Finite Element Model Calibration of An Instrumented Six-story Steel Moment Frame Building in Burbank, California

By Erol Kalkan

Disclamer

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Acknowledgments

Ground motions were recorded at this 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=CE24370*.

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Introduction

The six-story instrumented building (Figure 1) is located in Burbank, California. The structure designed in 1976 as per the 1973 UBC requirements. The building has been the subject of previous investigations (Kalkan and Kunnath 2006, 2004; Kunnath et al. 2004, Kalkan and Chopra, 2010).

The rectangular plan of the building measures 120 feet by 120 feet (36.6 x 36.6 m) with a 3 ¼ in. (8.3 cm) thick light weight concrete (110 lb /ft^3) slab over 3 in. (7.6 cm) metal decking. Shear studs between the slab and beams were provided on the interior beams in the North-South direction only. The primary lateral load resisting system is a moment frame around the perimeter of the building. The structural system is essentially symmetrical. Moment continuity of each of the perimeter frames is interrupted at the ends where a simple shear connection is used to connect to the weak column axis. The plan view of the building and the elevation of a typical frame are shown in Figure 2. The beam and column sizes of a typical exterior frame are listed in Table 1.

The interior frames of the building were designed as gravity frames and consist of simple shear connections only. All columns are supported by base plates anchored on foundation beams which in turn are supported on a pair of 9.75 m - 0.75 m diameter concrete piles. Section properties were computed for A-36 steel with an assumed yield stress of 303 MPa as established from coupon tests conducted on the steel used in the building (Anderson and Bertero, 1991). The minimum concrete compressive strength at 28 days was 3000 psi, except for slab on grade that was 2000 psi. The total building weight (excluding live loads) was estimated to be approximately 34,644kN.

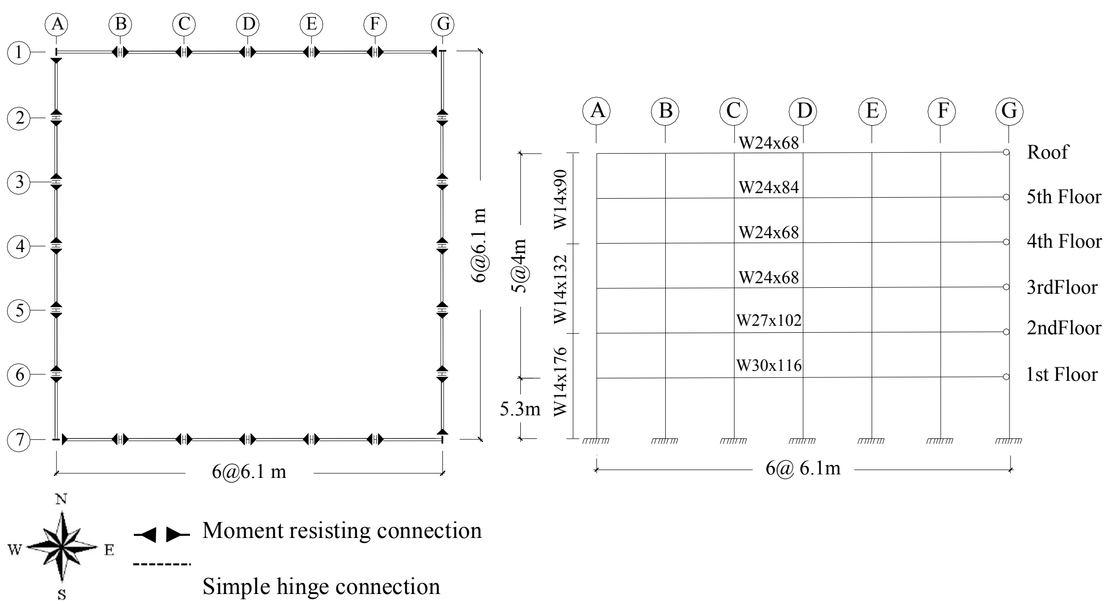
Figure 1. Photo of six-story instrumented building (source: *http://www.strongmotioncenter.org/cgi-bin/CESMD/stationhtml.pl?stationID=CE24567&network=CGS*).

The building has been instrumented with a total of 13 strong motion sensors at the ground, 2nd, 3rd and roof levels as displayed in Figure 3. Instrumentation at the third floor level was not fully functional during the Northridge earthquake.

