CALIBRATING COMPUTER MODELS FOR SEISMIC ANALYSIS: CASE STUDIES USING INSTRUMENTED BUILDING RECORDS

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Abstract

Modern performance-based seismic evaluation of buildings calls for nonlinear analysis of the structural system to estimate seismic demands and assess building performance. The availability of new software with expanded capabilities is gradually making it more feasible to conduct fully nonlinear simulations of building systems. However, at present, there are no readily available guidelines to aid a structural engineer in the process of building an appropriate nonlinear model of the system. As an initial step towards developing such guidelines, the suitability of three widely used computer programs (SAP2000, Perform-3D and OpenSEES) for seismic evaluation of buildings is investigated in this project by utilizing response data recorded from instrumented buildings and comparing the performance of different nonlinear models and methods in terms of their predictive abilities and response sensitivity to modeling choices. Preliminary findings from a preliminary set of simulations on a 9-story steel moment frame building are reported in this paper.

Introduction

The development and application of performance-based seismic design and evaluation of buildings has been hindered by the lack of general guidelines for the practicing engineer regarding the effective use of nonlinear analysis in structural design. There are various nonlinear analysis programs in use today, and an even greater number of modeling choices within and between computer programs. It is essential for engineers to understand the nuances of nonlinear modeling so as to construct a reliable simulation model and analyze its seismic behavior.

Recorded motions from building structures provide engineers and researchers with invaluable data to calibrate simulation models of complex three-dimensional structures. The suitability of existing nonlinear tools for seismic evaluation of buildings is investigated in this project by utilizing response data recorded from instrumented buildings and comparing the performance of different nonlinear models and methods in terms of their predictive abilities. The results presented in this paper represent preliminary findings from the first phase of a more comprehensive study involving several steel frame buildings of varying height.

Case Study: 9-Story Steel Moment Frame Building

The building considered in the evaluation is the Aliso Viejo 9-story office building (CSMIP Station No. 13364). This 9-story office building located in Aliso Viejo, California was designed in 2006 according to the 2001 California Building Code, and constructed in 2008. The building is rectangular in plan with dimensions of approximately 220 ft. x 120 ft. The first floor story height is 17 ft. while the remaining story heights are 13.5 ft. for a total building height of 125 ft. There is a helistop located near the center of the building about 11 ft. above the roof level.

Figure 1: 9-Story office building considered in evaluation (courtesy of CSMIP)

The framing system consists of 3.25" of lightweight concrete over 3" steel deck at the second through the ninth floor levels, and 2.5" of lightweight concrete over 3" steel deck at the roof level. The helistop is 3.5" of normal weight concrete over 3" steel deck. Each level is supported by steel beams and columns. The ASTM designation for the steel beams and columns is A992. Steel columns are supported at ground level by 14" square prestressed precast concrete piles in groups of five or seven piles at each pile cap. The pile caps at the perimeter are tied together by reinforced concrete grade beams, while those at the interior are isolated. Lateral forces are resisted in each direction by steel special moment resisting frames located at the perimeter of the building. The connection used in the moment frames is SSDA's proprietary slotted beam connection. Braced frames resist lateral loading at the helistop level only.

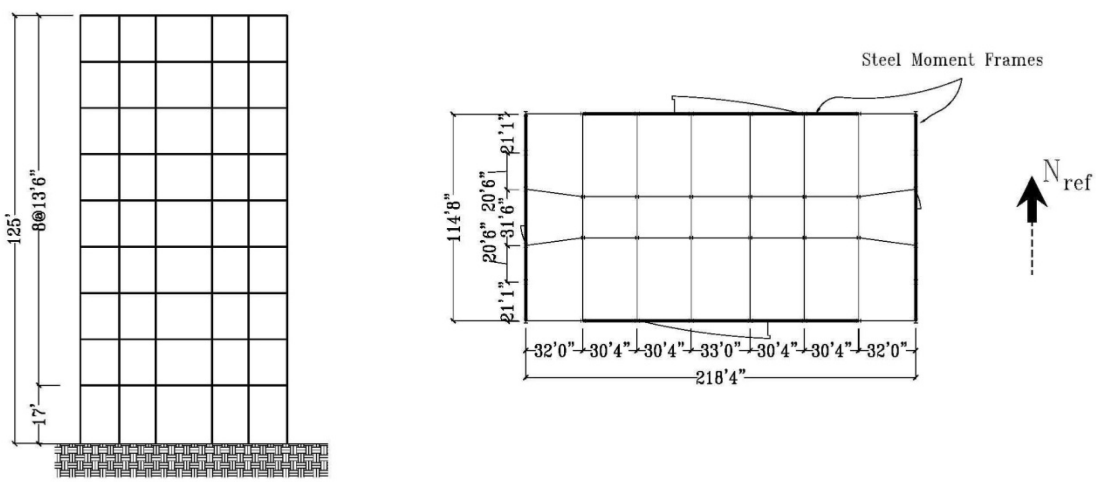

Figure 2: Elevation of typical steel moment frame (N-S Direction) and floor plan of the building

Instrumentation and Recorded Data

This office building was instrumented in 2007 with a total of 15 accelerometers. There are 4 accelerometers located at the ground floor level, 2 accelerometers at the second and fifth floors, 3 accelerometers at the sixth floor and roof, and 1 accelerometer at the ninth floor, as can be seen in Figure 3. The instrumentation of this structure allows for the measurement of the following motions:

1. Ground Floor (foundation): vertical, horizontal in two directions and torsional
2. Second Floor: horizontal in two directions
3. Fifth Floor: horizontal in two directions
4. Sixth Floor: horizontal in two directions and torsional
5. Ninth Floor: horizontal in one direction
6. Roof: horizontal in two directions and torsional

This station has recorded data from two earthquakes: the Chino Hills earthquake of 2008 with a PGA of 0.026g and the Laguna Niguel earthquake of 2012 with a PGA of 0.029g. The noted PGAs are based on the recorded motion at the base of the building.

