demonstrated that nonlinear response history analysis (RHA) of the building for a small set of records scaled by 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. Furthermore, the MPS procedure is shown to be much superior to the procedure specified in the ASCE/SEI 7-05 standard for scaling two components of ground motion records.

INTRODUCTION

The earthquake engineering profession has been moving away from traditional code procedures to performance-based procedures for evaluating existing buildings and proposed designs of new buildings. Although nonlinear static (or pushover) analysis continues to be used for estimating seismic demands, nonlinear response history analysis (RHA) is now being increasingly employed. In the latter approach, engineering demand parameters (EDPs)—floor displacements, story drifts, member forces, member deformations, etc.—are

[a] Assistant Professor, Department of Civil & Environmental Engineering, Universidad de los Andes, Bogota, Colombia. Formerly Graduate Student, University of California, Berkeley, CA 94720
[b] Johnson Professor, Department of Civil & Environmental Engineering, University of California, Berkeley, CA 94720

determined by nonlinear RHA of a computer model of the building for an ensemble of multi-component ground motions. Fraught with several challenging issues, selection and scaling of ground motions necessary for nonlinear RHA remains a subject of much research in recent years. Most of the procedures proposed to modify ground motion records fall into one of two categories: spectrum matching (Lilhanand and Tseng 1987) and amplitude scaling.

The objective of amplitude scaling procedures is to determine scale factors for a small number of records such that the scaled records provide an accurate estimate of median structural responses, and, at the same time, are efficient, i.e. reduce the record-to-record variability of response. In earlier approaches, ground motion records were scaled to match a target intensity measure such as peak ground acceleration (PGA), effective peak acceleration, Arias intensity, or effective peak velocity (Nau and Hall 1984, Kurama and Farrow 2003). These approaches were generally inaccurate and inefficient for structures responding in the inelastic range (Shome and Cornell 1998, Kurama and Farrow 2003). Scaling of records to match the target spectrum at the fundamental vibration period T_1 of the structure provides improved results for elastic response of structures whose response is dominated by the first mode of vibration (Shome et al. 1998). However, if the contributions of higher modes are important or the structure deforms significantly into the inelastic range, this scaling method becomes less accurate and less efficient (Mehanny 1999, Alavi and Krawinkler 2000, Kurama and Farrow 2003). Methods that consider the target spectrum ordinates at the first and second vibration periods have also been proposed (Bazzurro 1998, Shome and Cornell 1999); however, efficiency of these modified methods is compromised for near-fault records with a dominant velocity pulse (Baker and Cornell 2006).

To account for higher-mode contributions to response and the lengthening of the apparent vibration period after the structure deforms into the inelastic range, 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 (Kennedy et al. 1984, Malhotra 2003, Alavi and Krawinkler 2004, Naeim et al. 2004, Youngs et al. 2007, PEER 2009). The period ranges recommended include $0.2T_1$ to $1.5T_1$ (ASCE 2005), and T_{min} to $\sqrt{\mu}\,T_1$, where T_{min} is the period of the highest vibration mode that contributes significantly to the response, and μ is the displacement ductility demand imposed on the structure (Beyer and Bommer 2007).

Because the preceding methods do not consider explicitly the inelastic behavior of the structure, they 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 SDF system are more appropriate (Luco and Cornell 2007, Tothong and Cornell 2008, PEER 2009).

Kalkan and Chopra (2010a) used 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. This procedure explicitly considers structural strength, determined from the first-"mode" pushover curve, and determines a scaling factor for each record to match a target value of the deformation of the first-"mode" inelastic SDF system. The MPS procedure has proven to be accurate and efficient for low-, medium-, and high-rise buildings with symmetric plan subjected to one component of ground motion (Kalkan and Chopra 2010a and 2010b).

The preceding studies are all limited to scaling one component of ground motion, although at least the two horizontal components of ground motion should be considered in three-dimensional analysis of multistory buildings to compute their seismic demands. Scaling two components of ground motion has received little attention. Researchers have proposed that both components of a record are scaled by the same factor selected to match their geometric mean spectrum to the target spectrum over a period range (Malhotra 2003; Beyer and Bommer 2007). Existing methods for scaling two components of ground motion do not consider the inelastic behavior of the structure.

To overcome this limitation, this paper extends the MPS procedure for one component of ground motion (Kalkan and Chopra 2010) to two horizontal components, and investigates the accuracy and efficiency of the developed MPS procedure for nonlinear RHA of three-dimensional structural systems. In addition, the accuracy and efficiency of the scaling procedure recommended in the ASCE/SEI 7-05 standard (ASCE 2005) is evaluated. Based on the results for a 9-story symmetric-plan building with ductile frames, it is shown that the MPS procedure provides much superior results than the ASCE/SEI 7-05 scaling procedure.

MPS PROCEDURE

ONE COMPONENT

The existing MPS procedure to scale one component of ground motion, 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 (Kalkan and Chopra 2010). 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, the target deformation may be estimated by either (1) performing nonlinear RHA of the 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 elastic design spectrum, by the inelastic deformation ratio, estimated from an empirical equation. The final set of records are selected by ranking the scaled ground motions based on the difference between the peak deformation of the second-"mode" SDF system—treated as elastic—and the target deformation for that mode; the record with the smallest difference is ranked the highest. A small number of the top-ranked scaled records then define the ensemble for which nonlinear RHA of the structure is implemented. This scaling procedure has been demonstrated to be accurate and efficient (Kalkan and Chopra 2010).

TWO COMPONENTS

The MPS procedure has been extended to scale two horizontal components of ground motion by implementing the original scaling procedure for each component (x and y), independently, resulting in different scaling factors SF_x and SF_y, but considering both components together in ranking ground motions. The step-by-step procedure is presented here in a general form, valid for three-dimensional analysis of multistory buildings, including those that are unsymmetric in plan:

1. For the given site, define the target spectrum $\hat{A}(T)$, in this study taken as the median pseudo-acceleration spectrum for a large ensemble of (unscaled) earthquake records compatible with the site-specific seismic hazard.
2. Compute the natural frequencies ω_n (periods T_n) and modes ϕ_n of the first few modes of linearly elastic vibration of the building. For each ground motion component direction (x

or y), identify the first and second modes as the two modes with the largest effective modal mass.

First-Mode Dominated Structures

3. Develop the base shear-roof displacement, $V_{b1} - u_{r1}$, relation or pushover curve by non-linear static analysis of the building subjected to the first-"mode"—as defined above, which may differ with the direction of ground motion component—force distribution (Chopra and Goel 2004):

$$\mathbf{s}_n^* = \begin{bmatrix} \mathbf{m}\phi_{xn} \\ \mathbf{m}\phi_{yn} \\ \mathbf{I}_O\phi_{\theta n} \end{bmatrix}, \tag{1}$$

where $\mathbf{m}$ is a diagonal matrix of order N with $m_{jj} = m_j$, the mass lumped at the jth floor level; $\mathbf{I}_O$ is a diagonal matrix of order N with $I_{O\,jj} = I_{O\,j}$, the moment of inertia of the jth floor diaphragm about a vertical axis through the center of mass (C.M.); and subvectors ϕ_{xn}, ϕ_{yn}, and $\phi_{\theta n}$ of the nth mode ϕ_n represent x, y and θ components of ground motion, respectively. Gravity loads are applied before the lateral forces causing lateral displacement u_{rg} at the roof.

4. Idealize the $V_{b1} - u_{r1}$ pushover curve as a bilinear or trilinear curve, as appropriate. Starting with this initial loading curve, define the unloading and reloading branches appropriate for the structural system and material being considered.

5. Convert the idealized pushover curve to the force-deformation, $F_{s1} / L_1 - D_1$, relation for the first-"mode" inelastic SDF system using well-known relations (Chopra, 2007):

$$\frac{F_{s1}}{L_1} = \frac{V_{b1}}{M_1^*} \qquad\qquad D_1 = \frac{u_{r1} - u_{rg}}{\Gamma_1 \phi_{r1}}$$

where M_1^* is the effective modal mass for the first mode,

$$\Gamma_1 = \frac{L_1}{M_1} = \frac{\phi_1^T \mathbf{M}\boldsymbol{\iota}}{\phi_1^T \mathbf{M}\phi_1} \qquad \mathbf{M} = \begin{bmatrix} \mathbf{m} & \mathbf{0} & \mathbf{0} \\ \mathbf{0} & \mathbf{m} & \mathbf{0} \\ \mathbf{0} & \mathbf{0} & \mathbf{I}_O \end{bmatrix} \qquad \boldsymbol{\iota}_x = \begin{bmatrix} \mathbf{1} \\ \mathbf{0} \\ \mathbf{0} \end{bmatrix}, \quad \boldsymbol{\iota}_y = \begin{bmatrix} \mathbf{0} \\ \mathbf{1} \\ \mathbf{0} \end{bmatrix}$$

1 and **0** are vectors of dimension N with all elements equal to one and zero, respectively; and ϕ_{r1} is the value of ϕ_1 at the roof.

6. Establish the target value of deformation $\hat{D}_1$ for the first-mode inelastic SDF system. For a system with known T_1, damping ratio ζ_1, and force-deformation curve (Step 5), determine the peak deformation D_1 due to each of the unscaled ground motions $\ddot{u}_g(t)$ by solving

$$\ddot{D}_1(t) + 2\zeta_1\omega_1 \dot{D}_1(t) + \frac{F_{s1}}{L_1} = -\ddot{u}_g(t), \tag{2}$$

and compute $\hat{D}_1$, the median of the D_1 values.