Table 1. Column and beam sections.

	COLUMNS						
Story	A	B	C	D	E	F	G
1	W14x176	W14x176	W14x176	W14x176	W14x176	W14x176	W14x176
2	W14x176	W14x176	W14x176	W14x176	W14x176	W14x176	W14x176
3	W14x132	W14x132	W14x132	W14x132	W14x132	W14x132	W14x132
4	W14x132	W14x132	W14x132	W14x132	W14x132	W14x132	W14x132
5	W14x90	W14x90	W14x90	W14x90	W14x90	W14x90	W14x90
6	W14x90	W14x90	W14x90	W14x90	W14x90	W14x90	W14x90

	BEAMS					
Story	A-B	B-C	C-D	D-E	E-F	F-G
1	W30x116	W30x116	W30x116	W30x116	W30x116	W30x116
2	W27x102	W27x102	W27x102	W27x102	W27x102	W27x102
3	W24x68	W24x68	W24x68	W24x68	W24x68	W24x68
4	W24x68	W24x68	W24x68	W24x68	W24x68	W24x68
5	W24x84	W24x84	W24x84	W24x84	W24x84	W24x84
6	W24x68	W24x68	W24x68	W24x68	W24x68	W24x68



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

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

OpenSEES Model

Analytical model of the six-story building is created using a typical two-dimensional frame (see line-1 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 3 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 two 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.

Calibration of OpenSEES Model to Observed Response

Recorded response data on this building is available for five earthquakes: 1987 Whittier Narrows, 1991 Sierra Madre, 1992 Landers, 1992 Bigbear and lastly 1994 Northridge earthquakes. The building performed well in all these earthquakes with no visible signs of damage. Recorded data indicates an essentially elastic response in each case (see Table 2 for recorded PGA values in the structure). The analytical model of six-story building was validated using available recorded data from the Northridge Eq. (since it provides the largest recorded PGA in the structure) from different levels of buildings, and a typical comparison of recorded and computed displacement at the roof level of the building is exhibited in Figure 4. Note that the simulation models of the frame used in the evaluation represent the actual state of the building and the corresponding fundamental periods are calibrated to observed response.

