VALIDATION OF EVALUATION METHODS AND ACCEPTANCE CRITERIA IN EVOLVING PERFORMANCE-BASED SEISMIC CODES

Sashi K. Kunnath, Quan Nghiem, Alfred John Jr. and Sherif El-Tawil

Department of Civil and Environmental Engineering
University of Central Florida, Orlando

ABSTRACT

A critical evaluation of the four analytical methods recommended in FEMA-273 for the estimation of seismic demands is carried out using measured response characteristics of four instrumented steel buildings. Prior to conducting the FEMA-273 analyses, computer models of each structural system were calibrated to the observed response. Two of the buildings which experienced little or no damage were modeled as fully three-dimensional systems while the remaining two buildings which suffered moderate damage in recent seismic events were further tuned to simplified two-dimensional models to permit detailed inelastic evaluations. Dozens of linear and nonlinear computer analyses under both static and seismic loads were carried out on each building, however, only a few typical results are presented in this paper. Results of the evaluations provide insight into both modeling issues and the validity of the four analytical procedures outlined in the FEMA-273 document. It is found that calibrating structural models to observed response is sensitive to mass and stiffness modeling assumptions. Linear and nonlinear static procedures do not adequately predict inter-story drift estimates, a critical parameter in seismic evaluation and design.

INTRODUCTION

The design philosophy advocated in FEMA-273 (1997) is roughly composed into three steps: definition of a performance objective which incorporates a seismic hazard level, estimation of seismic demands in the system and its components, and verification of acceptance criteria which determines if the design objective has been met. Of these, the second step can be considered the most vital since an accurate estimate of expected performance is essential to assessing the suitability of the final design. Two types of analysis methods, which can be broadly classified as linear and nonlinear procedures, are outlined in the FEMA document. This implies that an engineer can make reliable estimates of deformation demands using either one of these analysis methods. The estimation of seismic demands using linear or nonlinear *static* procedures are inevitably going to be favored by practicing engineers over nonlinear time-history methods because of the complexity and uncertainty involved in material modeling and identification of appropriate ground motion characteristics for fully nonlinear dynamic procedures. The four FEMA analytical approaches evaluated in this study are:

Linear Static and Dynamic Procedures (LSP and LDP): This procedure is recommended for regular buildings with heights not exceeding 100 ft. Additionally, the demand to capacity ratio for any element in the structure should typically not exceed 2.0. Certain exceptions to the rule are outlines in Section 2.91 of FEMA-273.

Nonlinear Static and Dynamic Procedures (NSP and NDP): Nonlinear procedures are generally applicable for all buildings with the exception that NSP is limited to buildings where high mode effects are small. Again, FEMA-273 has specific guidelines using response spectrum methods to determine the limits of NSP. It is also suggested that two lateral load patterns be considered in the analysis: an inverted triangular pattern (referred to as NSP-1 in this paper) based on Equation (3-7) in FEMA-273, and a uniform distribution (referred to as NSP-2).

One of the primary aims of this project is to evaluate the four analytical methods. The ability of each analytical procedure to predict inter-story drift demands will be examined. In the case of damaged buildings, the ability of the nonlinear static method to identify critical connection fracture locations in the structure will be verified. A secondary objective of the project deals with system identification and issues related to calibrating building models to observed response.

Four steel buildings, all instrumented by the California Strong Motion Instrumentation Program (CSMIP), were considered in this evaluation exercise. The recorded data was made available by CSMIP and details of the instrumented information are available in numerous publications (Darragh et al., 1995; Shakal et al, 1995 etc.). Building data, the system identification tasks leading to the calibration of the building models followed by the FEMA-273 evaluations of the buildings are summarized in this paper.

It is essential to point out that each building was analyzed using different building models and numerous loading scenarios. In all, over a hundred simulations were carried out including linear and nonlinear evaluations of static and seismic loading. The results presented in this paper cover only a typical subset of evaluations.

SIX-STORY COMMERCIAL BUILDING, BURBANK

This is the first of four buildings evaluated in this study. Recorded response data on this building is available for three earthquakes: the 1987 Whittier Narrows earthquake, the 1991 Sierra Madre earthquake and the 1994 Northridge earthquake. Studies on this building comprised of system identification studies to calibrate the building model and FEMA-273 evaluation using the four different analytical approaches.

Building Details

This building is a 6-story steel structure designed in 1976 as per the 1973 UBC requirements. 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 is shown in Figure 1. The interior frames were designed as gravity frames and consist of simple shear connections only. Exterior columns are supported on piles while the interior columns are supported on spread footings. The building was instrumented with a total of 13 strong motion sensors at the ground, 2nd, 3rd and roof levels, as displayed in Figure 2. Instrumentation at the third floor level was not fully functional during the Northridge earthquake and was not available for this study.

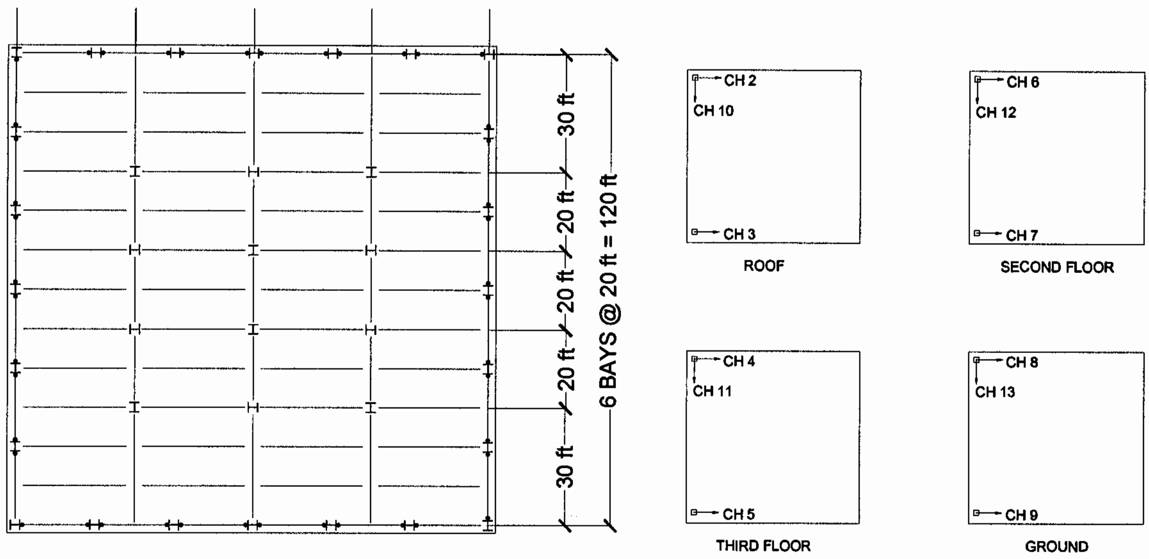


Figure 1. Plan View of Building and Location of Instrumentation

The building performed well in all three earthquakes with no visible signs of damage. Recorded data indicates an essentially elastic response. In constructing the building model, the base was assumed to be fixed (all columns are supported by base plates anchored on foundation beams which in turn are supported on a pair of 32 feet 30-inch diameter concrete piles). Section properties were computed for A-36 steel with an assumed yield stress of 44 ksi as established from coupon tests conducted on the steel used in the building (Anderson and Bertero, 1991). The total building weight (excluding live loads) was estimated to be approximately 7600 kips.

