Finite Element Model Calibration of An Instrumented Thirteen-story Steel Moment Frame Building in South San Fernando Valley, California

By Erol Kalkan

Disclamer

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Acknowledgments

Ground motions were recorded at a station owned and maintained by the California Geological Survey (CGS). Data can be downloaded from CESMD Virtual Data Center at: *http://www.strongmotioncenter.org/cgi-bin/CESMD/StaEvent.pl?stacode=CE24567*.

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Introduction

The thirteen-story instrumented building (Figure 1) is located in South San Fernando Valley about 5 km southwest of the Northridge epicenter. The structure is composed of one basement and thirteen floors above ground (Figure 2). It was built in 1975 on a design based on the 1973 UBC. The building has been the subject of previous investigations (Kalkan and Kunnath 2004, 2006; Kunnath et al. 2004, Uang et al. 1995; Kalkan and Chopra, 2010). The footprint of the building is 160 x 160 feet (53.3 x 53.3 m). The exterior frames are the moment resisting frames and interior frames are for load bearing. The foundation consists of piles, pile caps and grade beams. The floor plan of the perimeter frames and a typical elevation of one of these frames are shown in Figure 3. Member sizes are given in the Table 1. The corner columns are composed of box sections.

The typical floor system consists of 2.5 in (6.4 cm) of concrete fill over 3 in (7.5 cm) 20-gage steel decking. The roof system is lighter with 2.25 in (5.7 cm) vermiculite fill on 3 in 20-gage steel decking. 3 ksi concrete is specified for all deck fill. Exterior walls are composed of 6 in (15.2 cm) 22 gage steel studs with 0.25 in (0.6 cm) opaque glass and 2 in (5 cm) precast panels.

A total uniform load of 102.5 psf is used to calculate the building mass properties and axial load on columns.

Figure 1. Photo of thirteen-story instrumented building (source: *http://www.strongmotioncenter.org/cgi-bin/CESMD/stationhtml.pl?stationID=CE24567&network=CGS*).

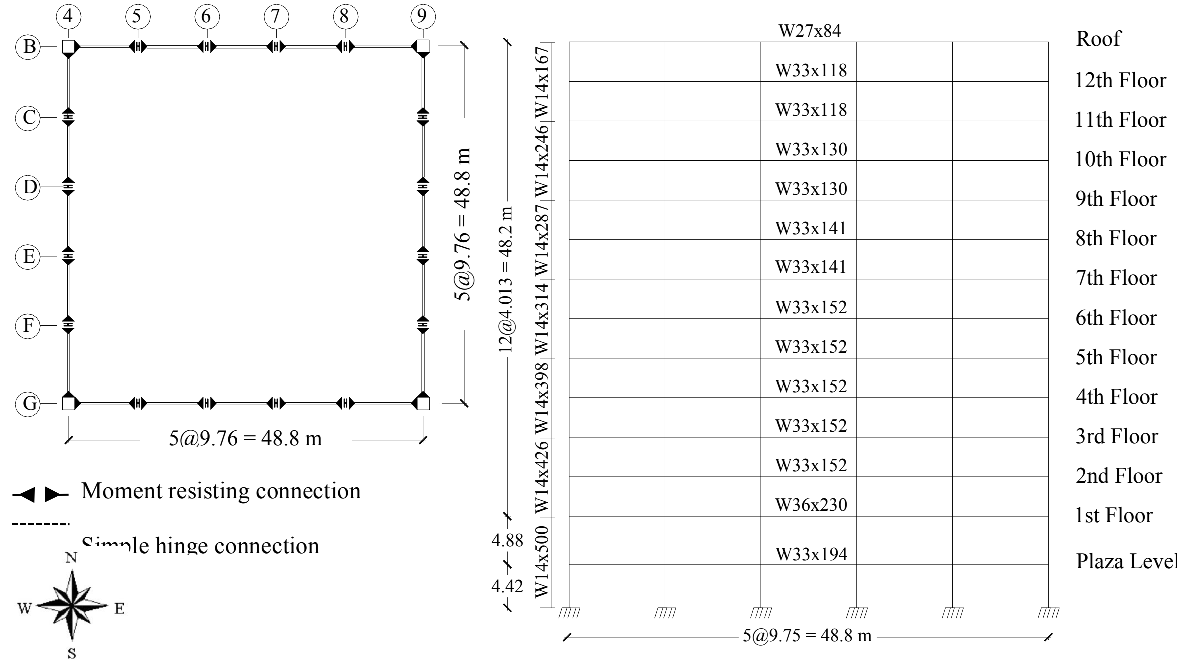
Table 1. Column and beam sections.