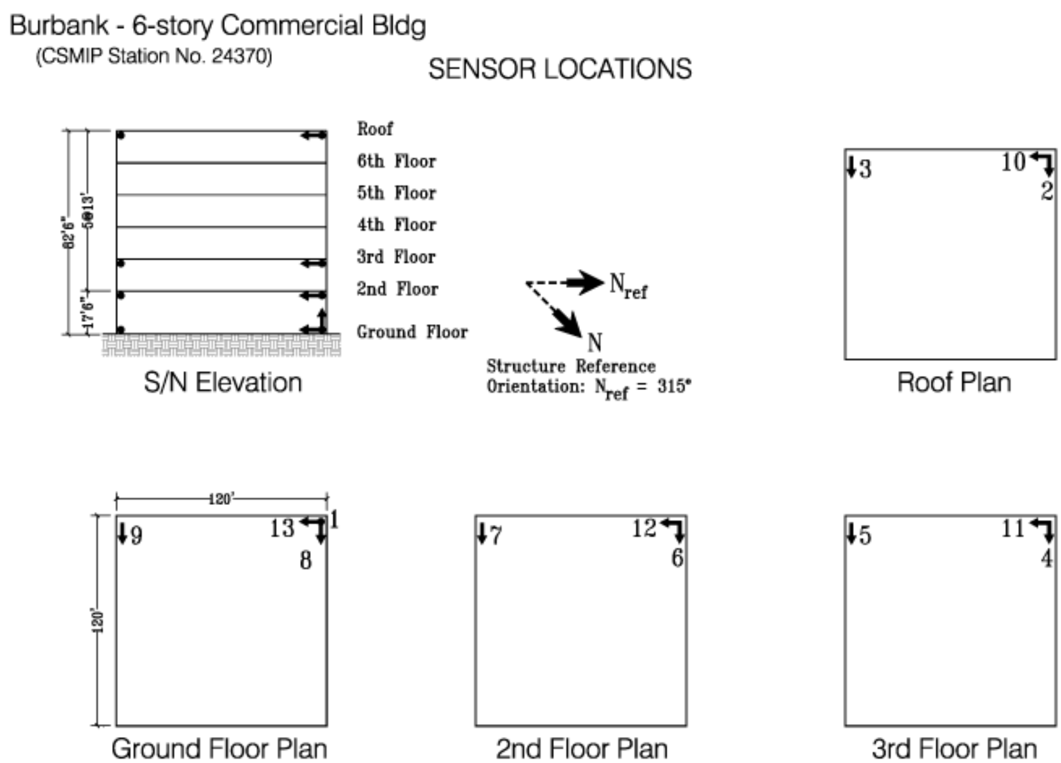


Figure 3. Sensor locations (source: *http://www.strongmotioncenter.org/NCESMD/photos/CGS/bldlayouts/bld24370.pdf*).

For this building, computed total story stiffness and mass values are given in Table 3. These values were used to calculate the elastic modal attributes of the system including modal periods, modal participation factors and modal mass ratios for the first three modes as described in Table 4. The corresponding modal shapes (***f’m f*** =1.0) are portrayed in Figure 5.

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

Earthquake	Magnitude (Mw)	Epicentral Distance (km)	PGA Base Level (g)	PGA Roof Level (g)
1994 Northridge	6.7	22	0.35	0.49
1992 Bigbear	6.5	137	0.04	0.11
1992 Landers	7.3	172	0.05	0.22
1991 Sierra Madre	5.8	30	0.11	0.16
1987 Whittier	6.1	26	0.22	0.30

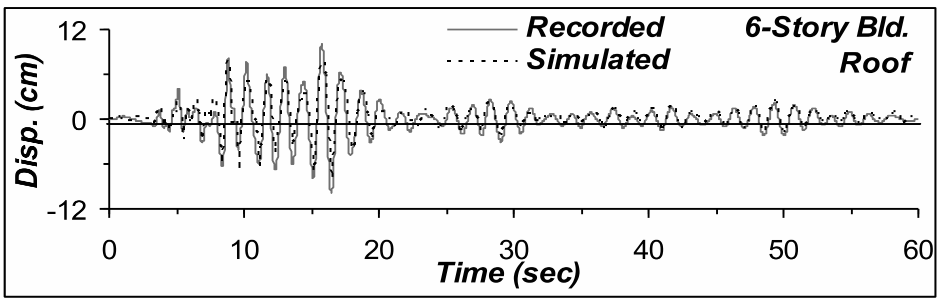


Figure 4. OpenSEES model validation (recorded and computed response at mid and roof levels).

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 six-story building.

Story Level	Story Mass (kip-sec²/in)	Story Stiffness (k/in)	Diagonal of Stiffness Matrix (k/in)
1	1.55	482.5	1659.5
2	1.32	1177.0	2018.5
3	1.32	841.5	1683.0
4	1.32	841.5	1391.0
5	1.32	549.5	1099.0
6	1.04	549.5	549.5

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

6-Story Building	Elastic Modes Mode-1	Mode-2	Mode-3
Modal Periods (sec), Tn	1.39	0.51	0.31
Modal Participation Factors, Γn	2.58	0.96	0.46
Mass Participation Factors, αn	0.85	0.12	0.03