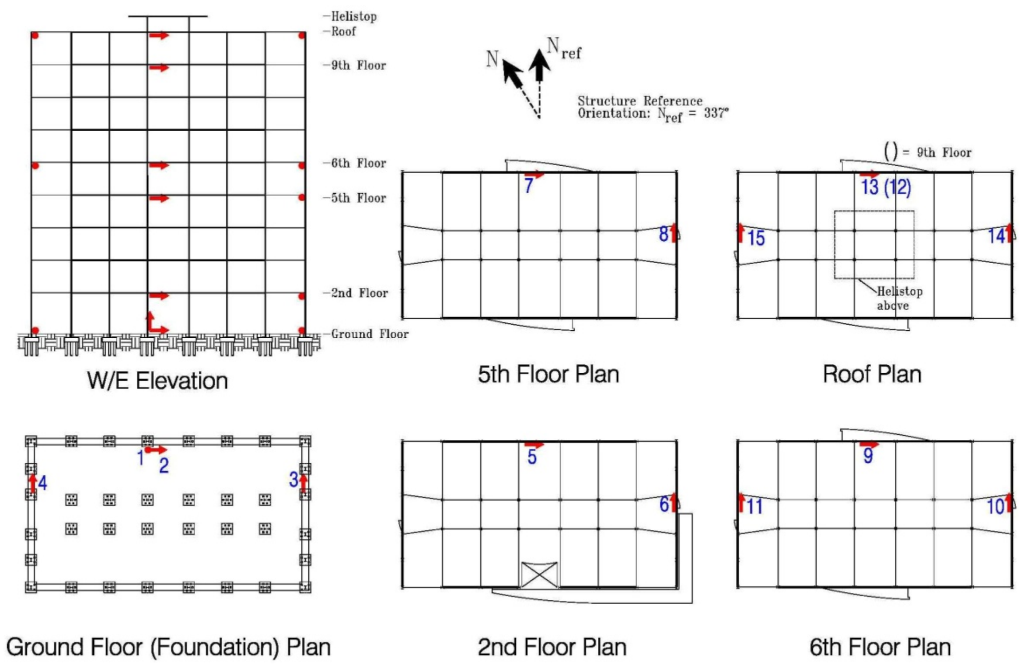

Figure 3: Layout of accelerometers in the building (courtesy of CSMIP)

System Identification Studies

The acceleration time histories recorded during the Chino Hills event were used to generate Fourier amplitude spectra for each instrumented level. At the Ground Floor, Sixth Floor, and Roof levels the average of the transverse accelerometers was used in the generation of the spectra. In this way the torsional modes of vibration were suppressed. The Fourier amplitude spectra were then used to develop the transfer functions which can be seen in Figure 4.

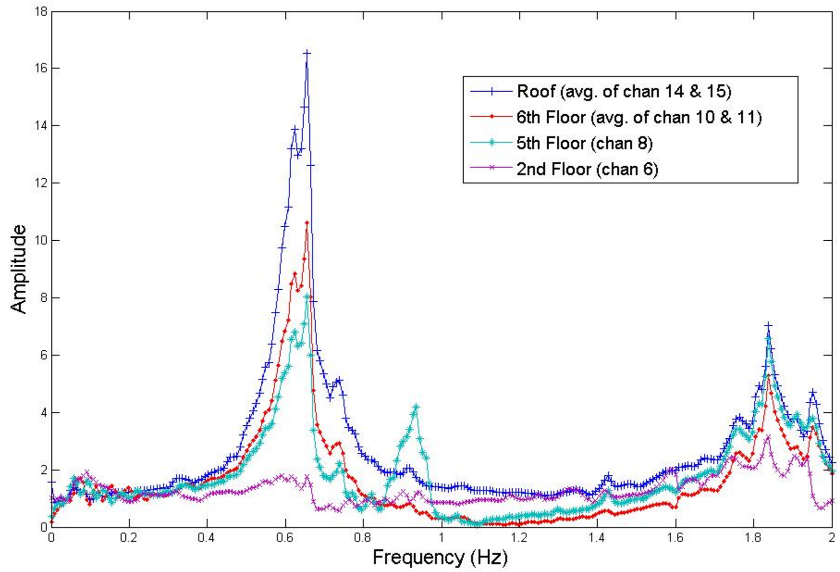

Figure 4: Transfer functions at instrumented levels of the building

The peak marking the first lateral mode of vibration can be seen at approximately 0.63 Hz, corresponding to a fundamental period of vibration of 1.59 seconds. Likewise, the second mode of lateral vibration can be seen at approximately 1.84 Hz, corresponding to a period of 0.54 seconds. In the Fifth Floor transfer function a peak can be seen at approximately 0.94 Hz. This peak corresponds to the first torsional mode of vibration. A similar peak would be seen in the Sixth Floor and Roof level transfer functions if the torsional response had not been suppressed by taking the average of the transverse channels.