COLUMNS

Story	A	B	C	D	E	F
Plaza Level	W14x500	W14x500	W14x500	W14x500	W14x500	W14x500
1	W14x500	W14x500	W14x500	W14x500	W14x500	W14x500
2-3	W14x426	W14x426	W14x426	W14x426	W14x426	W14x426
4-5	W14x398	W14x398	W14x398	W14x398	W14x398	W14x398
6-7	W14x314	W14x314	W14x314	W14x314	W14x314	W14x314
8-9	W14x287	W14x287	W14x287	W14x287	W14x287	W14x287
10-11	W14x246	W14x246	W14x246	W14x246	W14x246	W14x246
12-13	W14x167	W14x167	W14x167	W14x167	W14x167	W14x167

BEAMS

Story	A-B	B-C	C-D	D-E	E-F
Plaza Level	W33x194	W33x194	W33x194	W33x194	W33x194
1	W36x230	W36x230	W36x230	W36x230	W36x230
2-6	W33x152	W33x152	W33x152	W33x152	W33x152
7-8	W33x141	W33x141	W33x141	W33x141	W33x141
9-10	W33x130	W33x130	W33x130	W33x130	W33x130
11-12	W33x118	W33x118	W33x118	W33x118	W33x118
Roof	W27x84	W27x84	W27x84	W27x84	W27x84



(a) Plan view of perimeter frames (b) Elevation

Figure 2. (a) Plan and (b) elevation views of thirteen-story building.

Strong motion data is available for seven sensors: three each in the North-South and East-West directions, respectively, and one in the vertical direction. The sensors are located in the basement and on the sixth and twelfth floors as shown in Figure 3.

OpenSEES Model

Analytical model of the thirteen-story building is created using a typical two-dimensional frame (see line-G in Figure 2). A force-based nonlinear beam-column element that utilizes a layered '*fiber'* section is utilized to model all components of the frame models. Centerline dimensions are used in the element modeling. For the time-history evaluations, one half of the total building mass is applied to the frame distributed proportionally to the floor nodes. The modeling of the members and connections is based on the assumption of stable hysteresis derived from a bilinear stress-strain model with 2 percent strain hardening. In constructing the computer models, the columns are assumed to be fixed at the base level. Rayleigh damping of 5 percent is taken for the first three vibration modes.

The FEM model has the following modules:

1. gravity load analysis,
2. Eigen analysis,
3. nonlinear static (pushover) analysis, and
4. nonlinear response history analysis.

An example ground motion set is provided under "GMs" folder. To run the FEM model, call “main.tcl” using the opensees.exe file provided, other tcl files are supplementary. The model may not run properly if different exe file is used. Alternatively, run.m may be used in MatLAB to run the model. It is tested only for Windows. The OpenSEES model is used in Kalkand and Kunnath (2004; 2006) and Kalkan and Chopra (2010).

Calibration of OpenSEES Model to Observed Response

The strong motion accelerograms recorded in this building come from 1991 Sierra Madre and 1994 Northridge earthquakes (Table 2). The recorded accelerations at the basement indicate that the building experienced a PGA of 0.41 g in the North-South (NS) direction and 0.32 g in the East-West direction. Approximately 12 percent of the connections on the west perimeter of the NS frame fractured during the Northridge earthquake. Connection fractures on the remaining three sides were less than half this number. The stronger component of the Northridge earthquake was oriented in the NS direction. Damages in the NS frame consisted of (i) full or partial cross-flange cracks in the columns, (ii) flange cracking away from the heat-affected zone, (iii) fracture through the weld metal across partial or full width of the beam, (iv) weld fractures at beam-column interface, and (v) crack at the root of the weld (as identified by ultrasonic testing). Additional details related to observed damage are in Uang et al. (1995).