Calibration of Observed Response

A three-dimensional computer model of the building was created using SAP2000 (Computers and Structures, 2000). The model included both the lateral load resisting perimeter frames and the interior gravity frames. An isometric view of the SAP2000 model is displayed in Figure 2 along with an elevation of a typical frame showing member sizes and dimensions. Several modeling assumptions were tested: 1) beam models with and without full composite action; 2) increased damping in higher modes; 3) effect of panel zone yielding.

An earlier study by Shen and Astaneh (1990) on the same building indicated that proper modeling of the floor diaphragm was crucial in reproducing observed response. It is important to point out that their evaluation was based on the building response to the 1987 Whittier earthquake which was smaller in magnitude than the Northridge event. Given the low magnitude response, it is likely that both composite floor action and the participation of non-structural elements were significant. To gain a better understanding of the system characteristics during the three different earthquakes, a spectral analysis of the roof vs. ground accelerations was carried out. Figure 3 shows the transfer function of the roof accelerations for the three recorded ground motions.

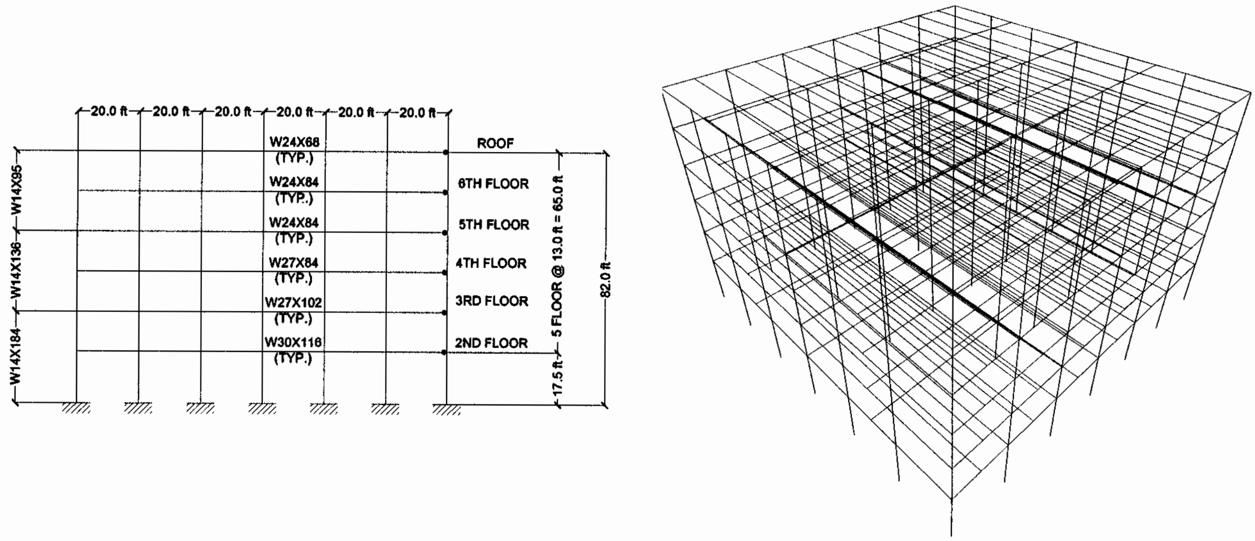


Figure 2. Elevation of Typical Perimeter Frame and SAP2000 Model of Building

It is evident from Figure 3 that the system response is a function of both the ground motion spectral characteristics and the ground motion intensity. The predominant frequency for the relatively low magnitude events (Whittier and Sierra Madre) is 0.78 Hz (1.28 sec) and for Northridge is 0.71 Hz (1.41 sec).

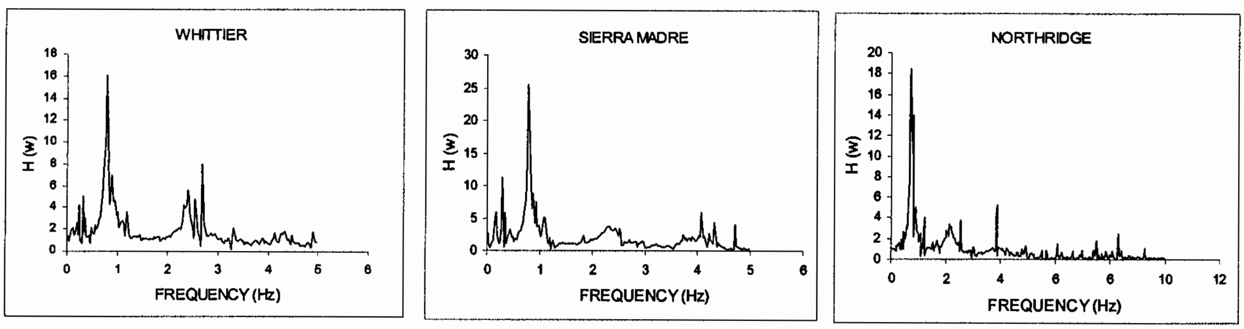
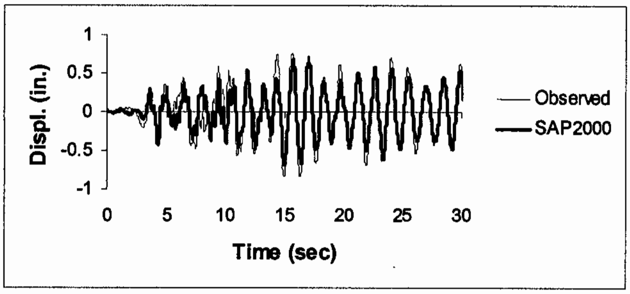
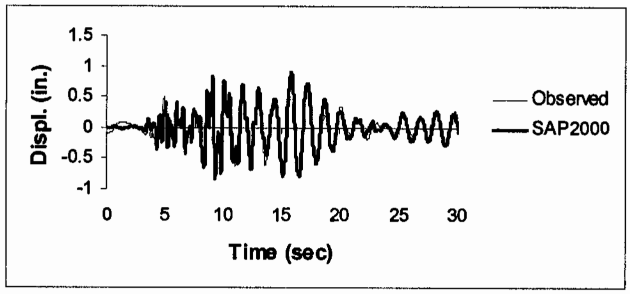


Figure 3. Transfer Function of Observed Roof Acceleration During Whittier, Sierra Madre and Northridge Earthquakes