7. Determine the scaling factor SF for each ground motion by solving the nonlinear equation:

$$D_1^S - \hat{D}_1 = 0, \tag{3}$$

where D_1^S is the peak deformation due to the scaled ground motion determined by solving Eq. (2) with the right side equal to $-(SF)\,\ddot{u}_g(t)$. Because the system is nonlinear, SF cannot be determined *a priori*, but requires iteration or a numerical algorithm to solve Eq. (3). If Eq. (3) is satisfied by more than one value of SF, the SF closest to one is chosen.

8. Repeat Step 7 for more records than required for the final nonlinear RHA of the building; obviously the scaling factor SF will be different for each record.

By performing Steps 1 to 8, implemented separately for the x and y components of a ground motion record, scaling factors SF_x and SF_y have been determined. Note that the target spectrum (Step 1), the pushover curve (Step 3), and the target deformation (Step 6) will all be different for the two components.

Higher Mode Considerations

9. Establish target values of deformation of the second-mode SDF system, treated as a linear system, directly from the target (pseudo-acceleration) spectrum $\hat{A}(T_n)$: $\hat{D}_n = (2\pi / T_n)^2 \hat{A}_n$; where $\hat{A}_n = \hat{A}(T_n)$ and the mode number n = 2. Note that the definition of "second mode" may differ with the ground motion component.

10. By linear RHA, calculate the peak deformation D_2 of the second-mode elastic SDF system due to $(SF)\,\ddot{u}_g(t)$, where SF is the scale factor determined in Step 7.

11. Compute the difference between the peak deformation D_2 determined in Step 10 and the target value $\hat{D}_2$ determined in Step 9 for each component of ground motion (denoted by subscript *x* or *y*), and define the normalized error

$$E_2 = \frac{\left|D_{2x} - \hat{D}_{2x}\right| + \left|D_{2y} - \hat{D}_{2y}\right|}{\hat{D}_{2x} + \hat{D}_{2y}}, \tag{4}$$

and rank the scaled records based on their E_2 value; the record with the lowest value of E_2 is ranked the highest.

12. From the ranked list, select the final set of records with their scale factors determined in Step 7 for which nonlinear RHA of the structure will be conducted.

The preceding procedure differs from the original MPS procedure for one component of ground motion (Kalkan and Chopra 2010) in three aspects. (1) Instead of estimating the deformation of the first-mode inelastic SDF system by multiplying the deformation of the corresponding linear system by the inelastic deformation ratio, this target deformation is computed as the median of the peak deformation values of the inelastic system due to an ensemble of unscaled records determined by nonlinear RHA. (2) Steps 1 to 8 are implemented to scale both components of ground motion independently. (3) Higher mode effects in response due to both components of ground motion are considered in ranking ground motions (Steps 10 to 12).

Recognizing that seismologists may prefer to scale all components of a ground motion record by the same factor to preserve focal mechanism and wave propagation effects, the preceding scaling procedure may be modified to satisfy this constraint. Equation (3) is generalized to include both horizontal components of the record:

$$\left(D_{1x}^{s} - \hat{D}_{1x}\right) + \left(D_{1y}^{s} - \hat{D}_{1y}\right) = 0 \tag{5}$$

and used in Steps 7 and 8 to determine a single scaling factor for each record.

ASCE/SEI 7-05 PROCEDURE

The ASCE/SEI 7-05 standard (abbreviated to ASCE7) requires that both components of an earthquake record be scaled by the same factor, determined to ensure that the average of the SRSS response spectra over all records does not fall below 1.3 times the target spectrum by more than 10% over the period range $0.2T_1$ to $1.5T_1$. The SRSS spectrum is defined as the square-root-of-sum-of-squares (SRSS) of the 5%-damped response spectra for the two horizontal components of ground motion. The design value of an EDP—member forces, member deformations, story drifts, etc—is taken as the average value of the EDP over a set of seven scaled records, or the maximum value over three scaled records. Various combinations of scaling factors for individual records can satisfy the preceding requirement for the average SRSS response spectrum. To achieve the desirable goal of scaling each record by the smallest possible factor, the ASCE7 procedure was implemented as described in Appendix A.

GROUND MOTION RECORDS

The twenty-eight records selected for this investigation (listed in Table 1) were recorded from earthquakes with moment magnitude, $M_w \geq 6.5$ at distances ranging from 7 to 28 km. Because the twenty-eight ground motions selected were not intense enough to drive the building considered far into the inelastic range—an obvious requirement to test any scaling procedure—they were amplified by a factor of 3.0; the resulting 28 ground motions are treated as "unscaled" records for this investigation. Shown in Figure 1 are the 5%-damped median response spectra for *x* and *y* components of the "unscaled" ground motions. The median spectrum is taken as the design spectrum for purposes of evaluating the MPS and other scaling procedures.

Common practice determines the design value of an EDP as its median value over a set of seven ground motions. Thus, to evaluate the MPS scaling procedure, two sets of seven ground motions were selected. To facilitate this selection, the peak deformation of the first-"mode" inelastic SDF system due to the twenty-eight "unscaled" ground motions were determined, and sorted in ascending order. The seven ground motions that led to the smallest deformations in *y* direction were grouped as set 1, whereas the seven ground motions that drove the SDF system to its largest deformations in *y* direction were defined as set 2; obviously, the choice of these sets represents an extreme test of the scaling procedure.

Table 1. List of 28 ground motion records

Record	Earthquake Name	M_w	Mechanism	Station Name	V_{s30} (m/s)	NEHRP Based on Vs30	R_{JB} (km)	Component *a*			Component *b*		
								Comp 1 (deg)	PGA (cm/s^2)	PGV (cm/s)	Comp 2 (deg)	PGA (cm/s^2)	PGV (cm/s)
1	1994 Northridge	6.7	Reverse	Canyon Country - W Lost Cany	308.6	D	11.4	0	402.0	43.0	270	472.6	45.1
2	1994 Northridge	6.7	Reverse	LA - Saturn St	308.7	D	21.2	20	465.2	34.5	110	430.0	39.0
3	1994 Northridge	6.7	Reverse	Santa Monica City Hall	336.2	D	17.3	90	865.9	41.7	360	362.6	25.1
4	1999 Duzce, Turkey	7.1	Strike Slip	Bolu	326.0	D	12.0	0	713.4	56.5	90	806.3	62.1
5	1999 Hector Mine	7.1	Strike Slip	Hector	684.9	C	10.4	0	260.4	28.6	90	330.2	41.8
6	1979 Imperial Valley	6.5	Strike Slip	Delta	274.5	D	22.0	262	233.1	26.0	352	344.2	33.0
7	1979 Imperial Valley	6.5	Strike Slip	El Centro Array #11	196.3	D	12.5	140	356.9	34.4	230	372.2	42.1
8	1979 Imperial Valley	6.5	Strike Slip	Calexico Fire Station	231.2	D	10.5	225	269.5	21.2	315	198.0	16.0
9	1979 Imperial Valley	6.5	Strike Slip	SAHOP Casa Flores	338.6	D	9.6	0	281.8	19.4	270	496.1	31.0
10	1995 Kobe, Japan	6.9	Strike Slip	Nishi-Akashi	609.0	C	7.1	0	499.4	37.3	90	492.9	36.7
11	1995 Kobe, Japan	6.9	Strike Slip	Shin-Osaka	256.0	D	19.1	0	238.5	37.8	90	207.8	27.9
12	1995 Kobe, Japan	6.9	Strike Slip	Kakogawa	312.0	D	22.5	0	246.5	18.7	90	338.0	27.7
13	1999 Kocaeli, Turkey	7.5	Strike Slip	Duzce	276.0	D	13.6	180	306.0	58.8	270	350.9	46.4
14	1992 Landers	7.3	Strike Slip	Yermo Fire Station	353.6	D	23.6	270	240.0	51.5	360	148.6	29.7
15	1992 Landers	7.3	Strike Slip	Coolwater	271.4	D	19.7	0	277.3	25.6	90	408.7	42.3
16	1992 Landers	7.3	Strike Slip	Joshua Tree	379.3	C	11.0	0	268.4	27.5	90	278.5	43.2
17	1989 Loma Prieta	6.9	Reverse Oblique	Capitola	288.6	D	8.7	0	518.2	35.0	90	434.6	29.2
18	1989 Loma Prieta	6.9	Reverse Oblique	Gilroy Array #3	349.9	D	12.2	0	544.2	35.7	90	360.2	44.7
19	1989 Loma Prieta	6.9	Reverse Oblique	Hollister City Hall	198.8	D	27.3	90	241.7	38.5	180	210.6	45.0
20	1989 Loma Prieta	6.9	Reverse Oblique	Hollister Diff. Array	215.5	D	24.5	165	263.4	43.8	255	273.4	35.6
21	1990 Manjil, Iran	7.3	Strike Slip	Abbar	724.0	C	12.6	0	504.5	43.2	90	486.6	53.2
22	1987 Superstition Hills	6.5	Strike Slip	El Centro Imp. Co. Cent	192.1	D	18.2	0	350.9	46.3	90	253.3	40.8
23	1987 Superstition Hills	6.5	Strike Slip	Poe Road (temp)	207.5	D	11.2	270	437.6	35.7	360	294.4	32.8
24	1987 Superstition Hills	6.5	Strike Slip	Westmorland Fire Sta	193.7	D	13.0	90	168.5	23.5	180	206.7	31.0
25	1992 Cape Mendocino	7.0	Reverse	Rio Dell Overpass - FF	311.8	D	7.9	270	377.9	43.9	360	538.2	42.1
26	1999 Chi-Chi, Taiwan	7.6	Reverse Oblique	TCU070	401.3	C	19.0	160	250.2	52.1	250	165.4	62.3
27	1999 Chi-Chi, Taiwan	7.6	Reverse Oblique	CHY006	438.2	C	9.8	6	338.5	42.7	276	357.3	55.4
28	1971 San Fernando	6.6	Reverse	LA - Hollywood Stor FF	316.5	D	22.8	90	205.8	18.9	180	170.8	14.9