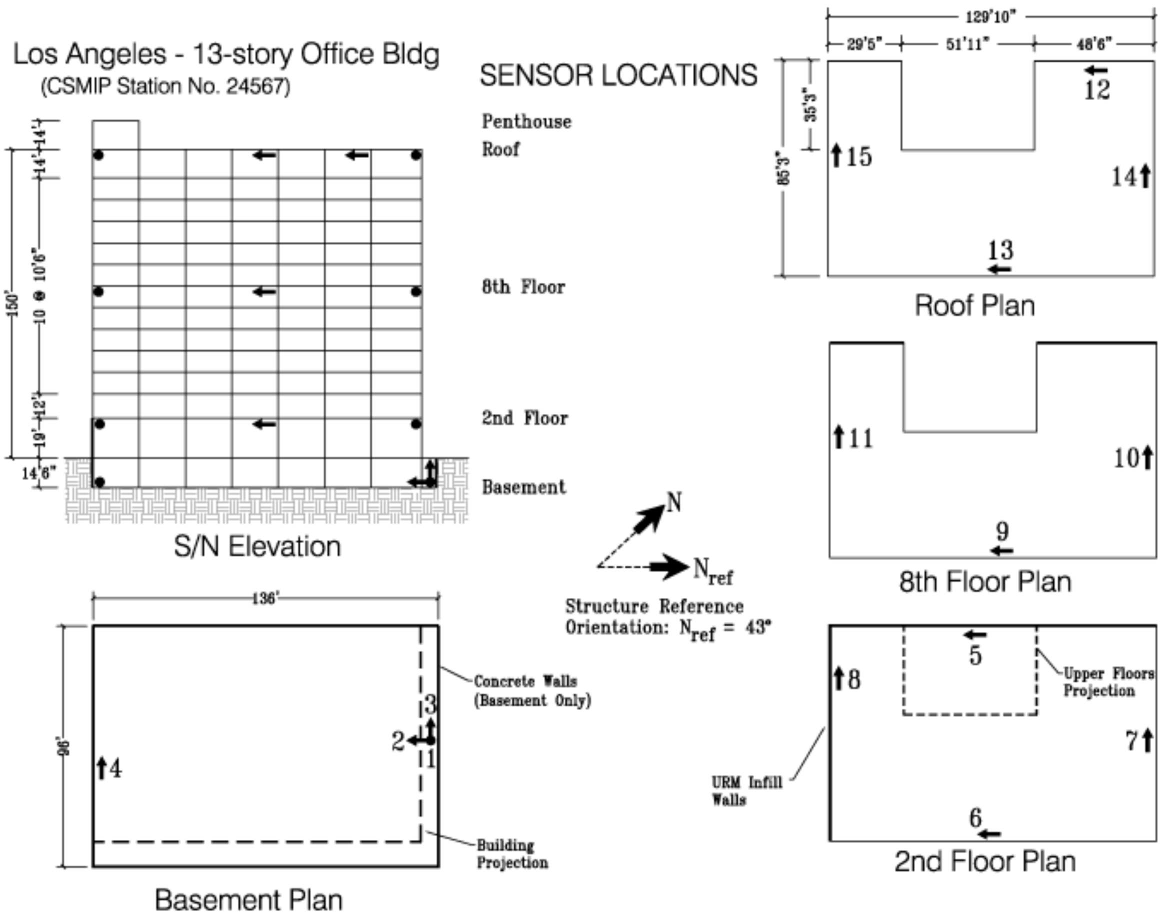


Figure 3. Sensor locations (source: *http://www.strongmotioncenter.org/cgi-bin/CESMD/stationhtml.pl?stationID=CE24567&network=CGS*).

The finite element model created in OpenSEES is not intended to simulate the connection fractures, therefore stable hysteretic model is used in material level. As shown in Figure 4, satisfactory results were obtained when the calibrated model response (displacement at the roof level) is compared with the recorded response from the Northridge Earthquake.

Table 2. Recorded PGA values in the thirteen-story building.