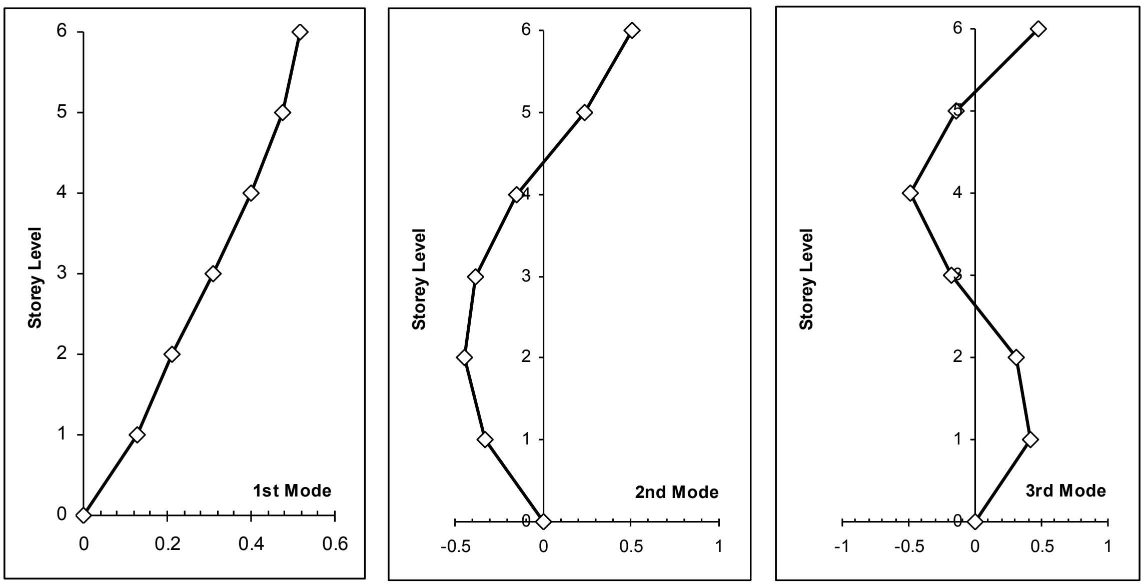


Figure 5. Elastic modal shapes of the first three modes.

Capacity Curves

The modal capacity curves for the six-story building are generated using an invariant load vectors based on individual mode shapes. The invariant load vector is patterned as $\boldsymbol{s_n} = \boldsymbol{mf}$, where $\boldsymbol{m}$ is the modal mass matrix and $\boldsymbol{f}$ is the mode shape of the nth mode. Figure 6 plots the load vectors generated for the first three elastic modes. Using these load vectors, pushover analyses were conducted where the building was first pushed to target displacement level corresponding to 2 percent roof drift using $\boldsymbol{s_1}$. For $\boldsymbol{s_2}$ and $\boldsymbol{s_3}$ loadings, the target displacement level was limited to 0.6 percent roof drift ratio (to satisfy convergence). The resultant modal 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. It is instructive to note that resultant deformed shapes are in strong agreement with the shape of applied load vectors. For that reason, selection of an appropriate load shape for any nonlinear static procedure is the key issue in accurate prediction of the structural response.

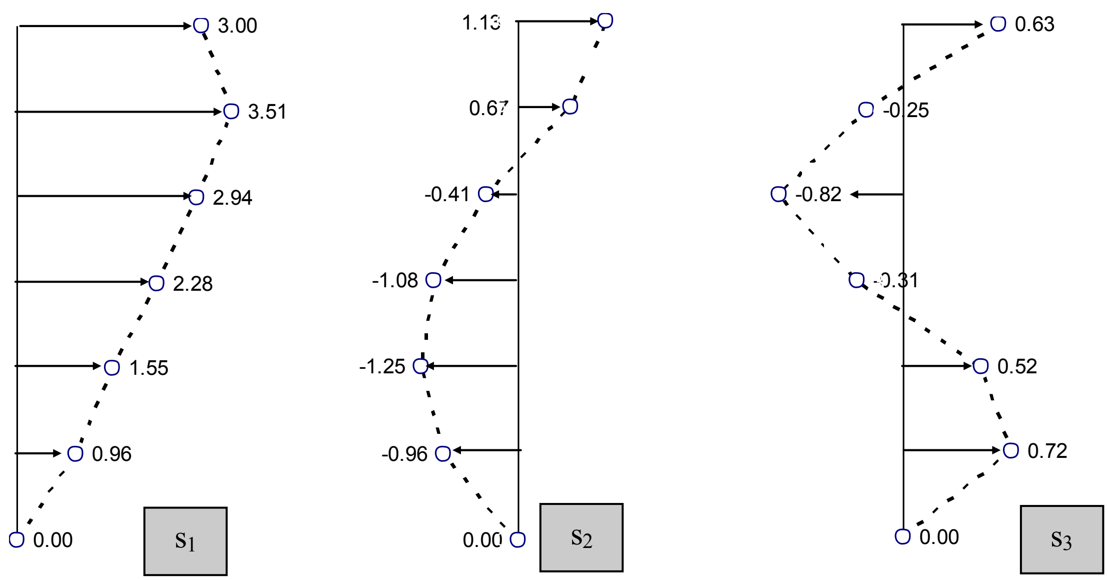


Figure 6. Height-wise distributions (s_n) of invariant modal load vectors ($s_n = mf$).