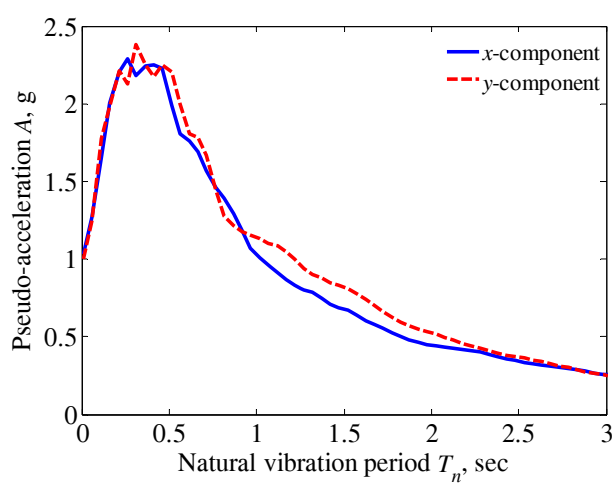

Figure 1. Median response spectra of 28 ground motions in the *x* and *y* directions; damping ratio 5%. Recorded ground motions were amplified by a factor of 3.0.

BUILDING SELECTED, COMPUTER MODEL, AND VALIDATION

The structure considered is an existing 9-story steel building, symmetric in plan, located in Aliso Viejo, CA (Figure 2); its west elevation and the plan of floors 3 to 8 are shown in Figure 3. The lateral load resisting system consists of two ductile steel moment frames in the longitudinal and transverse directions (Figure 3b) with SSDA beam slot connections; all structural members are standard I-sections and the typical floors are made-up of 3 in. metal deck with 3¼ in.-thick light weight concrete fill. The building façade consists of concrete panels and glass (Figure 2), and there is a heliport on the roof (Figure 3a). Designed as an office building according to 2001 California Building Code for seismic zone 4 and soil profile S_d, the earthquake forces were determined by linear response spectrum analysis (RSA) for the code design spectrum reduced by a response modification factor of 8.5.

Figure 4 shows the location and orientation of the fifteen strong-motion sensors installed in the building. The magnitude 5.4 Chino-Hills earthquake (2008)—centered at a distance of 40 km— did not cause any observable damage, and reliable data was recorded by all eight sensors in the *y*-direction, but only by sensors 9 and 13 in the *x*-direction. The acceleration records indicate that the peak acceleration of 0.026*g* at the ground was amplified to 0.042*g* at the roof of the building (Figure 5).

Analyzed by the PERFORM-3D computer program (CSI 2006), the building was modeled as follows: (1) Beams and columns were modeled by a linear element with tri-linear plastic hinges at the ends of the elements that can include in-cycle strength deterioration, but not cyclic stiffness degradation; the beam stiffness was modified to include the effect of the slab, and the axial load-moment interaction for the columns was based on plasticity theory; (2) The braces below the heliport were modeled using fiber sections to model buckling behavior; (3) Panel zones were modeled as four rigid links hinged at the corners with a rotational spring that represents the strength and stiffness of the connection; (4) The tab connections were modeled using rigid-perfectly-plastic hinges that can include in-cycle and cyclic degradation; (5) The contribution of non-structural elements was modeled by adding four shear columns located close to the perimeter of the building, with their properties obtained from simplified models of the façade and partitions; nonlinear behavior of these elements was represented using rigid-plastic shear hinges; (6) Ductility capacities of girders, columns, and panel zones were specified according to the ASCE/SEI 41-06 standard; (7) Columns of moment resisting frames and the gravity columns were assumed to be clamped at

the base; and (8) Effects of nonlinear geometry were approximated by a standard P-Δ formulation.

Figure 2. Nine-story symmetric-plan building in Aliso Viejo, CA.

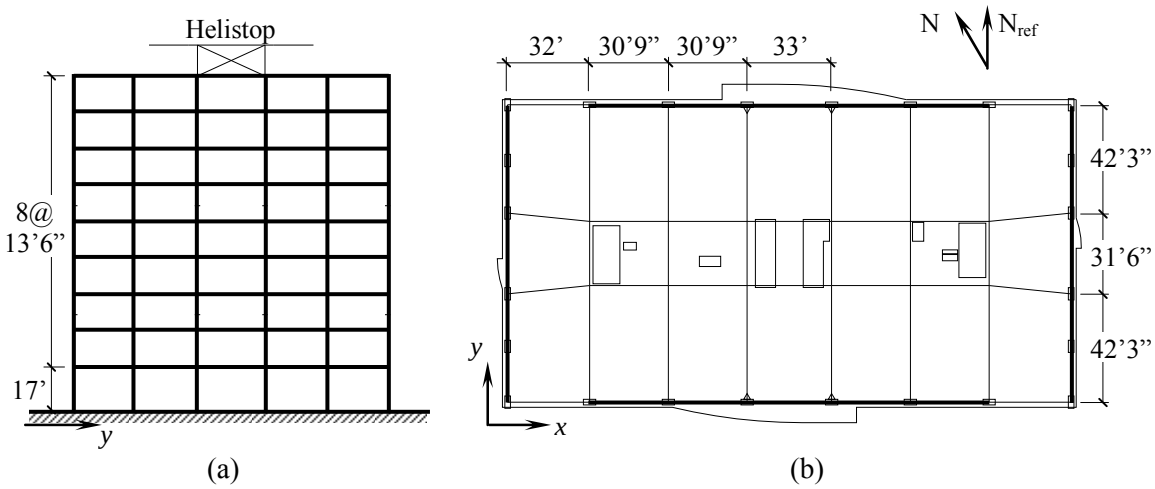

Figure 3. (a) West elevation; (b) typical floor plan of the selected 9-story symmetric-plan building.

Vibration properties—natural periods, natural modes, and modal damping ratios—of the building were determined from recorded motions (Figure 5) by two system-identification methods: the combined deterministic-stochastic subspace (DSS) method (Van Overshee and De Morre, 1996) and the peak-picking (PP) method. Remarkably close agreement between the calculated (from the computer model) and identified values of vibration periods and modes was achieved, as demonstrated in Table 2 (where the periods identified by both methods are listed) and in Figure 6 (where the natural vibration modes identified by the DSS

method are presented). As expected, the translational and torsional motions of this symmetric-plan building are uncoupled (Figure 6).

The DSS method also led to the modal damping ratios of 4.30, 3.30, 3.96, and 3.22 percent for the first, third, fourth, and sixth modes of vibration. Based on these data, energy dissipation in the building was modeled by Rayleigh damping, with its two constants selected to give 4.30% and 3.96% damping ratio at the vibration periods of the first and fourth modes, respectively. The resulting damping ratios for the first nine vibration modes of the buildings ranged from 3.7% to 7.4%.

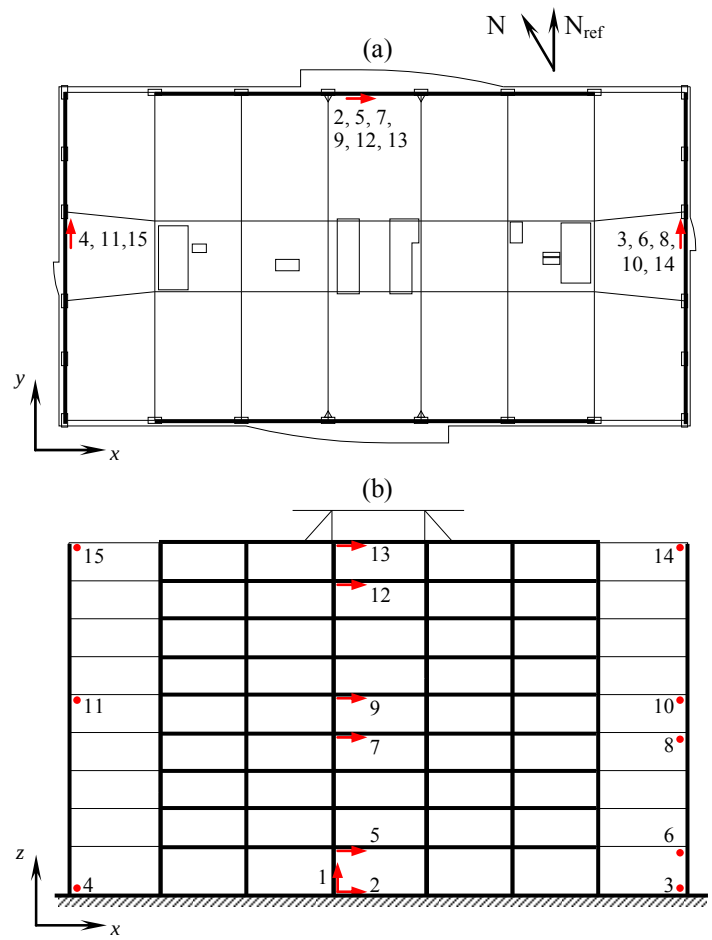

Figure 4. Locations of sensors in the 9-story symmetric-plan building: (a) plan view, and (b) south elevation.