Earthquake	Magnitude (Mw)	Epicentral Distance (km)	PGA Base Level (g)	PGA Roof Level (g)
1994 Northridge	6.7	32	0.18	0.37
1991 Sierra Madre	5.8	33	0.17	0.18

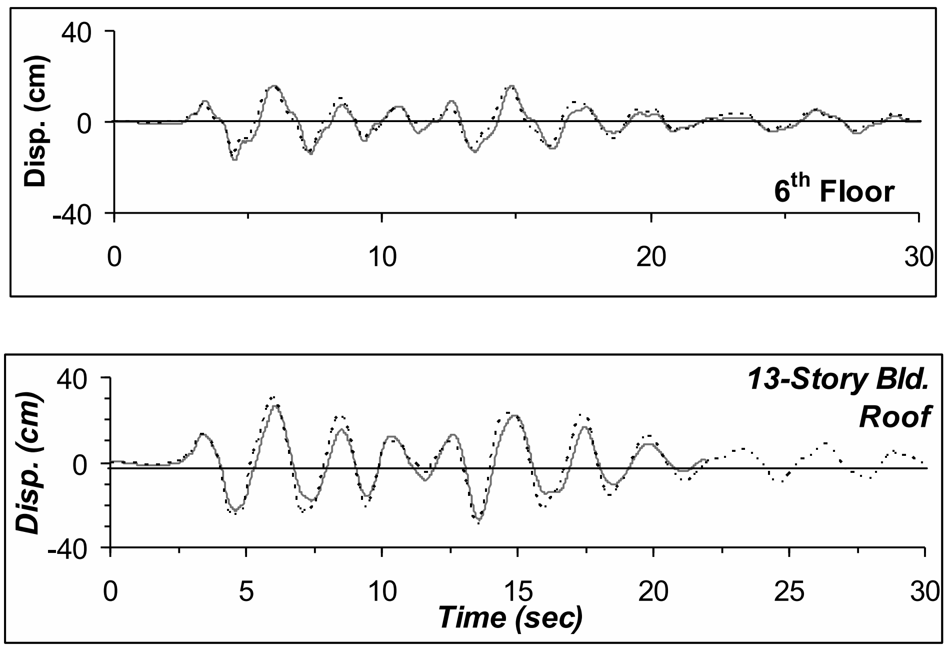


Figure 4. OpenSEES model validation (recorded and computed response at mid and roof levels).

Total story stiffness and mass values are listed in Table 3. These values are used to calculate the elastic modal attributes including modal periods, modal participation factors and modal mass ratios of the first three modes (see Table 4). The modal shapes for the first three modes are shown in Figure 5.

Table 3. Story stiffness and mass variation for the thirteen-story building.

Story No	Story Mass (kip-sec²/in)	Story Stiffness (k/in)	Diagonal of Stiffness Matrix (k/in)
1	4.07	3254.0	5676.0
2	3.89	2422.0	5916.0
3	3.50	3494.0	6988.0
4	3.50	3494.0	6670.0
5	3.50	3176.0	6352.0
6	3.50	3176.0	5468.0
7	3.50	2292.0	4584.0
8	3.50	2292.0	4324.8
9	3.50	2032.8	4065.6
10	3.50	2032.8	3831.8
11	3.50	1799.0	3598.0
12	3.50	1799.0	2804.8
13	3.50	1005.8	2011.6
14	1.75	1005.8	1005.8

Table 4. Elastic modal properties of the thirteen-story building.

	Elastic Modes		
13-Story Building	Mode-1	Mode-2	Mode-3
Modal Periods (sec), Tn	3.03	1.08	0.65
Modal Participation Factors, Γn	5.57	2.13	1.29
Mass Participation Factors, αn	0.77	0.11	0.04