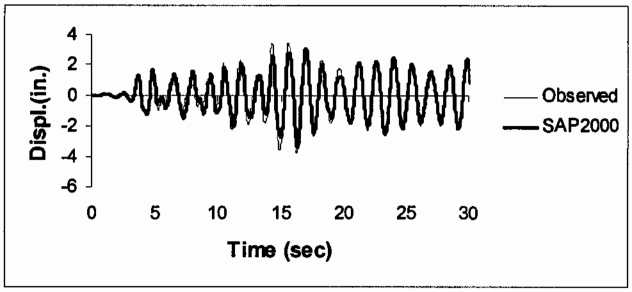
Several aspects of modeling the 6-story Burbank building were explored. Three separate models were developed as part of the calibration exercises: (i) a fully three-dimensional model including internal gravity frames; (ii) a 3D model considering the exterior frames only; and (iii) a 2D model of a typical exterior frame. Results of separate 3D lateral analysis indicated that the perimeter frames carry about 85% of the lateral load. Consequently, 2D models in which only 42.5% of the mass was considered produced the best correlations with observed response. In all cases, it was established that the bare frame stiffness was inadequate to represent the true structure stiffness. Considering full composite action of the floor slabs provided the additional 10% structural stiffness needed to calibrate the observed response. Figure 4 shows the correlation between computed and recorded response during the Northridge earthquake.



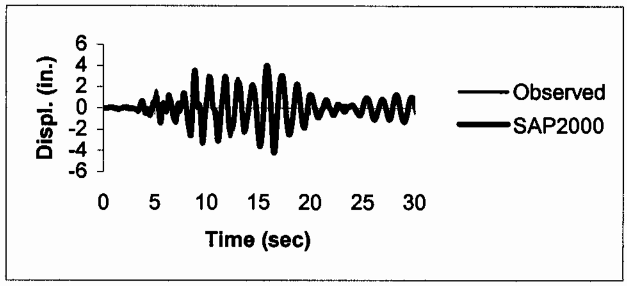
(a) N-S response (2nd level)



(b) E-W response (2nd level)



(c) N-S response (roof level)



(d) E-W response (roof level)

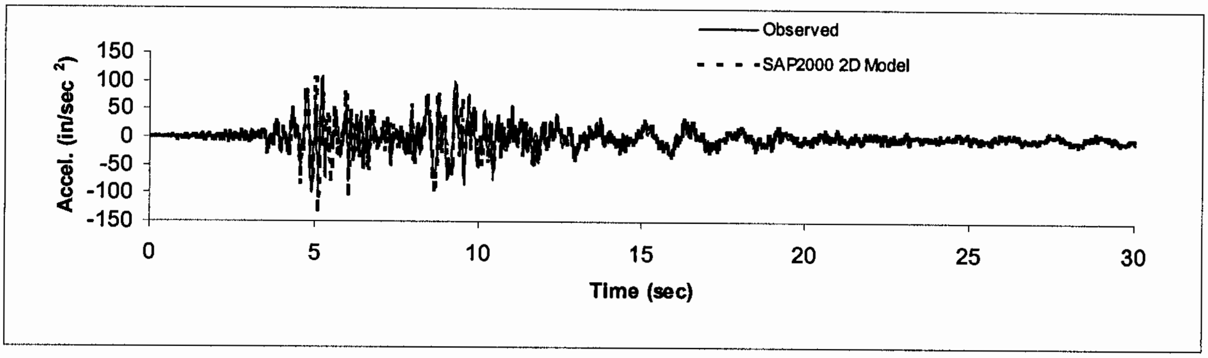
Figure 4. Calibrated Response of 6-Story Building to the Northridge Earthquake

A constant viscous damping of 3% in the first mode and 10% in the remaining modes was used in the analysis. The increased damping value used in higher modes was necessary to match the observed acceleration response and did not influence the displacement response. The computed acceleration response histories for the 2nd level and the roof are shown in Figure 5.

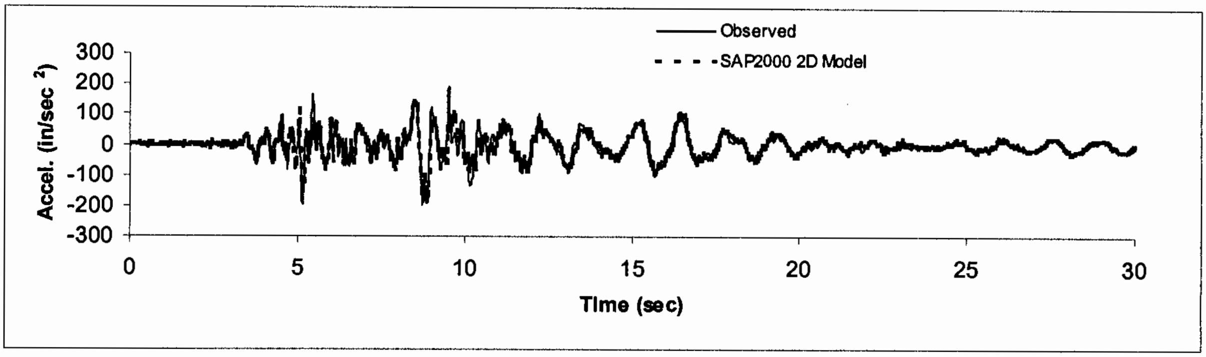
A systematic evaluation of the section properties of the building revealed the potential for weak panel zones. Given the fact that the structure experienced a slight period shift from 1.33 seconds during the early phase of the earthquake to 1.43 seconds, it was considered worthwhile to investigate potential panel zone yielding. A 2D model of the structure was, therefore, developed and calibrated. The results did show some panel zone yielding but the resulting period shift was not evident. Panel zone plastic rotations were less than 0.5% which would be categorized as being within FEMA-273 IO (Immediate Occupancy) limits.

Evaluation of FEMA-273 Analytical Procedures

The final set of evaluation tasks consisted of performing a FEMA-273 based evaluation of the building. To investigate the validity of static procedures to estimate seismic demands, the building was subjected to lateral loads of the form of Equation (3-7) in FEMA-273. The magnitude of the loads was adjusted uniformly so as to obtain the same maximum roof displacement observed in the Northridge earthquake. The resulting displacement and drift profiles are shown in Figure 6.



(a) Second Floor Level



(b) Roof Level

Figure 5. Correlation of Analytically Computed Acceleration Response with Recorded Data in EW Direction During Northridge Earthquake

Next, the building was subjected to lateral loads corresponding to BSE-1 and BSE-2 events as specified in FEMA-273. The Basic Safety Objective (BSO) as defined in FEMA-273 requires Life-Safety (LS) performance for BSE-1 and Collapse Prevention (CP) for BSE-2. In addition to LSP and NSP, a linear dynamic procedure (LDP) using a response spectrum analysis was also carried out. The spectra for both hazard levels were generated using procedures described in the FEMA guidelines. Some of the relevant results of the evaluation are summarized in Figures 7 and 8. A discussion of these results is presented in the next section.