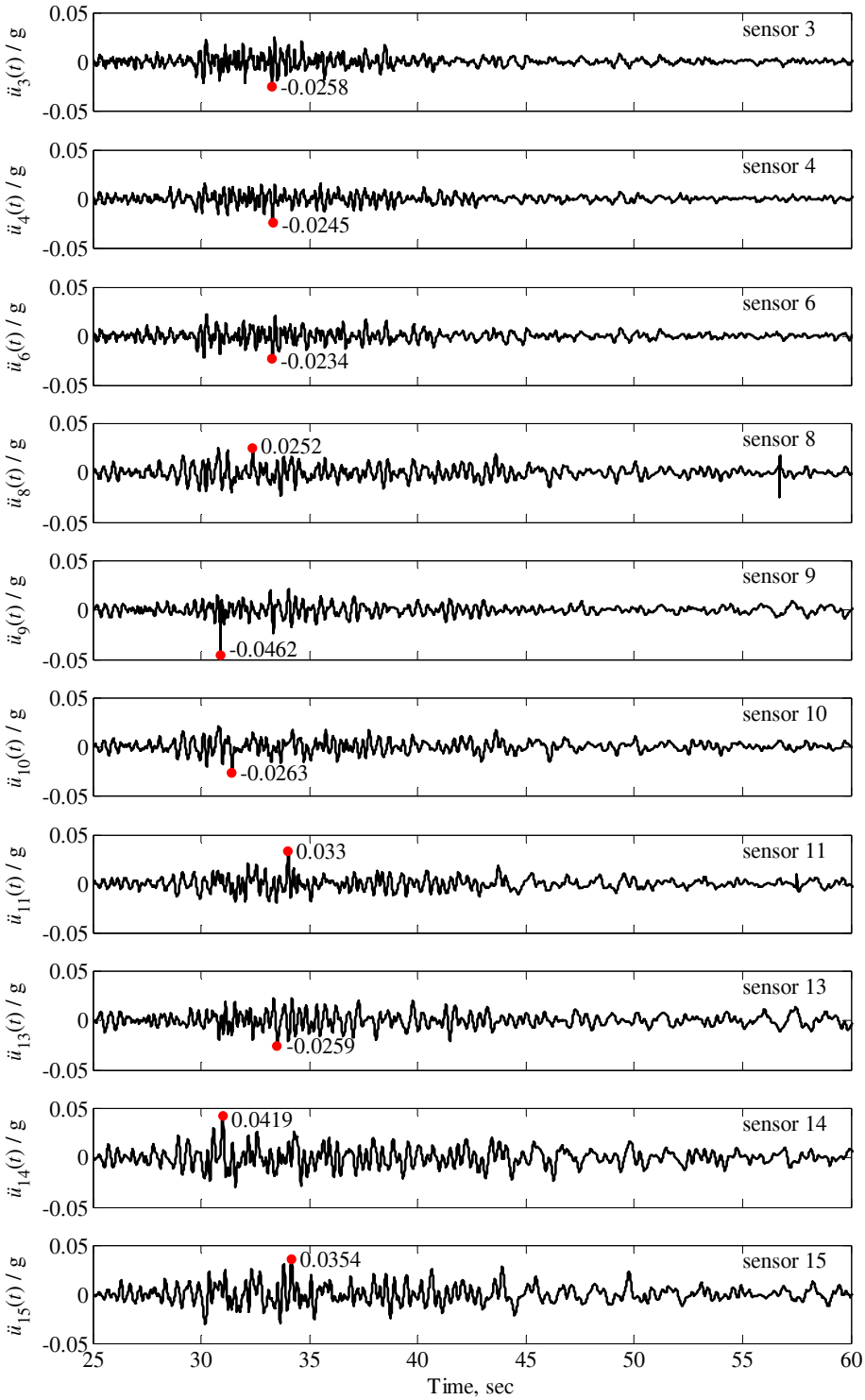

Figure 5. Floor accelerations recorded by the sensors showed in Figure 4 during the magnitude 5.4 Chino-Hills earthquake (2008).

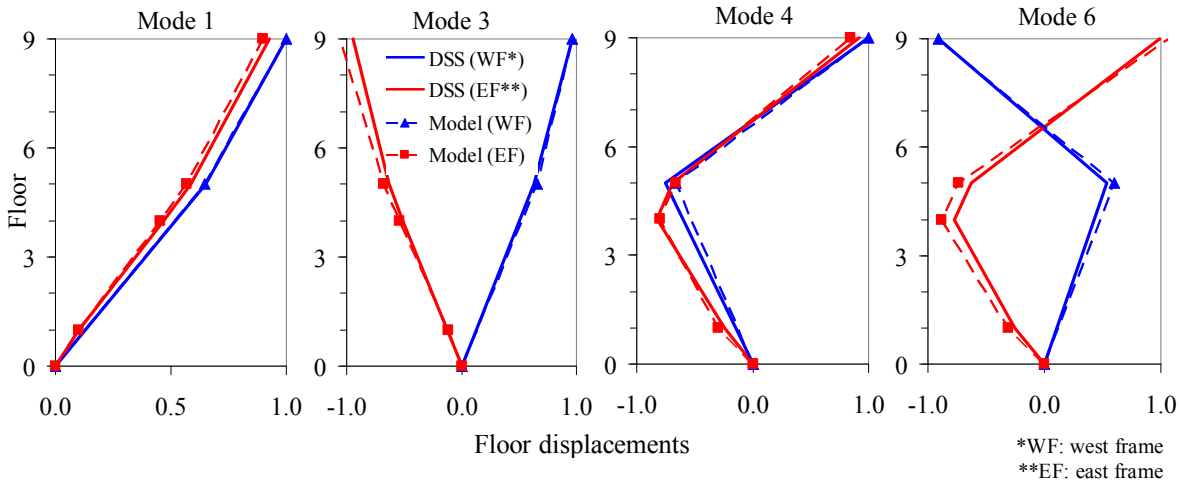

Figure 6. Comparison of natural vibration modes identified by the DSS method with modes of the computer model.

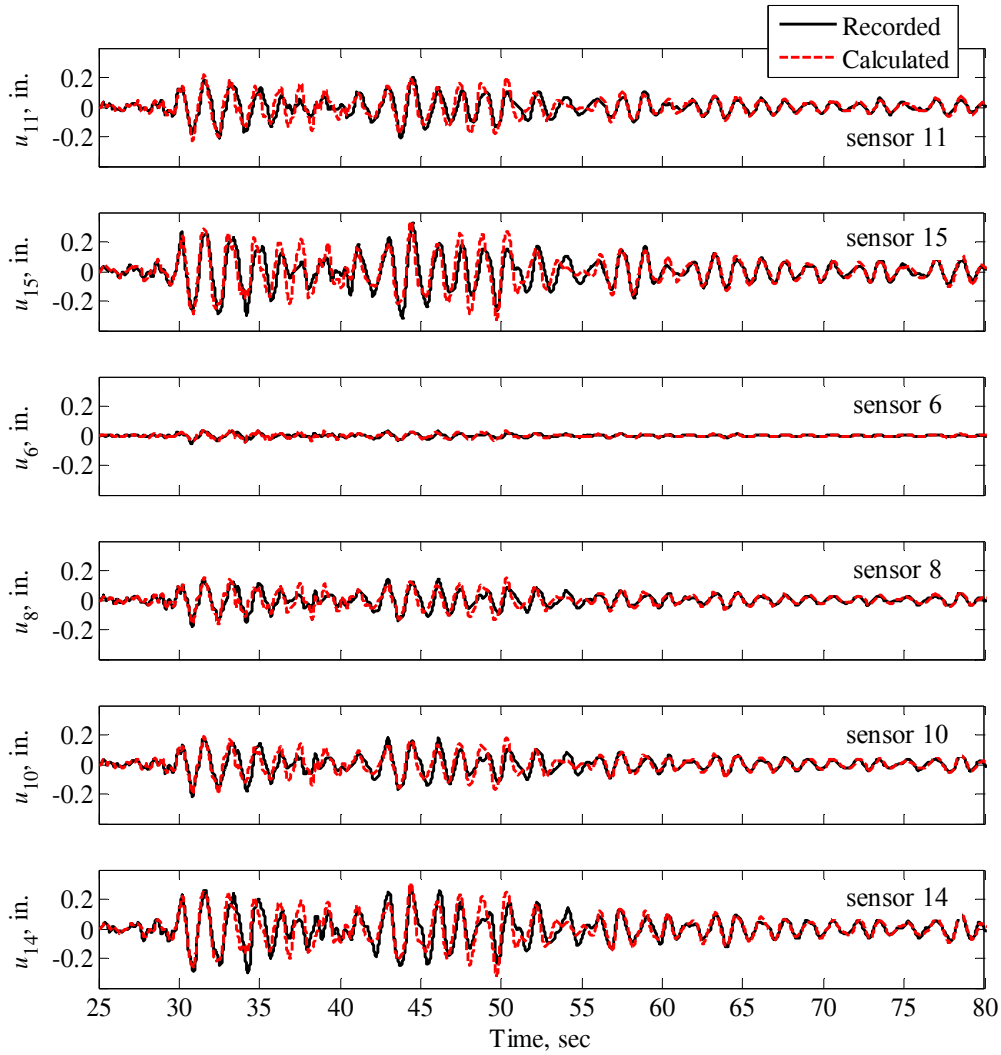

Figure 7. Comparison of recorded and computed floor displacements at the sensor locations (Figure 4). Recorded data is from the Chino Hills earthquake (2008).