Simulation Model of Building and Calibration

Two-dimensional linear models of the building were developed using the following software: SAP2000, Perform-3D, and OpenSEES. A three-dimensional linear model was developed using only the SAP2000 software. The two-dimensional models represent framing for the north-south reference direction of the structure. The details of the development of the models and relevant assumptions are summarized below:

- Centerline dimensions were used (i.e. panel zone were not modeled explicitly)
- All frame elements and connections are linear elastic
- Diaphragms were assumed to be rigid in plane
- Columns were assumed to be fixed at the base
- Moment connections were modeled as rigid, while gravity frame shear tab connections were modeled as partially rigid with rotational stiffness proportional to beam bolt group depth as outlined in Liu and Astaneh-Asl (2000).
- The stiffness resulting from composite action between the beam-slab system was included in the model. A composite moment of inertia was calculated based on the cross-sectional properties of the beam and slab and the moment of inertia of the beam was modified to reflect the increased stiffness. The composite moment of inertia was determined to be approximately three times that of the moment of inertia of the beam.

Only the moment frame beams were modified in this way; the stiffness of the gravity frame beams was left unaltered.

- The mass assigned at each level was estimated based upon the drawings and Table C3-1 (Minimum Design Dead Loads) of ASCE7-05. As there are two identical moment frames at the perimeter in the north-south direction of the building, only one-half of the total mass at each level was assumed to be tributary to the frame model. The seismic mass at the roof level was comprised simply of dead load, whereas at the floor levels additional mass was included to reflect the presence of partitions and some live load.
- The helistop framing was not explicitly modeled but the associated mass was assigned to the roof level nodes.

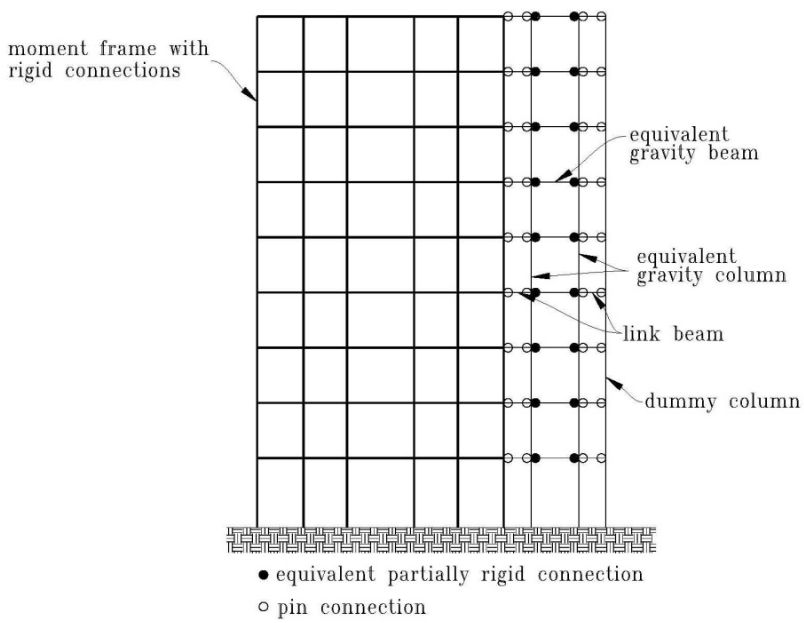

Figure 5: Elevation of two-dimensional model of building with equivalent gravity framing and dummy columns

- An equivalent gravity frame was included in the models in order to capture the stiffness contribution of the gravity framing, as well as to be able to account for any lateral force induced by P-delta effects within the gravity frame system. The cross-sectional area, shear area, and bending stiffness (EI/L) of the modeled gravity columns was made equivalent to that of one-half of the total of the gravity columns at each level. The shear area and bending stiffness of the gravity beams was modeled in a similar manner. The rotational stiffness of the modeled partially rigid beam-to-column connections was made proportional to one-half of the combined rotational stiffness of the gravity frame shear tab connections at each level. The equivalent gravity frame is tied to the moment frame by rigid links with pinned ends at each level.
- Dummy columns were included to account for the additional stiffness required at each level in order to calibrate the model, which additional stiffness represents the combined stiffening effect of those elements of the building not explicitly included in the model (e.g. non-structural components, partition walls, etc.). In order to adjust the stiffness of the dummy columns the moment of inertia of the elements was simply increased or

decreased as required. The dummy columns are tied to the gravity frame by rigid links with pinned ends at each level.

- Gravity loads were applied to the moment frame at each level based upon tributary area of the dead and live load estimates. The gravity loads applied at each level to the equivalent gravity columns are proportional to one-half of the total gravity load minus that which is tributary to the moment frame at each level.
- The models were calibrated to the motions recorded during the Chino Hills earthquake. In order to best match the acceleration and displacement amplitudes the damping was set at 5% of critical for all modes in SAP2000 and Perform-3D. In OpenSEES, Rayleigh damping was used with 5% damping assigned to modes 1 and 3.

Model Validation

The first and second modal periods resulting from the two-dimensional models in SAP2000, Perform-3D, and OpenSEES are shown in Table 2 below. The table also shows the effect of the inclusion of the gravity frame and dummy columns on the modal periods. It can be seen that a significant stiffness contribution was required of the dummy columns in order to lower the periods to the approximate 1.59 seconds for T_1 and 0.54 seconds for T_2, which were estimated from the transfer functions as described previously.