Summary

It was concluded that modeling the structural system using bare section properties did not represent the actual structural stiffness. The contribution of non-structural elements, semi-rigid connections, etc. could be considered in an equivalent sense by incorporating full composite action of the floor slabs. The peak drift profile is not adequately estimated by linear or nonlinear static procedures. Estimates of deformation demands using linear static methods are more conservative than linear dynamic methods based on a response spectrum analysis. FEMA-273 based evaluations suggest that the building will pass BSO requirements with the exception of the nonlinear static procedure which indicates CP requirements being violated at the lowest level.

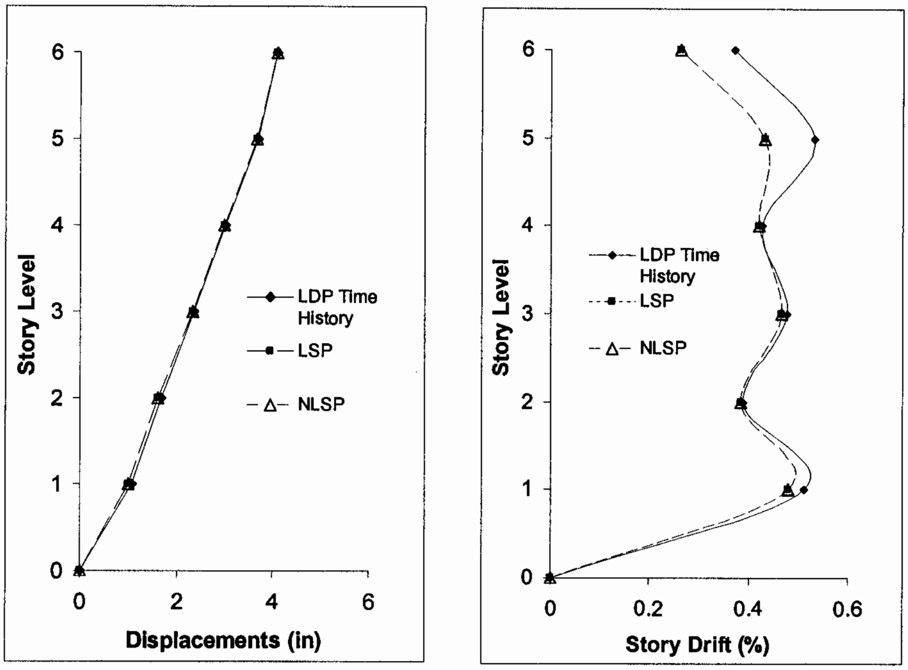


Figure 6. Peak Displacement and Drift Profile Estimates Using Various Analysis Methods (EW Direction Response During Northridge)

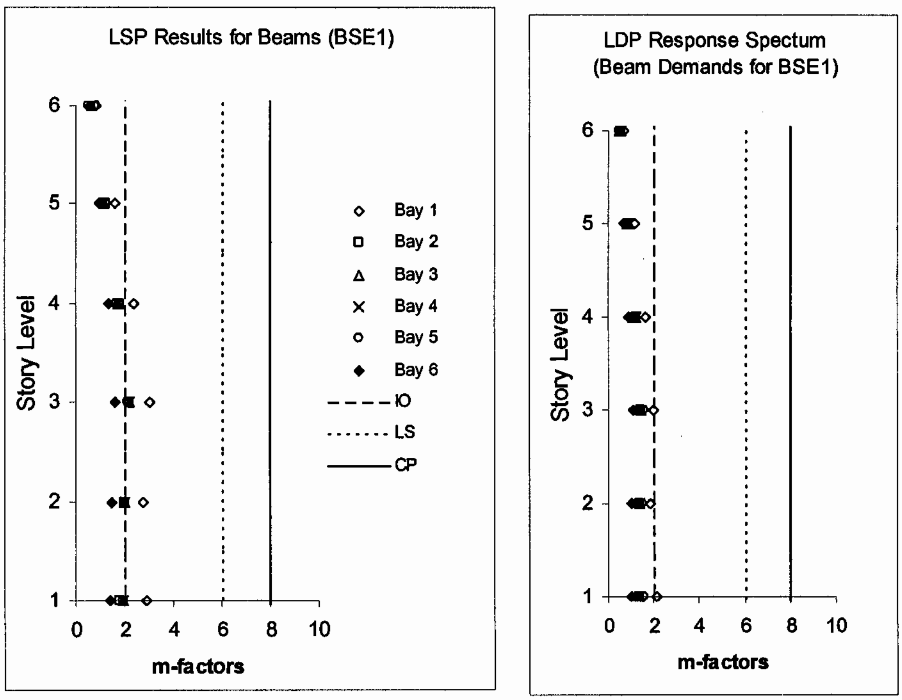


Figure 7. Comparison of Beam Deformation Demands with FEMA-273 Acceptance Criteria for Linear Procedures

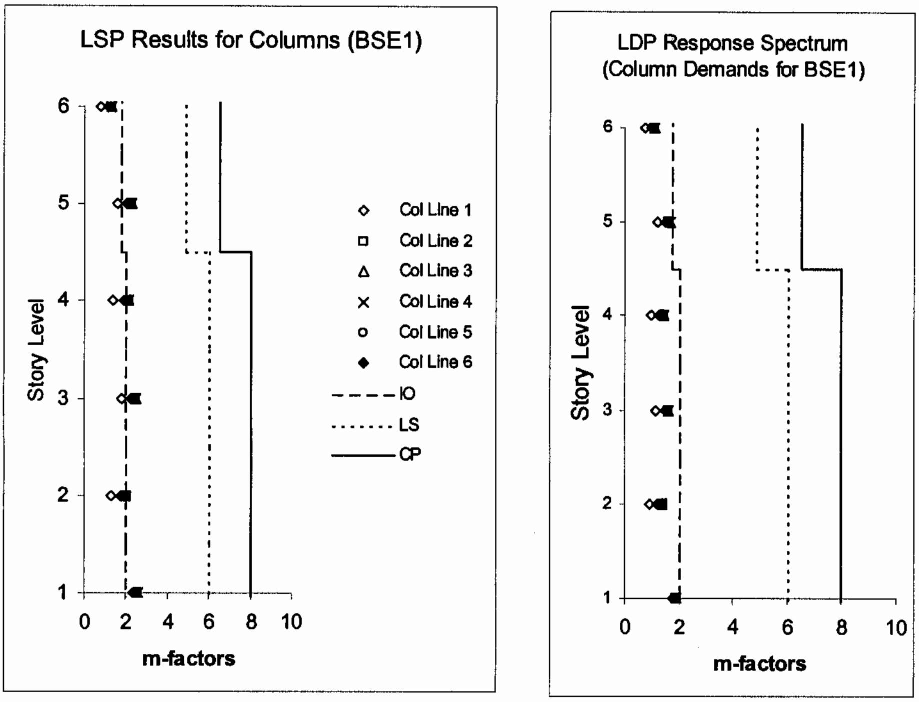


Figure 7 (continued). Comparison of Column Deformation Demands with FEMA-273 Acceptance Criteria for Linear Procedures