The computer model accurately represented the properties of the building, as demonstrated by the excellent agreement between the computed and "recorded" responses. Nonlinear RHA of the computer model subjected to the recorded ground motion led to the displacements (relative to the ground) at floors 2, 5, 6, and 9, shown in Figure 7, where these computed responses were compared with the motions recorded by sensors 6, 8, 10, 11, 14, and 15; clearly, the agreement between the two is very good.

EVALUATING MPS PROCEDURE

The accuracy of the MPS procedure was evaluated by comparing the median (defined as the geometric mean) value of an EDP due to a set of seven scaled ground motions against the benchmark value, defined as the median value of the EDP due to the twenty-eight unscaled ground motions. 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.

BENCHMARK RESPONSES

Figure 8 shows the benchmark values of the EDPs: floor displacements (normalized by building height) and story drift ratios (story drift÷story height). Also included are the responses to individual records to demonstrate their large dispersion. Most of these ground motions drive the building far into the inelastic range, as demonstrated in Figure 9 where the deformation values of the inelastic SDF system due to twenty-eight ground motions are identified on the first-mode pushover curve. The median deformation exceeds the yield deformation by factors of 3.5 and 3.1 in the *x* and *y* directions, respectively. Recall that the two values of median deformation shown in Figure 4 are the target values $\hat{D}_1$ that are to be matched by the scaling procedure (Step 6).

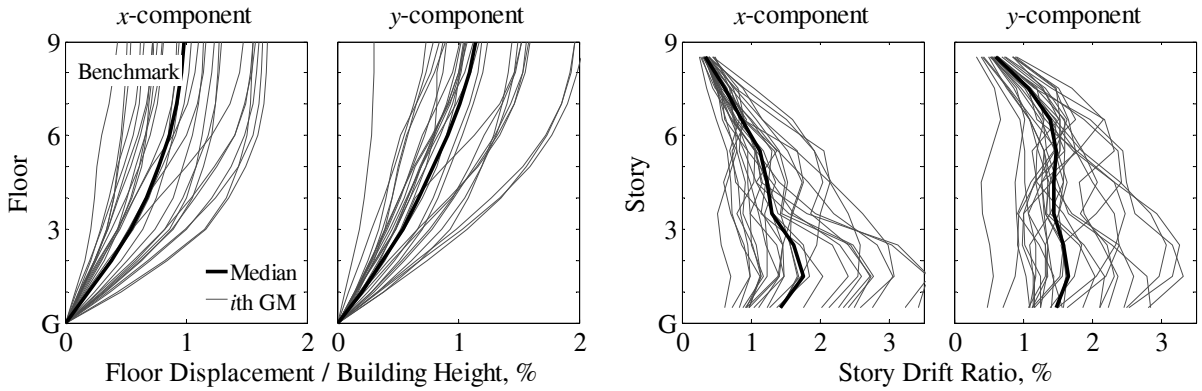

Figure 8. Median values of EDPs determined by nonlinear RHA of the building subjected to two components, simultaneously, of 28 unscaled records; individual results for the 28 excitations are included.

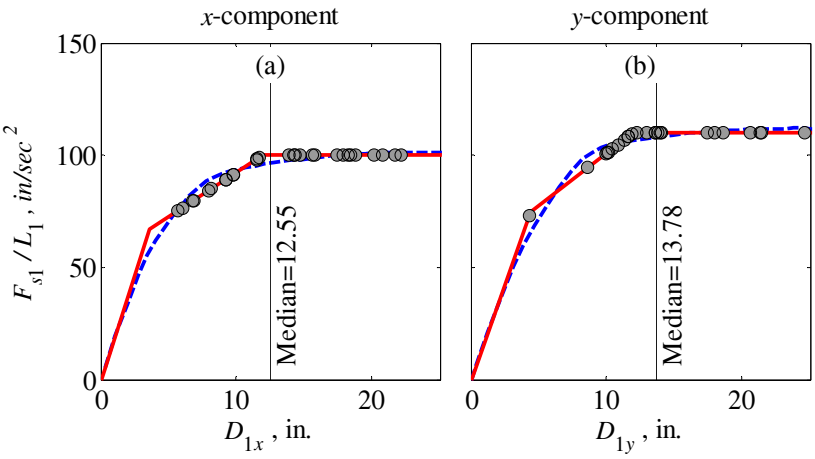

Figure 9. Force-deformation curves for the first "modes" of lateral vibration of the building in *x* and *y* directions and their tri-linear idealization. Peak deformations due to 28 unscaled records are identified.

ONE-MODE MPS PROCEDURE: SAME SCALING FACTOR

Scaling both components of a record by the same scaling factor does not ensure that the peak deformation of the first-"mode" inelastic SDF system to the scaled ground motion is close to the target value. Comparing the peak deformations of this system due to each of the scaled records (Figure 10)—with similar response data due to unscaled records (Figure 9) — indicates that the dispersion in response is reduced very little by scaling ground motions, implying that a principal goal of reducing dispersion is not achieved if both components are scaled by the same scaling factor.

If the intensities of the two horizontal components of a record are considerably different, perhaps no scaling procedure with a single scaling factor can lead to accurate values of EDPs in both *x* and *y* directions, simultaneously. This is demonstrated in Figure 11 where the

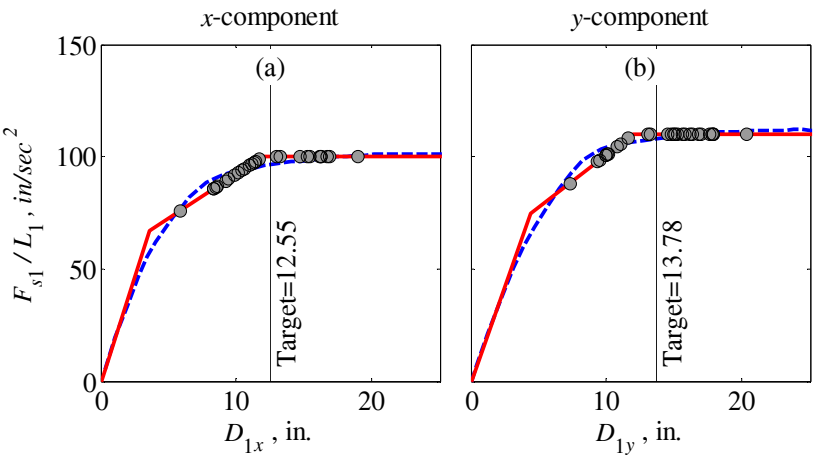

Figure 10. Force-deformation curves for the first "modes" of lateral vibration of the building in *x* and *y* directions and their tri-linear idealization. Peak deformations due to 28 records scaled by the one-mode MPS procedure with the same scaling factor for both components are identified.

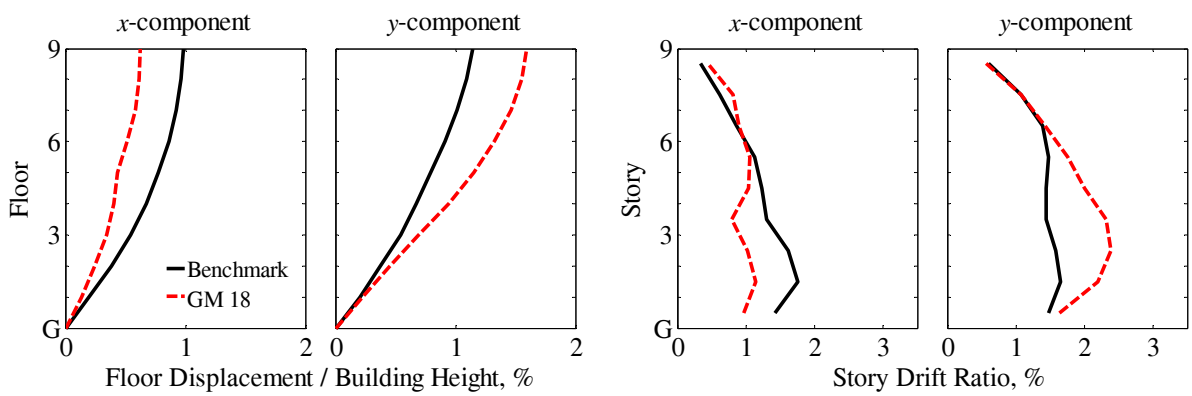

Figure 11. Comparison of EDPs due to unscaled record 18 (Table 1) against benchmark values.

values of EDPs due to the unscaled record 18 (Table 1) are shown together with the benchmark EDPs. This record underestimates the EDPs in the *x*-direction, but overestimates them in the *y*-direction. Obviously, no single scaling factor for the *x* and *y* components of this record would lead to accurate values of EDPs in both directions, simultaneously.