Table 2: Comparison of modal periods from different computer programs

	Moment Frame Only		Moment Frame + Gravity Frame		Moment Frame + Gravity Frame + Dummy Columns	
	T_1	T_2	T_1	T_2	T_1	T_2
SAP2000	2.08	0.72	2.00	0.69	1.58	0.50
Perform-3D	2.08	0.72	2.00	0.69	1.58	0.50
OpenSEES	2.08	0.72	1.97	0.69	1.56	0.50

Note: All period values are shown in seconds

Using the ratio of the amplitude of the transfer function at each level with the amplitude of the transfer function at the Roof level (at 0.63 Hz and 1.84 Hz), mode shapes can be estimated for the first and second modes of lateral vibration. The estimated mode shapes using the three computer programs is displayed in Figure 6. It can be seen from the figure that the mode shapes computed by each of the three programs match one another almost identically, and that the match to the estimated shapes is very close.

The average of the acceleration time histories recorded in the transverse direction during the Chino Hills earthquake at the Ground Floor level of the building was used as input motion for response history analyses of the two-dimensional linear models constructed in SAP2000, Perform-3D, and OpenSEES. A comparison of the computed acceleration time history response of each model with the actual acceleration response of the building to the Chino Hills earthquake can be seen in Figures 7-9.

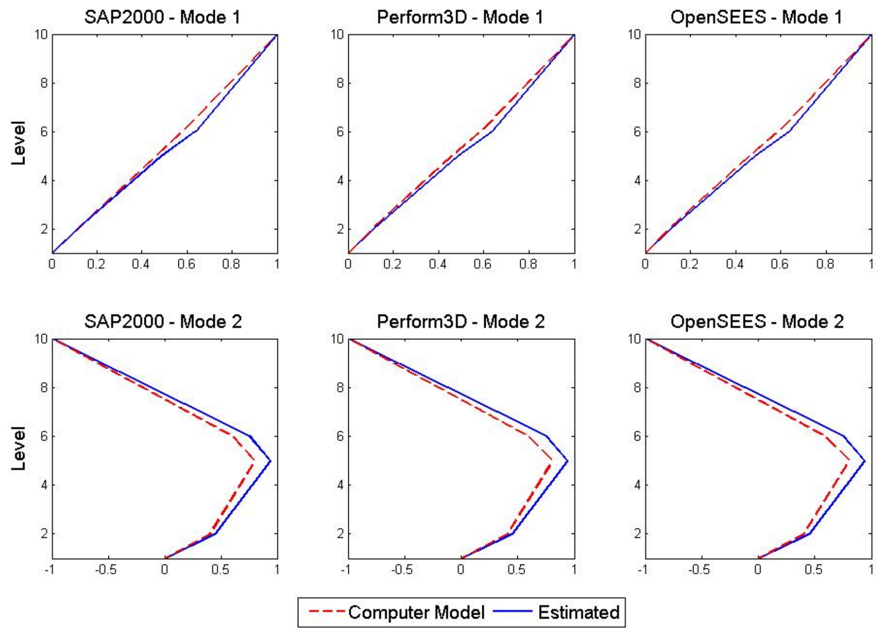

Figure 6: Comparison of computed vs. estimated shapes of first and second modes of vibration of the Aliso Viejo 9-story office building

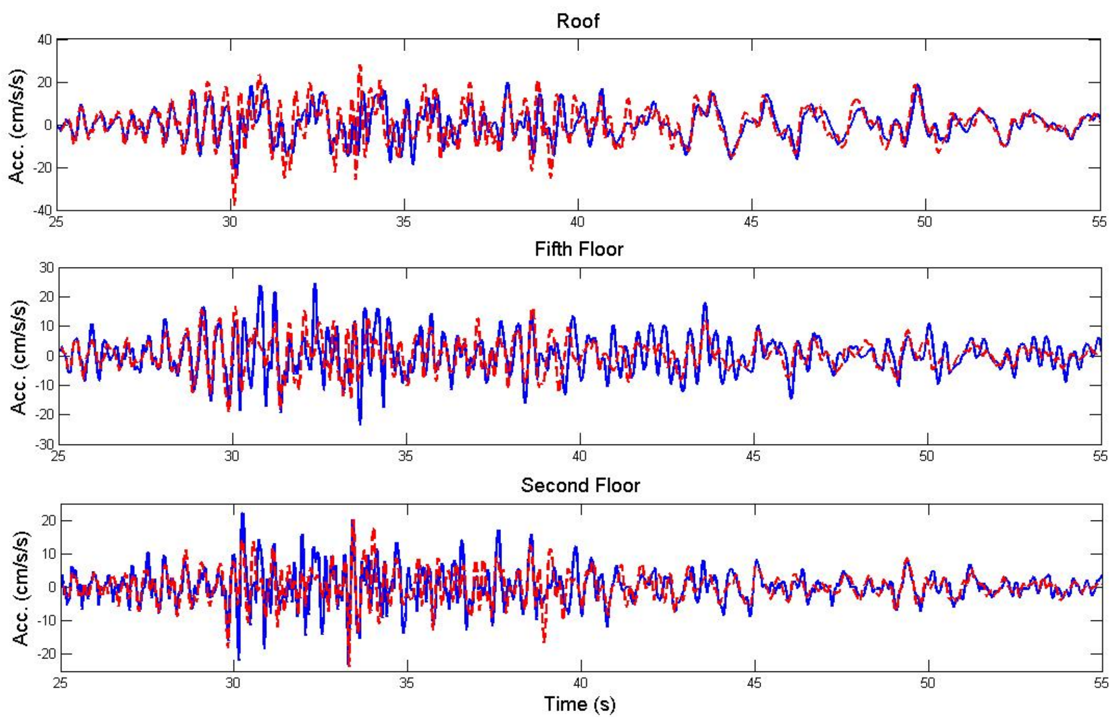

Figure 7: Comparison of computed (SAP2000) vs. actual acceleration time histories at selected instrumented levels of the building