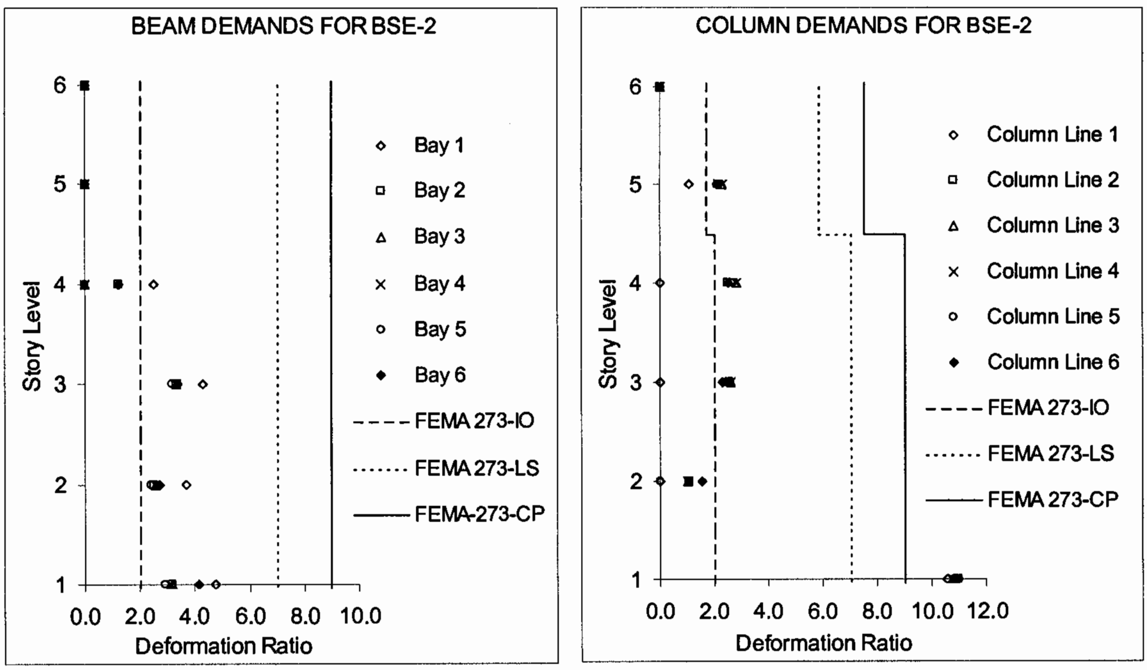


Figure 8. Comparison of Deformation Demands vs. FEMA-273 Acceptance Criteria for Nonlinear Static Procedure (NSP)

NINETEEN STORY OFFICE BUILDING, LOS ANGELES

The second building to be evaluated is a high-rise office building with nineteen stories above ground and four parking levels below ground. Instrumented data for this building are available since the 1971 San Fernando earthquake. While there were only three sensors present during the 1971 earthquake, as many as fifteen sensors during the Northridge earthquake. No structural damage has been reported in any earthquake, however, non-structural damage and resulting losses have not been insignificant. The evaluation reported in this study is limited to the response of the building to the Northridge event.

Building Details

The true geographical North-South direction does not coincide with the orientation of either of the building directions. The long direction of the building (referred to as the EW direction in this study) is oriented at S44°W. The lateral load resisting system consists of four ductile steel moment frames in the EW direction and five X-braced steel frames in the NS axis (short building direction). A typical plan of the building and the location of the strong motion sensors are indicated in Figure 9. The main structural system rests on steel I-beam piles up to a depth of 72.3 feet. The piles are capped in groups and connected by 24 inch square reinforced concrete tie beams.

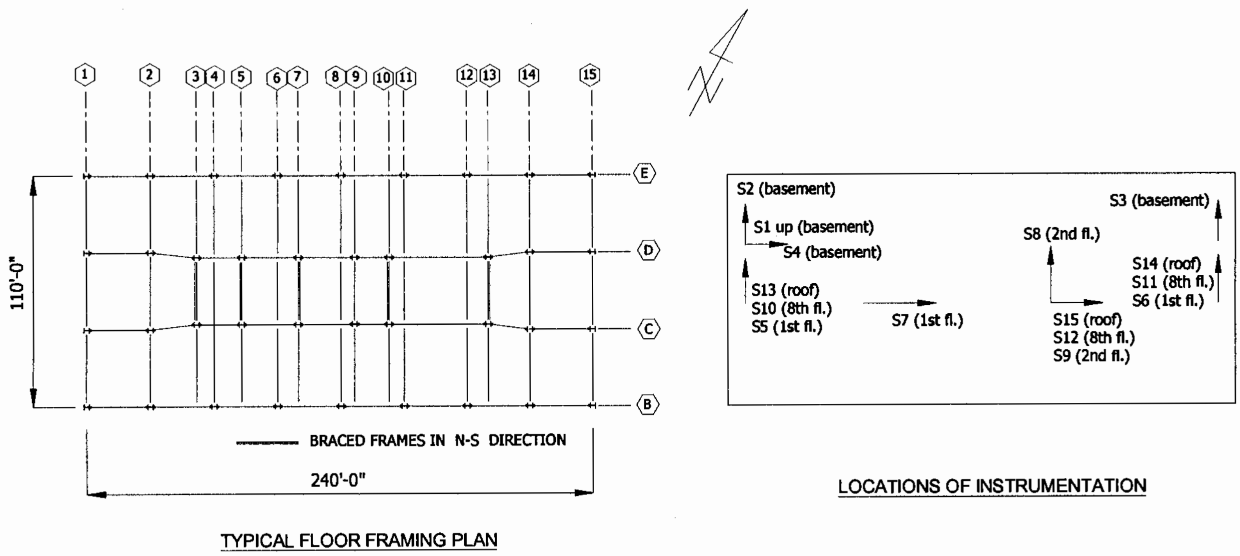
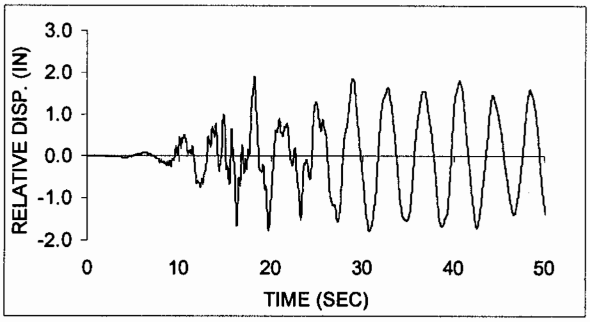


Figure 9. Typical Plan View and Instrumentation of 19-Story Building

Calibration of Observed Response

A three-dimensional model of the entire building was developed using SAP2000. In all, 58 separate column types and 23 different beam types were used to model the building. Though this appears reasonable for a building of this size, the difficulty with modeling and processing the results stems from the fact that different beam sizes are used in each bay at a given level. The building weight, including estimates of non-structural elements such as partition walls and the mechanical equipment in the roof, was estimated to be 58,341 kips. A constant viscous damping factor of 3% of critical was used in all modes. Note that the building has an intermediate level between the ground and first floor and this level has been modeled as the first level, hence results are shown for 20 levels. A wire-frame model of the building as used in the SAP2000 evaluation is shown in Figure 10.