Therefore, it is not surprising that the MPS procedure using a single scaling factor for both components of a record leads to inaccurate estimates of the median EDPs accompanied by large record-to-record variability of the responses. This is evident in Figure 12 where the median values of EDPs due to the seven scaled ground motions of sets 1 and 2 are shown together with the benchmark EDPs; also included are the EDPs due to the individual ground motions to show their dispersion.

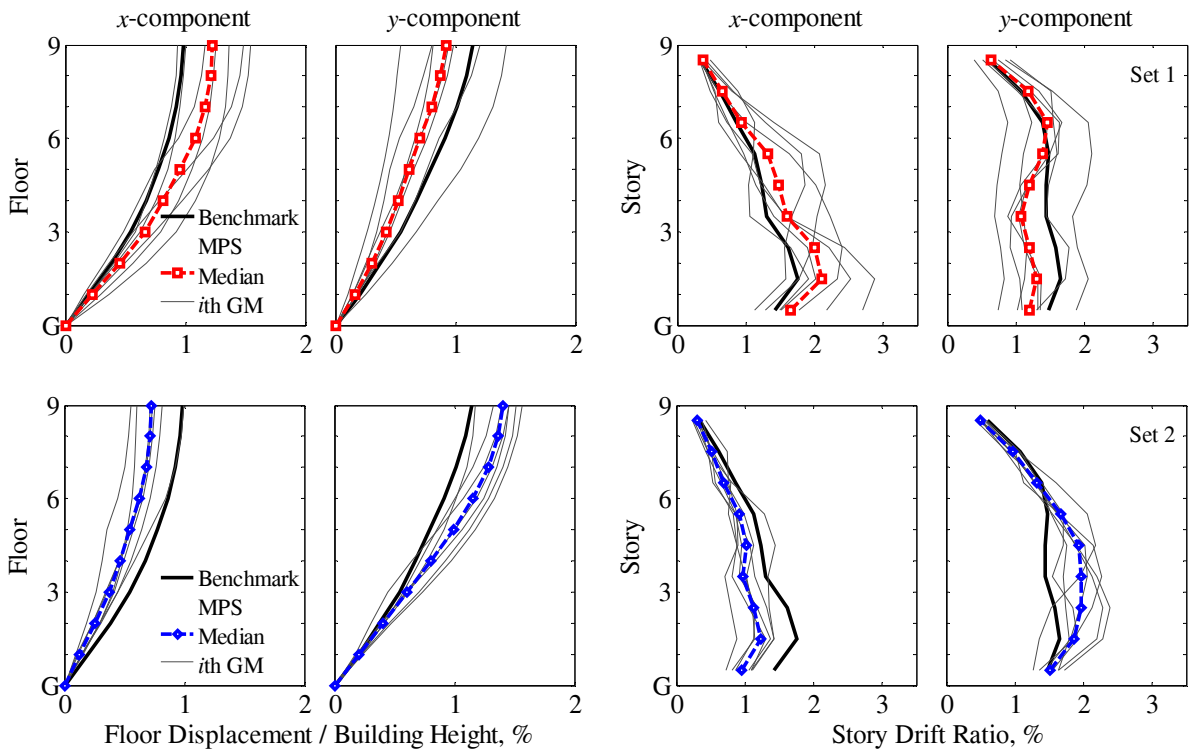

Figure 12. Median values of EDPs due to record sets 1 and 2 scaled by the one mode MPS procedure with the same scaling factor for both components of ground motion and the benchmark values of EDPs.

These results indicate that, in general, it may not be possible to achieve accurate estimates of EDPs if both horizontal components of a record are to be scaled by the same factor. This hurdle can be overcome by selecting those ground motions that satisfy the requirement that the peak deformation of the first-"mode" inelastic SDF system due to each unscaled component is close to the target value. Unfortunately, such a restriction will reduce the number of useable records and, in particular, eliminate near-fault records because their fault-normal and fault-parallel components are known to be very different due to directivity effects and fling. Furthermore, such restrictions may not be practical if the target deformations in the two directions are very different because the lateral-force-resisting systems of the structure are different.

To overcome these restrictions, two different scaling factors for the two components of ground motion is an alternative option. Seismologists may find this liberal approach to be unacceptable because it does not preserve focal mechanism and wave travel path effects, inherent in recorded motions. However, if the goal of any scaling procedure is to estimate the EDPs accurately—where the benchmark values are determined from a large set of unscaled

records, which obviously preserve all the seismological features—then such an approach seems justified.

ONE-MODE MPS PROCEDURE: DIFFERENT SCALING FACTORS

The MPS scaling procedure allowing for different scaling factors for the two components of a records is promising because the peak deformation of the first-"mode" inelastic SDF system due to each scaled component is identical to the corresponding target deformation, as shown in Figure 13. As a result, the MPS procedure provides an accurate estimate of the median EDPs and reduces the record-to-record variability of the responses. This is demonstrated in Figure 14 where the median values of EDPs due to the seven scaled records of sets 1 and 2 are shown together with the benchmark EDPs; also included are the EDPs due to each of the seven scaled records to show their dispersion. The height-wise average discrepancy in floor displacements is 11% and 5% for record sets 1 and 2, respectively; in story drifts this discrepancy is 12% and 10% for record sets 1 and 2, respectively. As will be seen later, this discrepancy will be greatly reduced when the response in the second "mode" of vibration is considered in ranking and selecting ground motions. When the two components were scaled by the same scaling factor, the height-wise average discrepancy was much larger: 22% and 24% in floor displacements due to sets 1 and 2, respectively; and 15% and 20% in story drifts due to sets 1 and 2, respectively. The MPS procedure is efficient in the sense that the dispersion (record-to-record variability) of the EDPs due to scaled records (Figure 14) is much smaller than the dispersion of the responses to unscaled records (Figure 12); numerical values of the dispersion are available in (Reyes 2009).

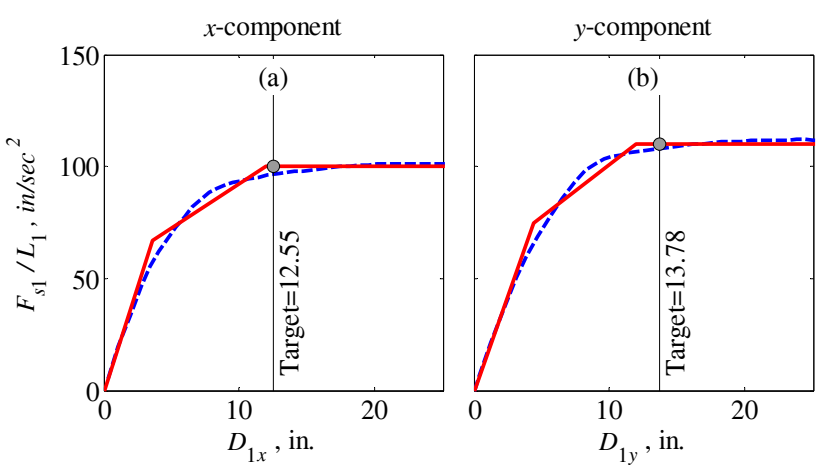

Figure 13. Force-deformation curves for the first "modes" of lateral vibration of the building in *x* and *y* directions and their tri-linear idealization. Peak deformations due to 28 records scaled by the one-mode MPS procedure are identified.

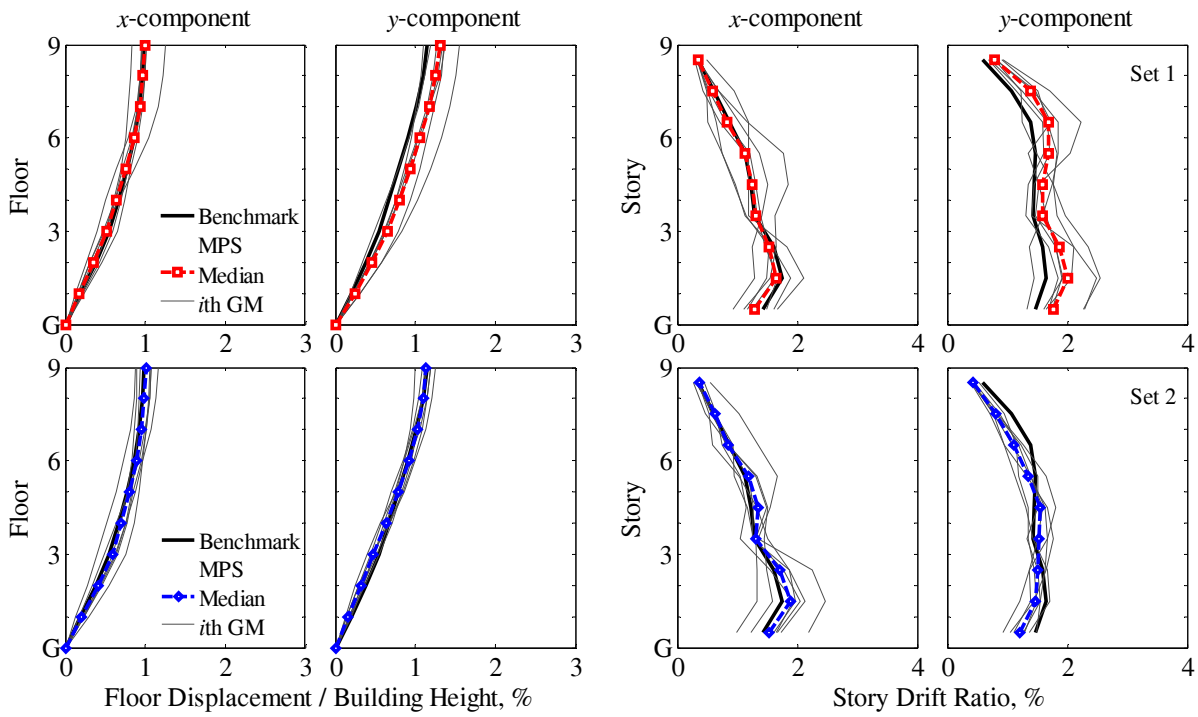

Figure 14. Comparison of EDPs due to record sets 1 and 2 scaled according to the one-mode MPS procedure and the benchmark EDPs; individual results for seven scaled records are included.