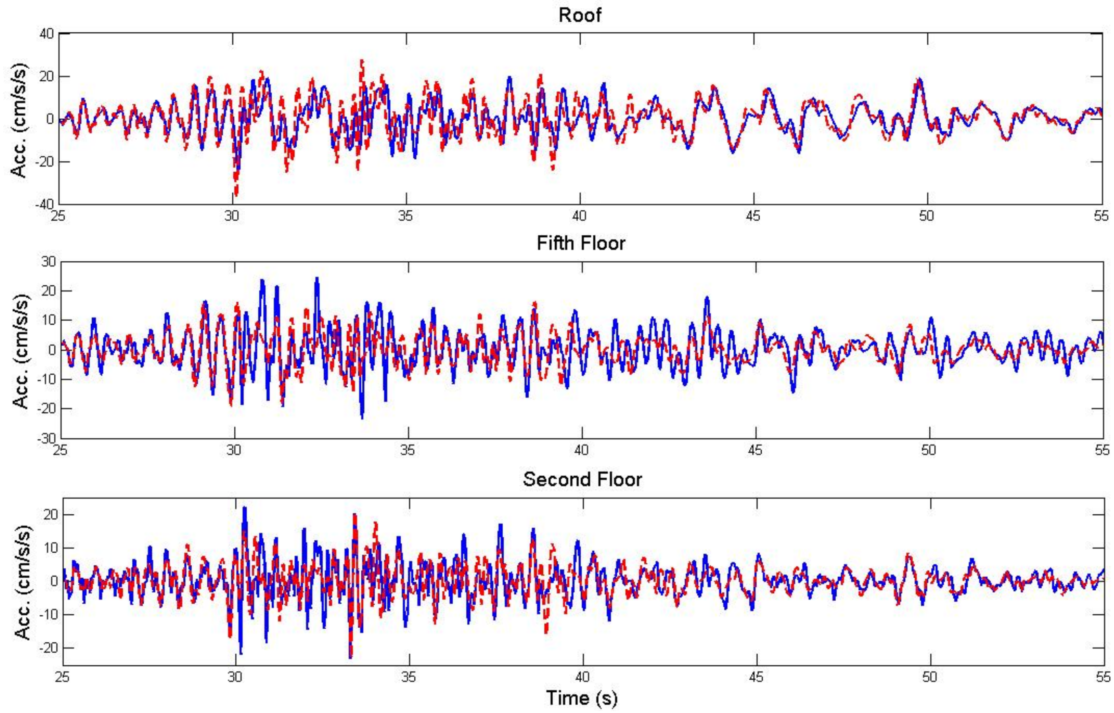

Figure 8: Comparison of computed (Perform-3D) vs. actual acceleration time histories at selected instrumented levels of the building

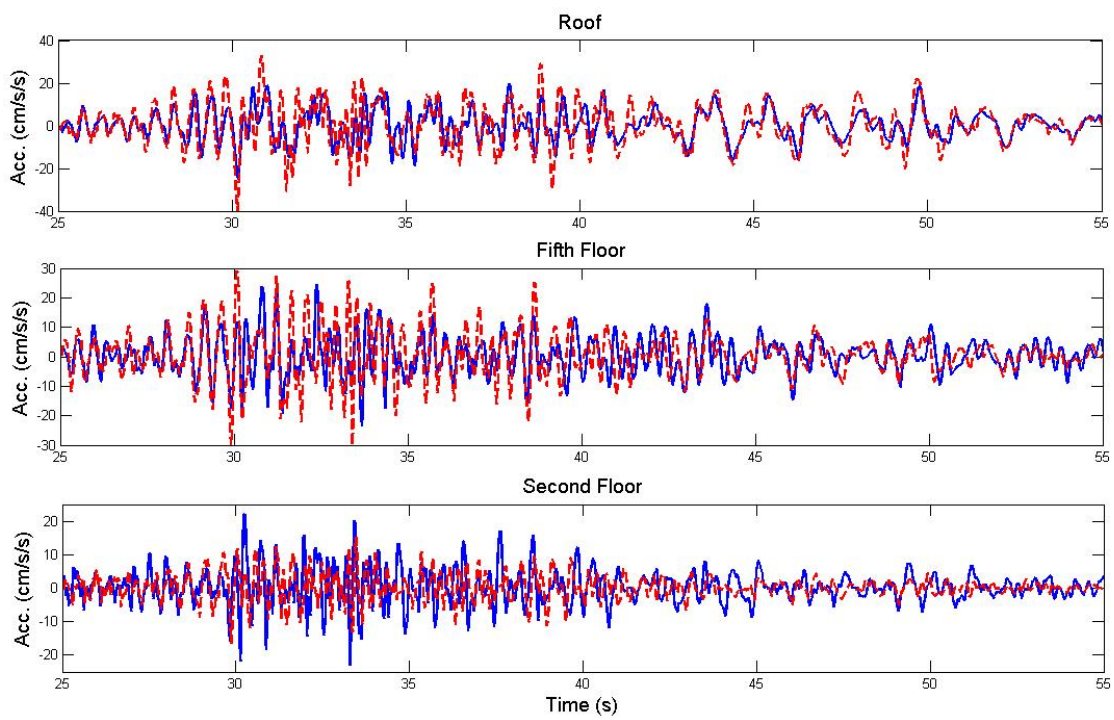

Figure 9: Comparison of computed (OpenSEES) vs. actual acceleration time histories at selected instrumented levels of the building

A comparison of the computed relative displacement time history response of each model with the computed relative displacement response of the building to the Chino Hills earthquake can be seen in Figures 10-12.