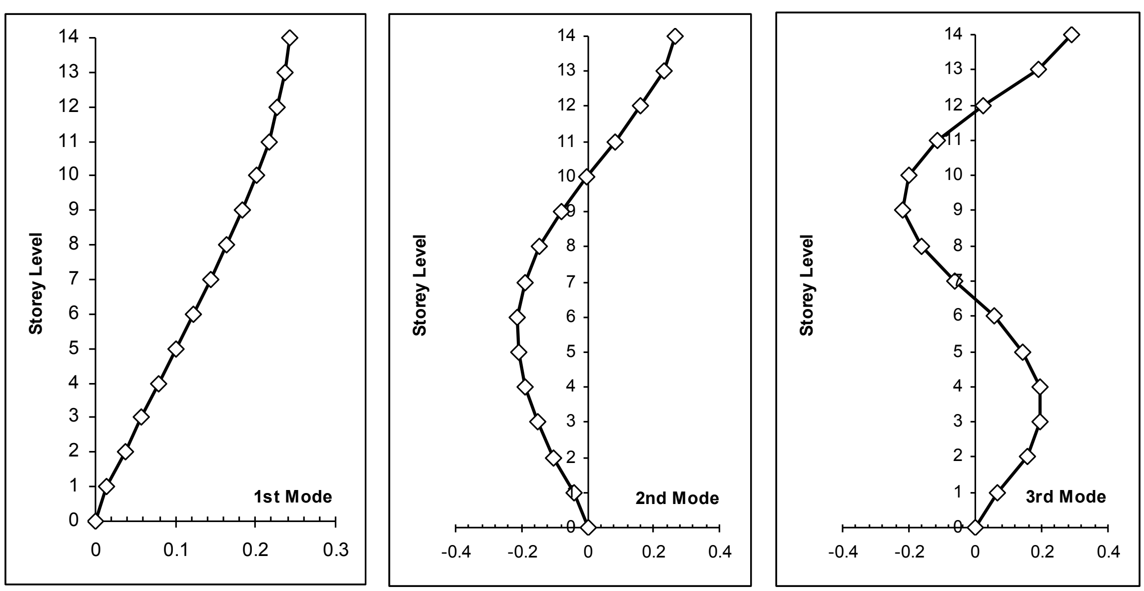


Figure 5. Elastic modal shapes of the first three modes.

Capacity Curves

Pushover analyses are applied using load vectors for the first three elastic modes as shown in Figure 6. The resultant capacity curves in terms of normalized base shear with reactive weight versus roof drift ratio are presented in Figure 7. Also shown in this figure are the interstory and roof drift ratio profiles obtained at the end of each pushover analysis. The resultant deformed shape is in agreement with the shape of applied load vectors.

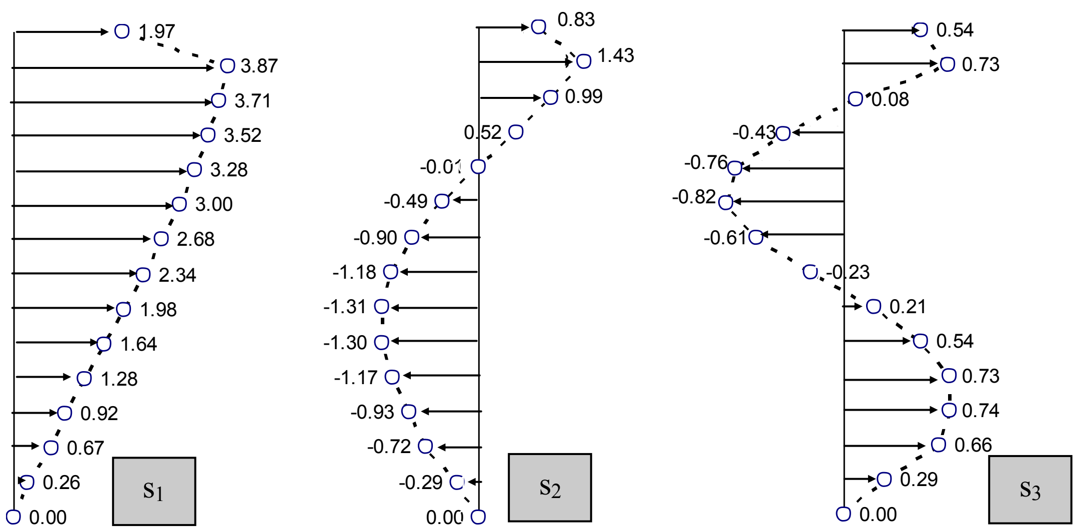


Figure 6. Height-wise distributions (s_n) of invariant modal load vectors ($s_n = mf$).