COMPARATIVE EVALUATION OF ONE-MODE MPS AND ASCE7 SCALING PROCEDURES

The one-mode MPS procedure for scaling ground motions leads to much more accurate estimates of seismic demands compared to the ASCE7 scaling procedure. Figure 15 presents the median values of EDPs due to the seven records of sets 1 and 2, scaled by the ASCE7 procedure together with the benchmark EDPs; also included are the EDPs due to each of the seven scaled records to show their dispersion. For each record set, the records scaled according to the MPS procedure provide median values of EDPs that are much closer to the benchmark values than is achieved by the ASCE7 scaling procedure; compare Figures 14 and 15. The height-wise average discrepancy of 22% in floor displacements encountered by scaling set 2 records according to the ASCE7 procedure is reduced to 5% when these records are scaled by the MPS procedure; likewise, the height-wise average error in story drift ratios is reduced from 16% to less than 10%. The record-to-record variability is much less in EDPs due to a set of records scaled by the one-mode MPS procedure (Figure 14) compared to the records scaled by the ASCE7 procedure (Figure 15); numerical values for the dispersion are available in (Reyes 2009).

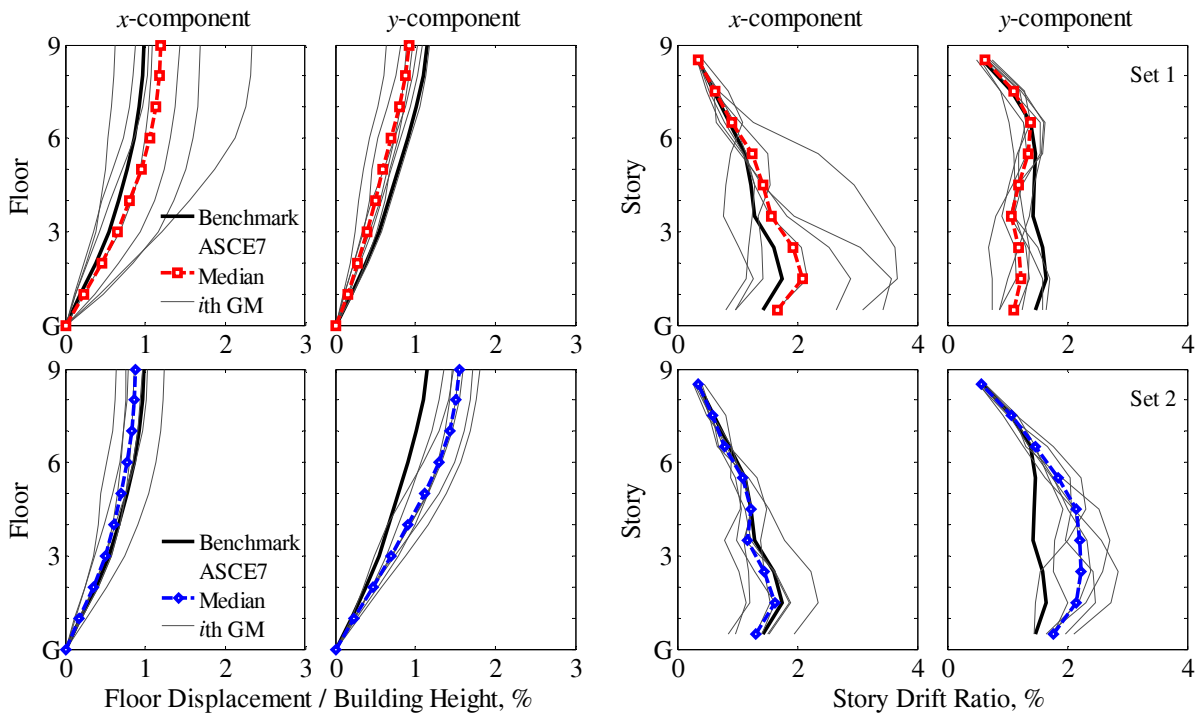

Figure 15. Comparison of EDPs due to record sets 1 and 2 scaled by the ASCE7 procedure against benchmark values; individual results for seven scaled records are included.

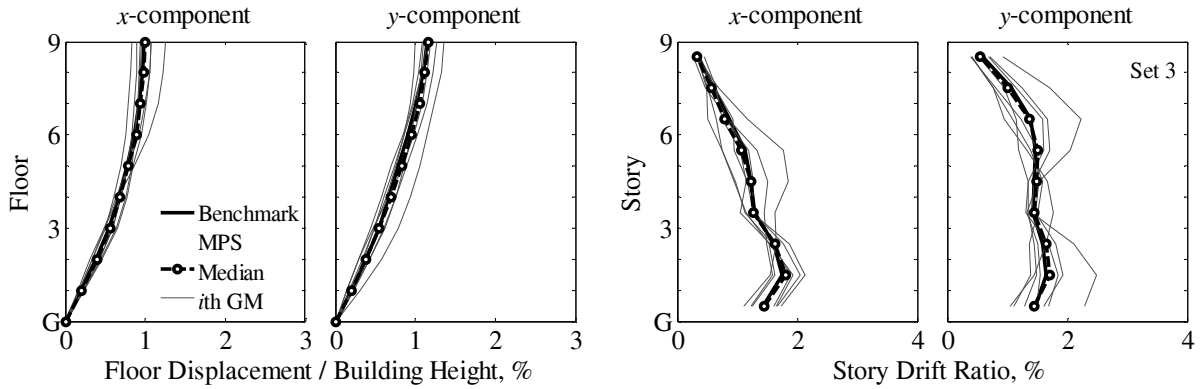

Figure 16. Comparison of median EDPs for record set 3 scaled by the MPS procedure (considering higher modes) with benchmark EDPs; individual results for the seven scaled records are included.

MPS WITH HIGHER MODE CONSIDERATIONS

The reduction in dispersion achieved by the MPS procedure in the drifts in higher stories of the building is less significant (Reyes 2009), suggesting that higher-"mode" responses should be considered in identifying the "best" scaled records. The fourteen records of sets 1 and 2, scaled by the one-mode MPS procedure, were ranked by considering their accuracy in estimating the second mode response (Steps 9 to 12 of the MPS procedure), and the seven records with the highest rank (according to Step 12) were defined as record set 3.

Considering the second "mode" in ranking and selecting the ground motions in the MPS procedure provides accurate estimates of the median EDPs and reduces slightly the record-to-record variability, compare Figures 14 and 16. This improvement in accuracy is demonstrated in Figure 16 where the median values of floor displacements and story drifts due to record set 3 are shown together with the benchmark values. It is evident by comparing Figures 14 and 16 that this new set leads to more accurate estimates of median demands compared to sets 1 and 2; the height-wise average discrepancy in floor displacements is reduced from 11% (set 1) to 2% (set 3); in story drift ratios, this discrepancy is reduced from 12% (set 1) to 4% (set 3). Thus, the MPS method considering higher mode contributions to response selects a set of scaled records for nonlinear RHA of the building that provide highly accurate estimates of EDPs, which are even more superior to the ASCE7 procedure than was possible with ground motion sets 1 and 2 where higher modes were not considered; this is evident by comparing Figures 14 and 15 to Figure 16.

The efficiency achieved by the MPS procedure is demonstrated in Figure 17, wherein the dispersion values, defined as the standard deviation of the logarithm of observed values of the EDPs, for demands due to scaled and unscaled ground motions are compared. It is apparent that the dispersion in the EDP values due to the seven scaled records around the median value is much smaller.