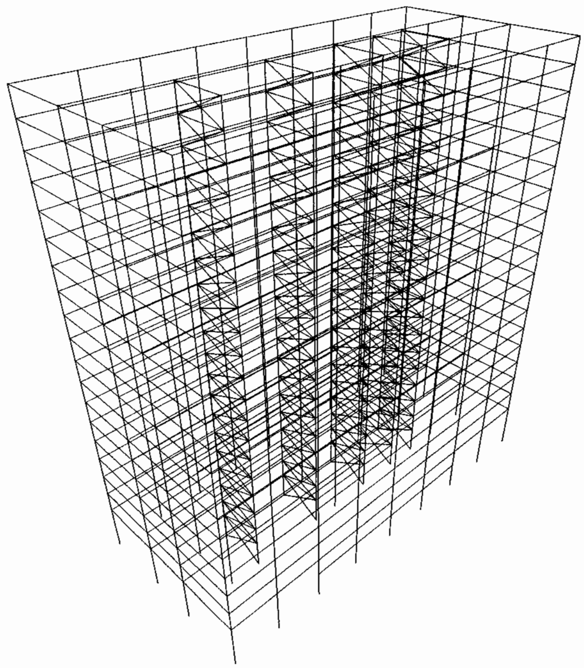
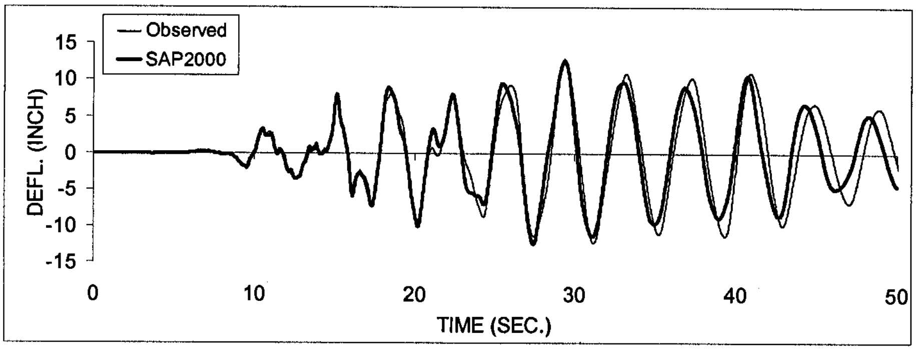


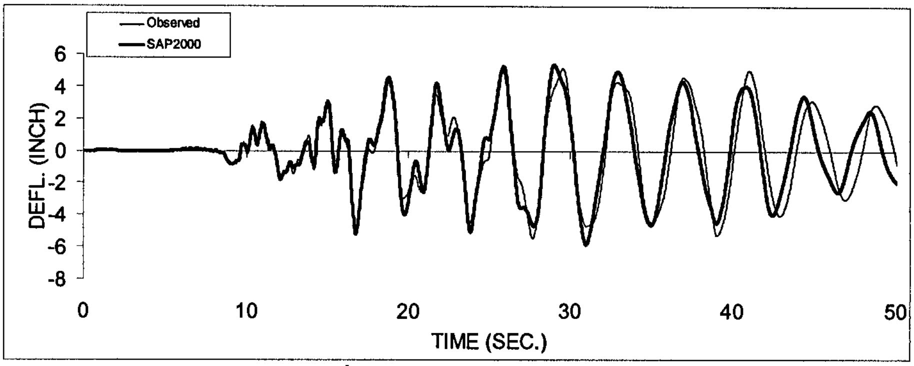
Figure 10. SAP200 Model of 19-Story Building and Relative Displacement Between Channel 13 and Channel 14 at Roof Level

An eigenvalue analysis of the structure indicated a dominant torsional mode of vibration at a period of 4.27 seconds. A plot of the relative displacement between channels 13 and 14 (see Figure 9) which are located at either end of the building shows a differential displacement of almost 2.0 inches (approximately 15% of the total displacement) supports this finding. The first lateral mode of vibration is in the long (EW) direction of the building at a period of 3.8 seconds and the next lateral mode in the NS direction has a period of 3.12 seconds.

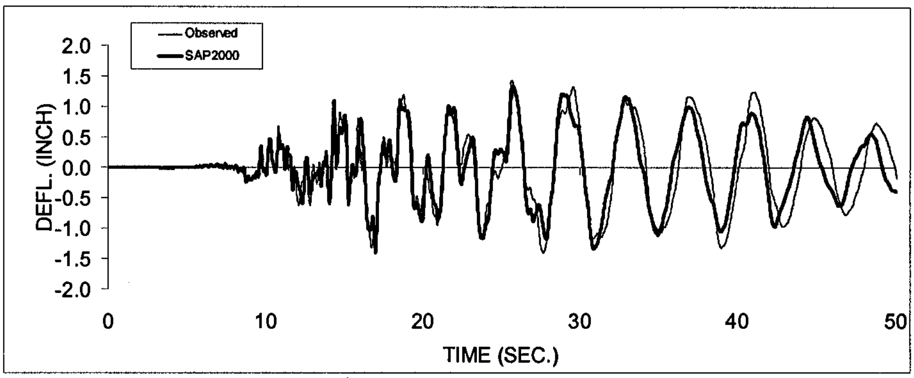
The building model was subjected to the recorded Northridge accelerations in both building directions, independently and simultaneously. A rigid diaphragm assumption was used in the analysis. Results of the analyses indicate that the structural system remained elastic throughout the duration. A more accurate match of the recorded response was obtained with both lateral components of the earthquake applied simultaneously. The resulting time histories of the displacements at various levels are shown in Figure 11. Additionally, it was observed that the torsional mode of vibration was not excited in the analysis and the relative displacement recorded in the actual building (Figure 10) was not captured in the SAP2000 run. It may be concluded that some diaphragm flexibility may have contributed to the relative movement.



(a) Roof Displacement History



(b) 8th Floor Displacement History



(c) 2nd Floor Displacement History

Figure 11. Validation of Building Model with Recorded Response

Evaluation of FEMA-273 Analytical Procedures

The calibrated building model was then evaluated using the different analytical approaches recommended in FEMA-273 for both BSE-1 and BSE-2 loading. Separate evaluations were carried out in each building direction. Since the building is composed of four moment frames in the long direction and five braced frames in the short direction and the elements in a given bay for each frame varies, the task of carrying out a detailed FEMA-273 evaluation was tedious. Due to space limitations, only essential results for one critical frame in the weak direction of the building is presented here.