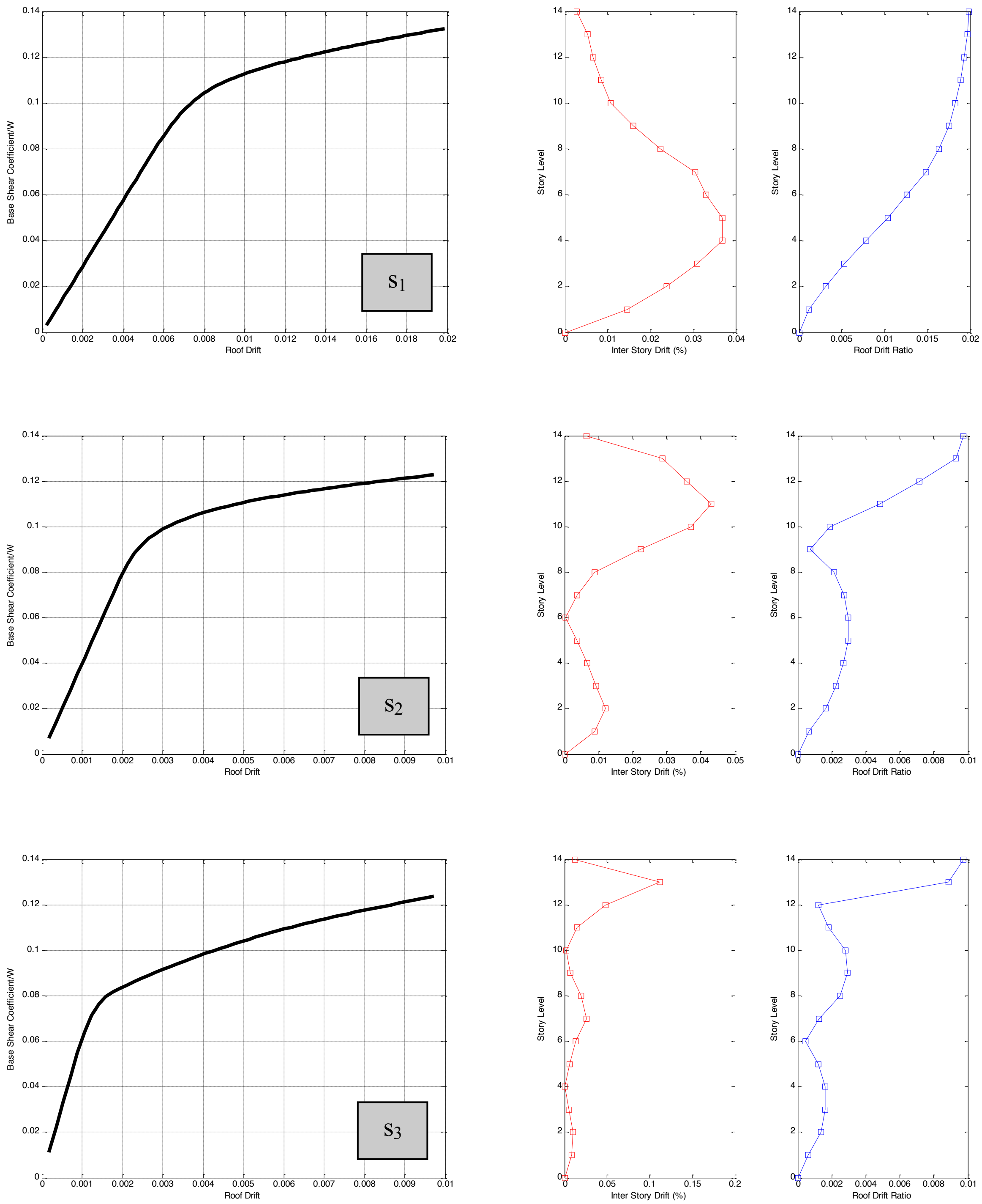


Figure 7. Capacity curve and interstory and roof drift profiles based on separate pushover analyses using invariant load distribution of (a) s1, (b) s2, (c) s3. Note: Target displacement is 2 percent of roof drift for the first mode and 1 percent for the second and third modes.

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