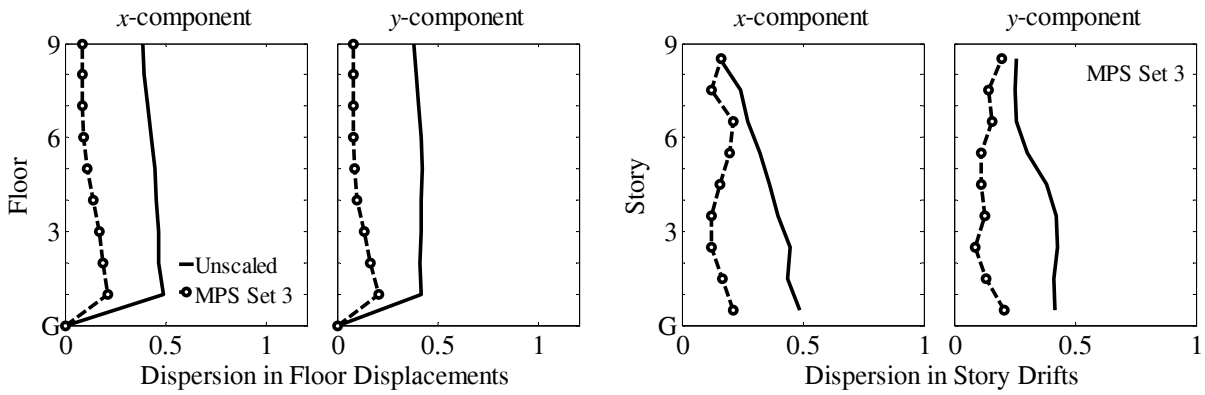

Figure 17. Comparison of dispersion of EDPs for record set 3 scaled by the MPS procedure (considering higher modes) against dispersion of 28 unscaled records.

CONCLUSIONS

With the goal of developing effective procedures for selection and scaling of multi-component ground motion records to be used in nonlinear RHA of structures, a modal-pushover-based-scaling (MPS) procedure was developed. The objective of this amplitude

scaling procedure was to determine scale factors for a small number of records such that the scaled records provide an accurate estimate of median structural responses, and are also efficient, i.e., reduce the record-to-record variability of response. The developed MPS procedure is an extension of the original MPS procedure for one component of ground motion to two horizontal components. The accuracy of the extended MPS procedure was evaluated by comparing the median values of the engineering demand parameters (EDPs) for a selected building due to a set of seven records scaled according to the MPS procedure against the benchmark values, defined as the median values of the EDPs due to 28 unscaled records. The efficiency of the MPS and ASCE7 scaling procedures was evaluated by computing the dispersion of the responses to the seven scaled ground motions; small dispersion indicates that the scaling procedure is efficient. Selected for this investigation is an actual 9-story symmetric-plan building with its computer model calibrated against its motions recorded during an earthquake. This evaluation of the MPS procedure has led to the following conclusions:

1. The one-mode MPS procedure using a single scaling factor for both components of ground motion was judged to be unacceptable because it led to inaccurate estimates of the median EDPs accompanied by large record-to-record variability of the responses.

2. If the intensity or frequency characteristics of the two components of a ground motion record differ significantly, as for example in many near-fault records, perhaps no amplitude scaling procedure with the same scaling factor for both components can be relied upon to always deliver accurate values of EDPs in both horizontal directions, simultaneously.

3. The MPS procedure allowing for different scaling factors for the *x* and *y* components of a record provided a highly accurate estimate of the median EDPs and reduced the record-to-record variability of the responses; in particular, the height-wise average discrepancy in story drift ratios was less than 4% relative to the benchmark values.

4. The MPS procedure is much superior compared to the ASCE7 procedure for scaling two components of 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 ASCE7 procedure. The height-wise average discrepancy of 15% in story drift ratios (relative to the benchmark values) determined by scaling records according to the ASCE7 procedure

is reduced to 4% 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 ASCE7 scaling procedure.

APPENDIX A

Our implementation of the ASCE7 procedure is summarized as a sequence of steps:

1. Obtain the target pseudo-acceleration spectra $\hat{A}_x(T)$ and $\hat{A}_y(T)$ for the x and y components of ground motion for the building site, as described in Step 1 of the MPS procedure. Define $\hat{\mathbf{A}}_x$ and $\hat{\mathbf{A}}_y$ as vectors of spectral values $\hat{A}_i$ at different periods T_i over the period range from $0.2T_1$ to $1.5T_1$, 100 equally spaced period values were chosen.
2. Calculate the amplified target spectrum $\hat{\mathbf{A}} = 1.3\left(\hat{\mathbf{A}}_x + \hat{\mathbf{A}}_y\right)/2$.
3. Select seven ground motion records appropriate for the site, based on the criteria specified in ASCE7.
4. For the x and y components of a record, calculate the 5%-damped response spectra $A_x(T)$ and $A_y(T)$, and the vectors $\mathbf{A}_x$ and $\mathbf{A}_y$ of spectral values at periods T_i (same as in Step 1).
5. Compute the SRSS spectrum $\mathbf{A}_{\text{SRSS}}$ for each record: $A_{SRSS,i} = \sqrt{\left(A_{x,i}\right)^2 + \left(A_{y,i}\right)^2}$ where $A_{x,i} = A_x(T_i)$ and $A_{y,i} = A_y(T_i)$.
6. Determine the scaling factor SF_1 for each record that minimizes the difference between the amplified target spectrum (Step 2) and the SRSS spectrum for the record (Step 5) by solving the following minimization problem: $\min_{SF_1} \left\| \hat{\mathbf{A}} - SF_1 \times \mathbf{A}_{\text{SRSS}} \right\| \Rightarrow SF_1$ where $\left\| \cdot \right\|$ is the Euclidean norm**.** Required for this purpose is a numerical method to minimize scalar functions of one variable. This minimization ensures that the scaled SRSS spectrum is as close as possible to the amplified target spectrum, as shown schematically in Figure A.1.
7. Determine $\hat{\mathbf{A}}_{\text{SRSS}}$, defined as the average of $SF_1 \times \mathbf{A}_{\text{SRSS}}$ over the set of records.

8. Calculate the maximum normalized difference $\varepsilon_{\text{ASCE}}$ between $\hat{\mathbf{A}}$ (Step 2) and $\hat{\mathbf{A}}_{\text{SRSS}}$ (Step 7) over the period range from $0.2T_1$ to $1.5T_1$; i.e. $\varepsilon_{\text{ASCE}} = \max_{0.2T_1 \le T_i \le 1.5T_1} (\hat{A}_i - \hat{A}_{\text{SRSS},i}) \div \hat{A}_i$, where $\hat{A}_i$ and $\hat{A}_{\text{SRSS},i}$ are the elements of $\hat{\mathbf{A}}$ and $\hat{\mathbf{A}}_{\text{SRSS}}$ at vibration period T_i, respectively. Determine the scaling factor $SF_2 = 0.9/1-\varepsilon_{\text{ASCE}}$ if $\varepsilon_{\text{ASCE}} > 0.1$; otherwise, $SF_2 = 1.0$; note that SF_2 applies to the entire set of records.

9. Determine the final scaling factor $SF = SF_1 \times SF_2$ for each record. Scaling each record by its unique factor SF ensures that the average of the SRSS spectra for the scaled records does not fall below 1.3 times the target spectrum by more than 10 percent over the period range $0.2T_1$ to $1.5T_1$ (Figure A.2).

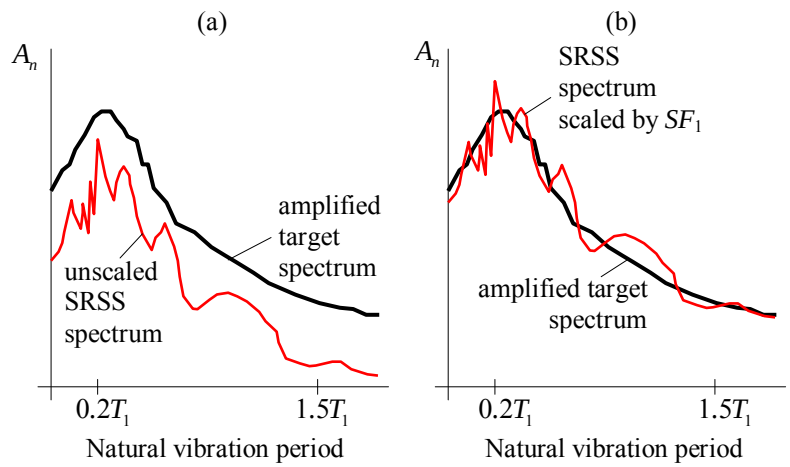

Figure A.1. Schematic illustration of Step 6 of the ASCE7 scaling procedure

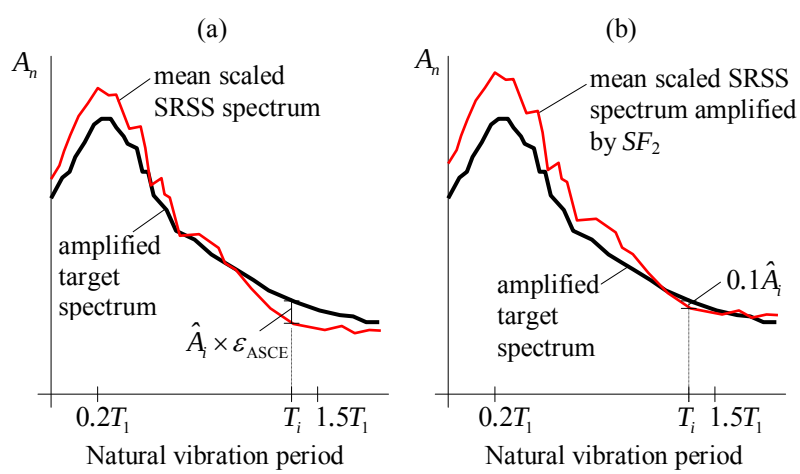

Figure A.2. Schematic illustration of Step 8 of the ASCE7 scaling procedure

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