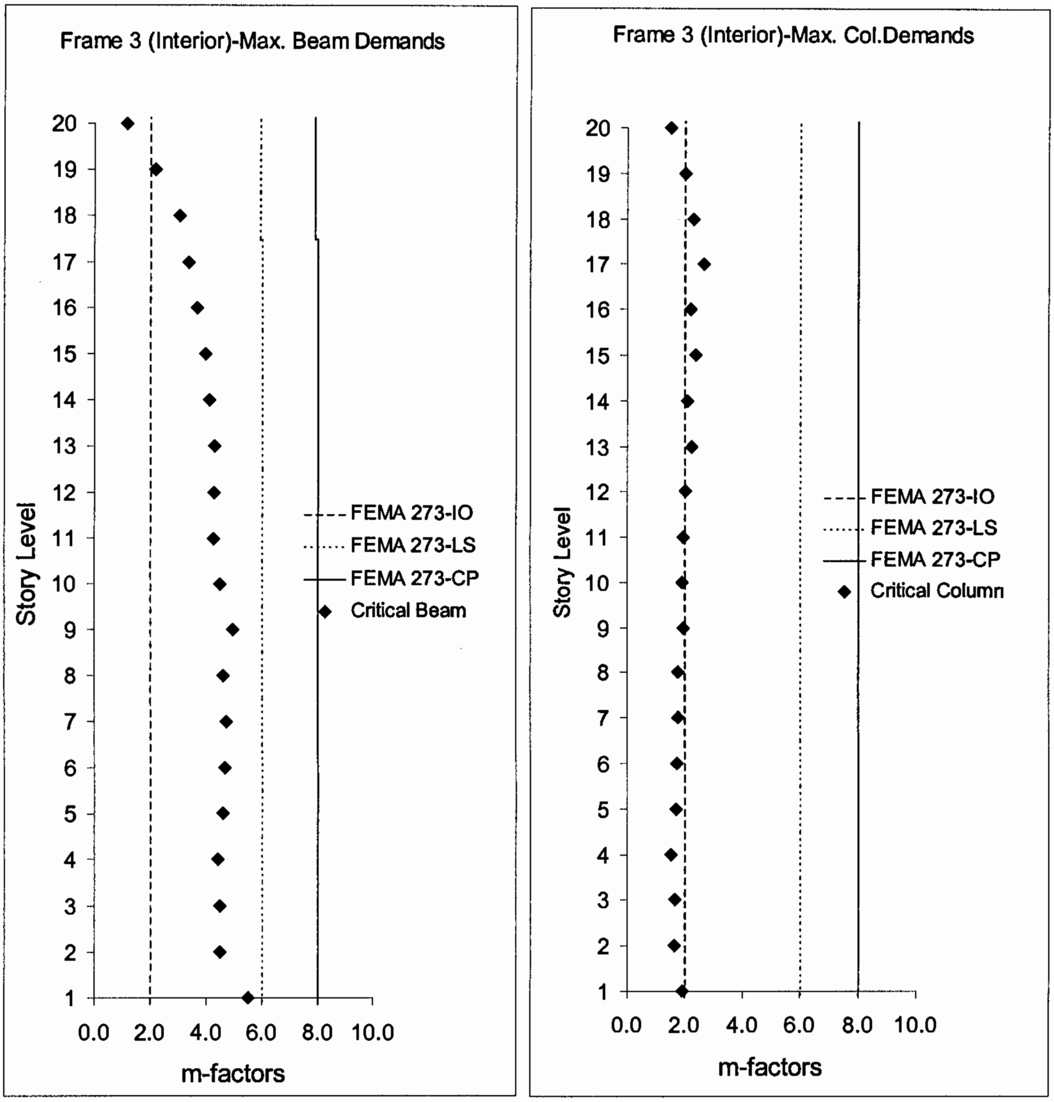


Figure 12. Comparison of Critical Component Demands using LSP with FEMA-273 Criteria for BSE-2 Loading

Figure 12 shows the demand to capacity ratios for one critical frame in the long building direction using LSP (linear static procedure). The applied lateral loads were based on BSE-2 loading. Also shown in the figure are the acceptance criteria based on the m-factor for the most critical component at each story level. In general, similar m-values were obtained for all components in a story except the interior frames in the EW direction which were composed of different sections in each bay. Figure 13 shows a similar set of results using the linear dynamic procedure (LDP). The dynamic analysis was carried out using a response spectrum approach using the FEMA-273 spectra for BSE-2 loading. Finally, results of the evaluation using an inverted triangular loading and NSP (nonlinear static procedure) are displayed in Figure 14. The target displacement, in this case, was estimated using BSE-1 loading.

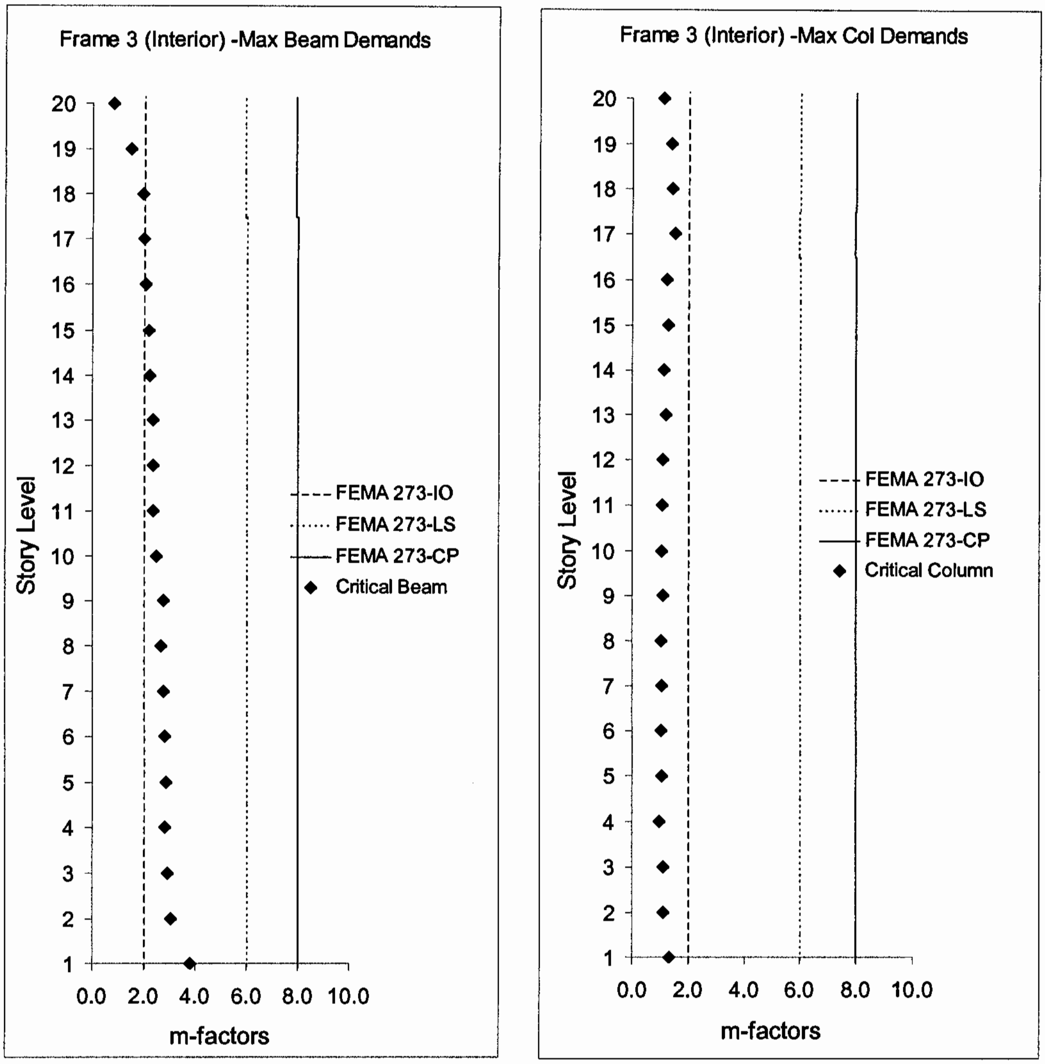


Figure 13. Comparison of Critical Component Demands using LDP with FEMA-273 Criteria for BSE-2 Loading