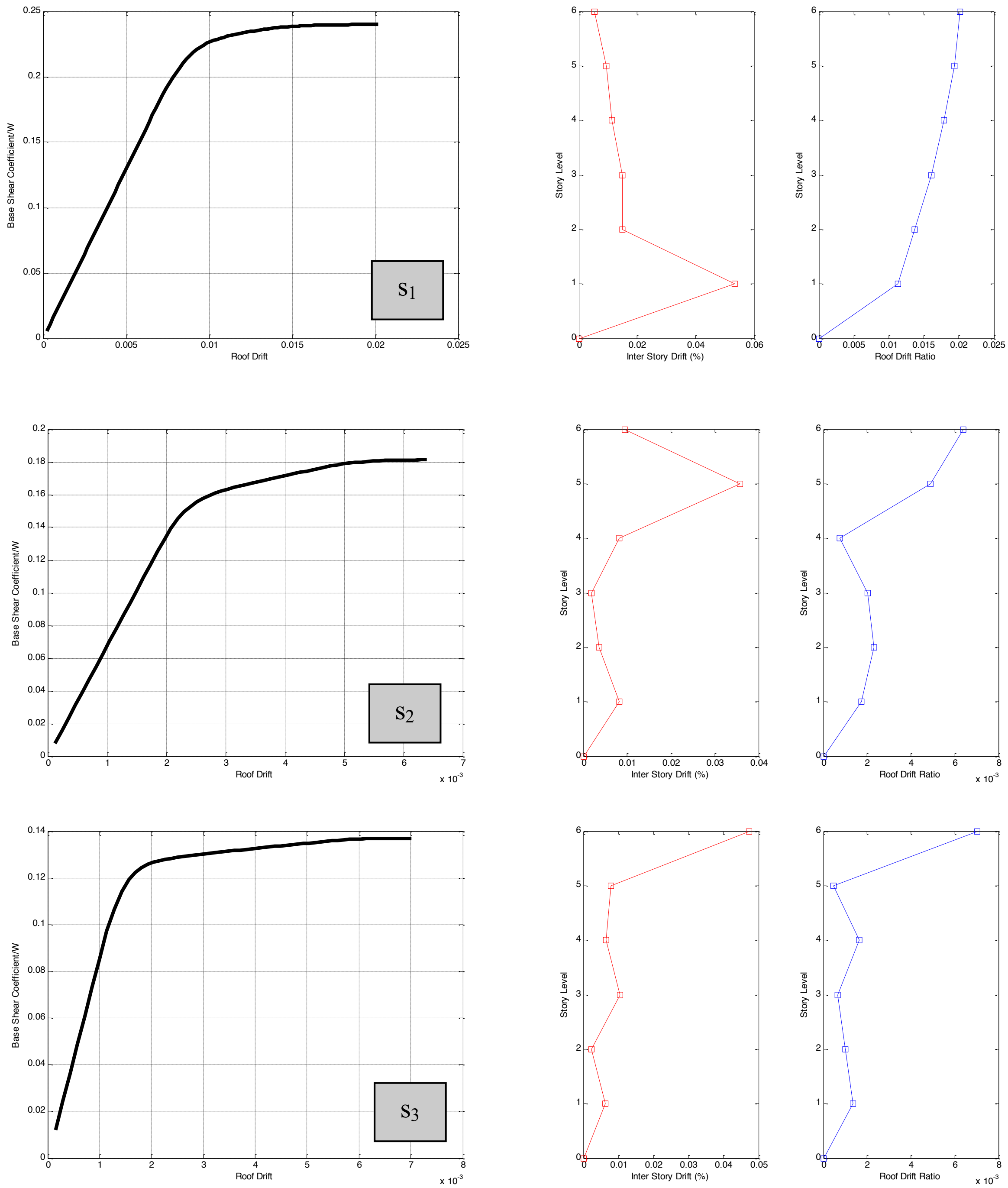


Figure 7. Capacity curve and interstory and roof drift profiles for six-story building based on separate pushover analyses using invariant load distribution of (a) s1, (b) s2 and (c) s3. Target displacement is 2 percent of roof drift for the first mode and 0.6 percent for the second and third modes.

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