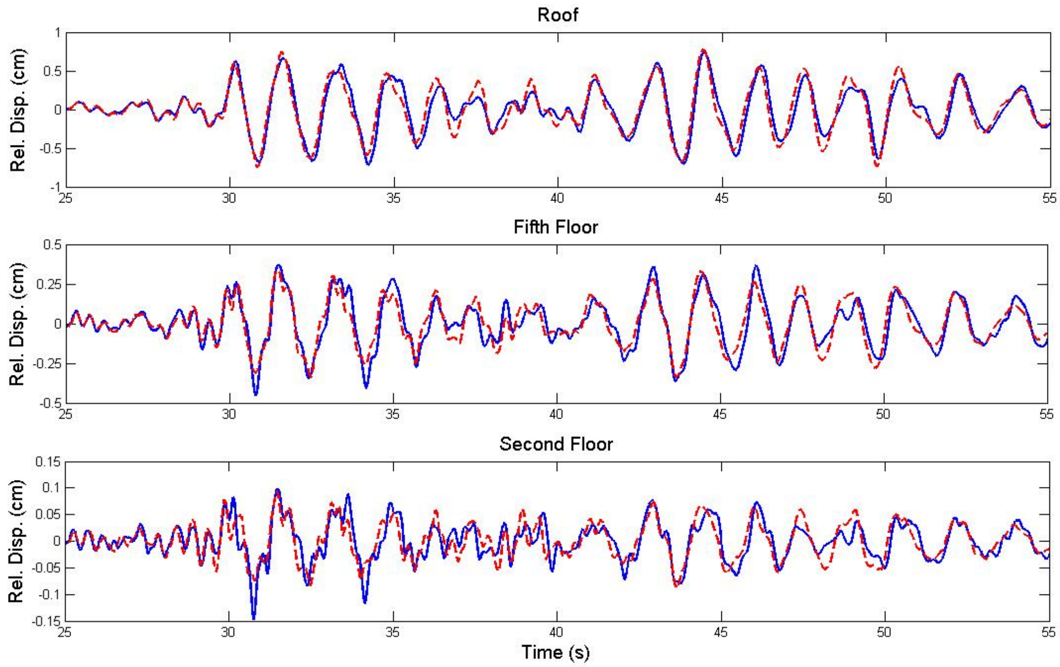

Figure 10: Comparison of computed (SAP2000) vs. recorded relative displacement time histories at selected instrumented levels of the building

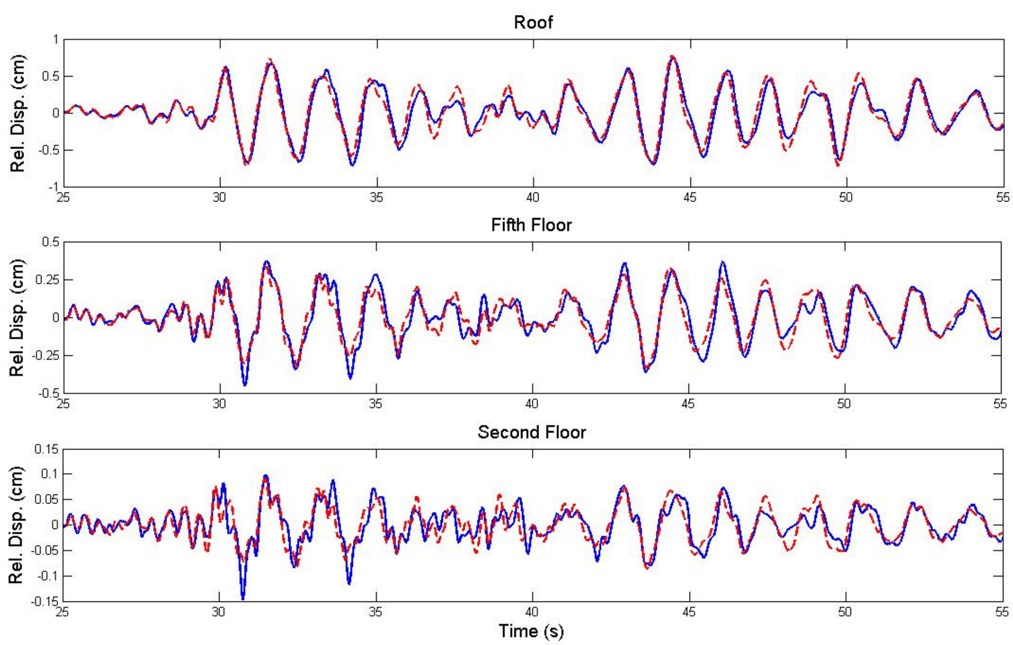

Figure 11: Comparison of computed (Perform-3D) vs. recorded relative displacement time histories at selected instrumented levels of the building

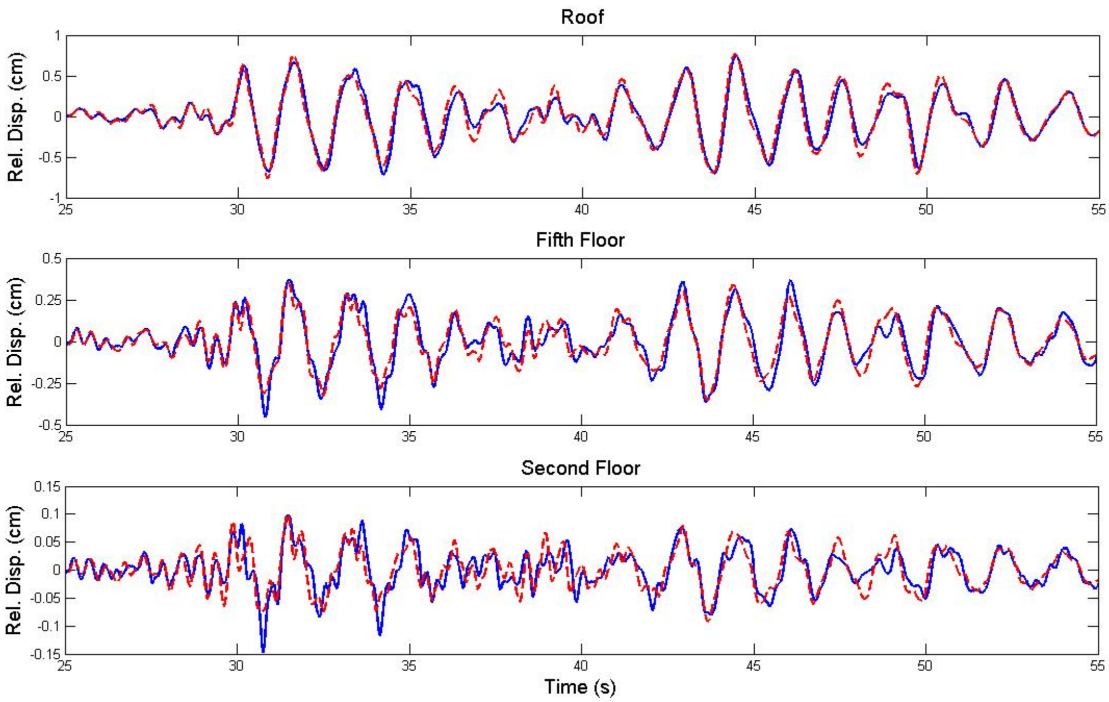

Figure 12: Comparison of computed (OpenSEES) vs. recorded relative displacement time histories at selected instrumented levels of the building

At the Roof, the average of the actual response recorded in the transverse direction was used for comparison in all cases. Overall, based on the assumptions previously noted, the computed roof accelerations compare well with observed responses for all three computer programs. The predictions of accelerations at the 5th and 2nd level are generally not as good as the estimates at the roof. In the case of displacements (relative to the ground), the predicted responses are quite good for both the roof and the 5th floor level. Some discrepancies are obvious in the computed responses at the 2nd floor level.

A direct comparison of the computed acceleration and relative displacement time history responses at the roof and second floor levels of each model to the Chino Hills earthquake can be seen in Figure 13. The relative displacement time history responses of the three different models match almost exactly. The SAP2000 and Perform-3D models match almost identically in acceleration as well, while the model developed in OpenSEES varies slightly from the other models in its acceleration response.

Nonlinear Sensitivity Analysis

Four different nonlinear models were generated from the elastic models in SAP2000, Perform-3D and OpenSEES. In SAP2000, concentrated hinges located at moment frame beam and column ends were used. The moment-rotation relationship of the hinges was assumed to be bilinear with 3% post-yield stiffness in one case, and elastic-perfectly plastic in another. The nonlinear model generated in Perform-3D used fiber hinges located at moment frame beam and column ends. The hinge length was assumed to be one-half of the member depth, and the stress-strain relationship assigned to each steel fiber was assumed to be elastic-perfectly plastic.

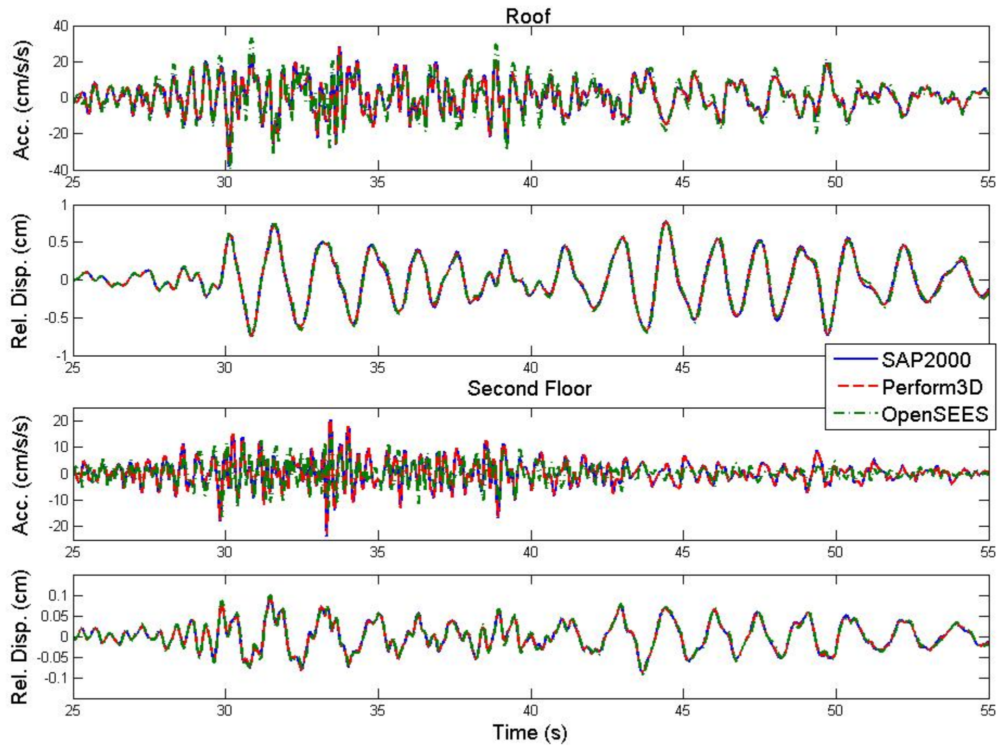

Figure 13: Comparison of computed acceleration and relative displacement time histories at roof and second floor levels using different computer programs.

In OpenSEES, two different nonlinear models were generated. The first used distributed plasticity elements for each moment frame beam and column. Five integration points were used for each distributed plasticity element. The second OpenSEES model used fiber hinges located at moment frame beam and column ends. Three different hinge lengths were assumed: one-half of the member depth, three-quarters of the member depth, and the full member depth. The stress-strain relationship assigned to each steel fiber in both models was assumed to be elastic-perfectly plastic.

For each of the four nonlinear models the expected yield stress of the steel wide flange framing (55 ksi) was used instead of the design yield stress (50 ksi) for establishing the associated strengths of the force-deformation or stress-strain relationships. In the equivalent gravity frames for each of these models, moment-rotation hinges were used at each end of the gravity beams with an assumed elastic-perfectly plastic force-deformation relationship. The plastic moment capacities for these partially-rigid connections were determined as outlined in Foutch and Yun (2002). Also, the dummy columns were not included in these models for the nonlinear response history analyses.

Figure 14 compares the inter-story drift ratios resulting from the nonlinear response history analyses which were performed by scaling the original ground motions by a factor of 10 to induce inelastic behavior in the building. The results from the SAP2000 model diverged quite significantly from the other programs, which were all based on models using elements with fiber sections. The results from the Perform-3D and OpenSEES models varied from one another, but only slightly.

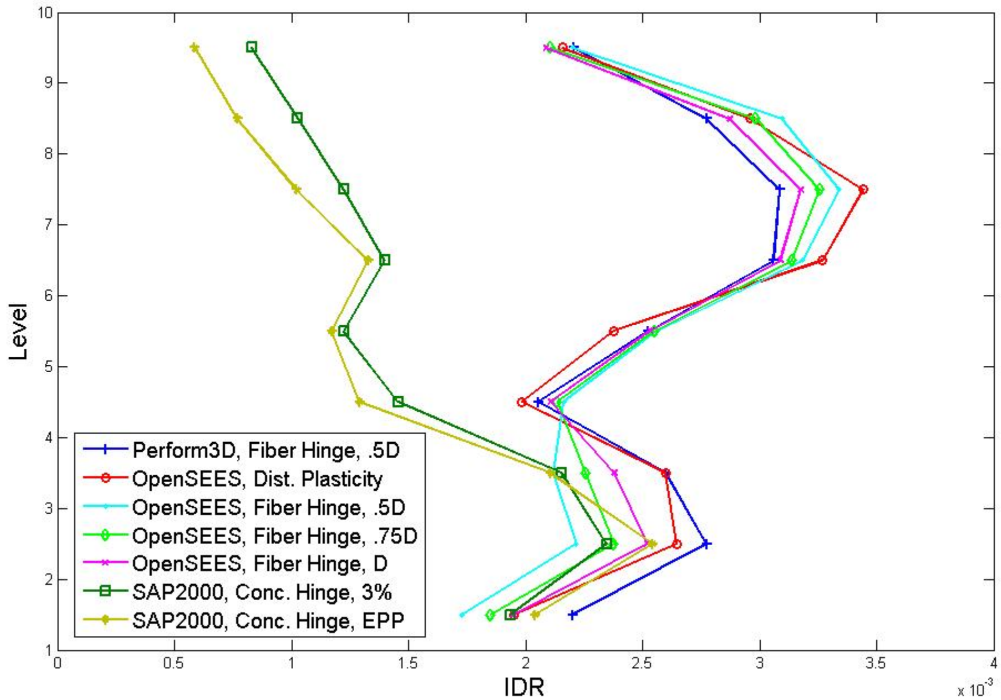

Figure 14: Comparison of computed peak inter-story drift ratios using the three computer programs and different nonlinear modeling assumptions

Concluding Remarks

For the case of purely elastic behavior, all three computer programs, under generally similar modeling assumptions, produce comparable results for the displacement response of the building compared to the actual recorded response. Some discrepancies in the acceleration response at the lower levels of the building are evident even at these low levels of ground shaking. At increased ground shaking intensities (achieved in this study by scaling the original recorded motion), the results from SAP2000 are seen to deviate from OpenSEES and Perform-3D given the modeling options used in the study. Further investigation is needed to characterize the noted differences in the three computer programs due to inherent modeling assumptions.

References

Foutch, D.A. and Yun, S. (2002). “Modeling of steel moment frames for seismic loads,” *Journal of Constructional Steel Research*, 58 (2002), 529-564.

Liu, J. and Astaneh-Asl, A. (2000). “Cyclic Tests on Simple Connections Including Effects of the Slab,” *Report SAC/BD-00/03*, SAC Joint Venture.