Summary

System identification studies revealed moderate torsional response in the building. Since most of the frames in this building was designed to resist lateral loads, the influence of the gravity frames was minimal. Good correlation was obtained with recorded response using bare section properties for all elements. Response results with base accelerations applied in both directions produced improved correlation than if the base motions were applied independently. As in the case of the previous building, damping did not influence displacement response but increased damping in higher modes improved the computed acceleration response. The FEMA-273 evaluations demonstrated once again that LSP demands are more conservative that LDP demands if a response spectrum analysis is used. Demands from NSP did not correlate well with the results from linear procedures.

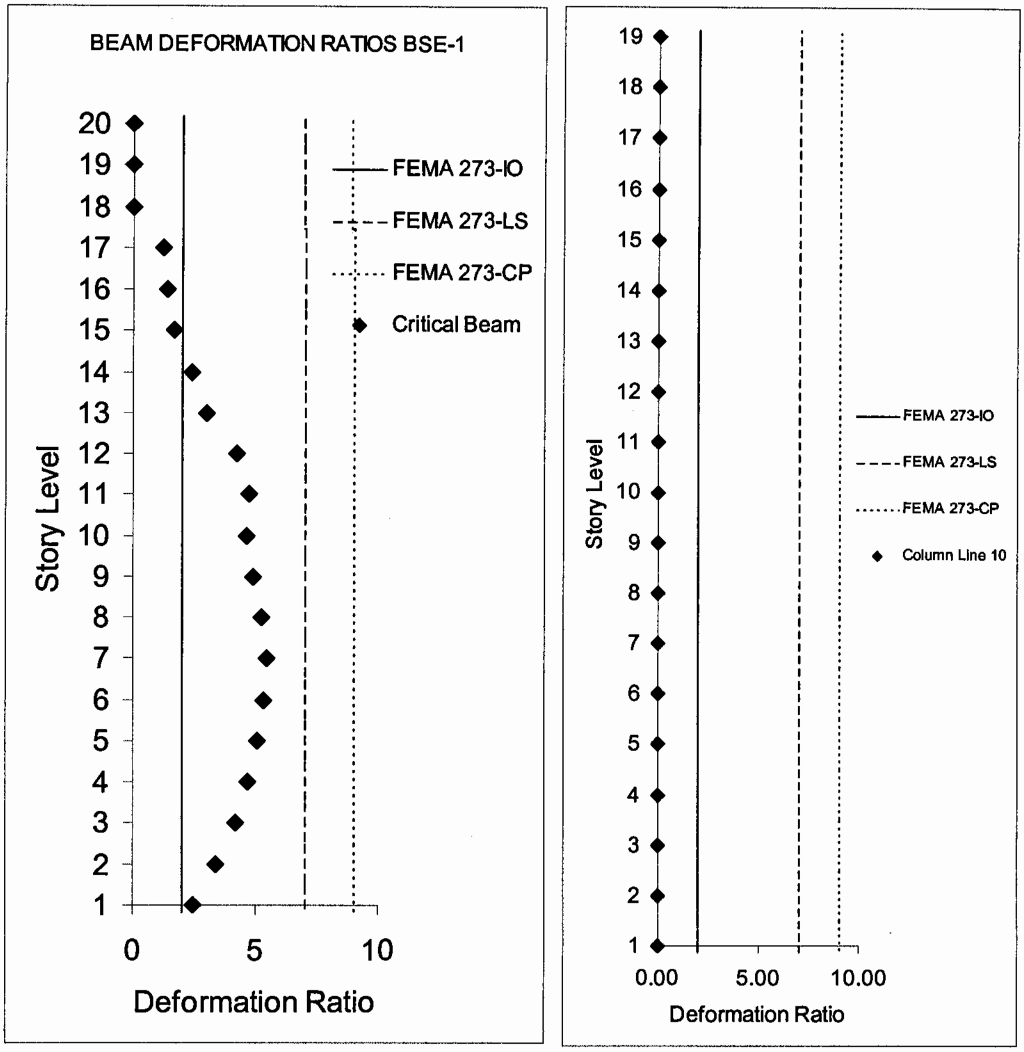


Figure 14. Comparison of Critical Component Demands using NSP with FEMA-273 Criteria for BSE-2 Loading

It is appropriate to point out that the pushover analysis of this building using SAP2000 was almost prohibitive. In addition to the excessive computational time (estimated at over 20 hours on a 400 MHz Pentium II Processor with 96mB RAM) for a single run in one direction, the enormous amount of output generated was extremely difficult to process. The feasibility of conducting routine iterative pushover analyses of tall buildings with a large number of degrees-of-freedom may be called into question, particularly in structural design offices.

Further, the results of the pushover analysis indicates that the deformation demands in the columns are significantly smaller than that predicted by linear approaches. In fact, no column yielding was reported. Of the three approaches, LSP demands clearly resulted in the most conservative estimates.

THIRTEEN STORY COMMERCIAL BUILDING, SOUTH SAN FERNANDO VALLEY

The Northridge earthquake caused widespread damage to numerous steel buildings, particularly local failures in beam-to-column welded connections. The third building considered in this study was among the many steel moment frame structures that experienced weld fracture at beam-to-column connections. This 13-story building is located in South San Fernando valley about 5 km southwest of the Northridge epicenter. This building is actually composed of one basement floor and 13 floors above ground. It was built in 1975 on a design based on the 1973 UBC code. This building has been the subject of a previous investigation (Uang et al., 1995).

Building Details

The floor plan of the perimeter frames and a typical elevation of one of these frames are shown in Figure 15. The overall building dimensions are 160 x 160 feet. Member sizes are also shown in the figure. It should be noted that the corner columns are composed of box columns. All structural steel was specified to be A36 steel, however, an expected yield strength of 45 ksi was used in the FEMA-273 evaluations. The floor plan increases at the second floor to form a plaza level which terminates on three sides into the hillside thereby making this level almost fixed against translation. There are concrete walls in the basement perimeter which also add to the stiffness of the basement level. The foundation consists of piles, pilecaps and grade beams.

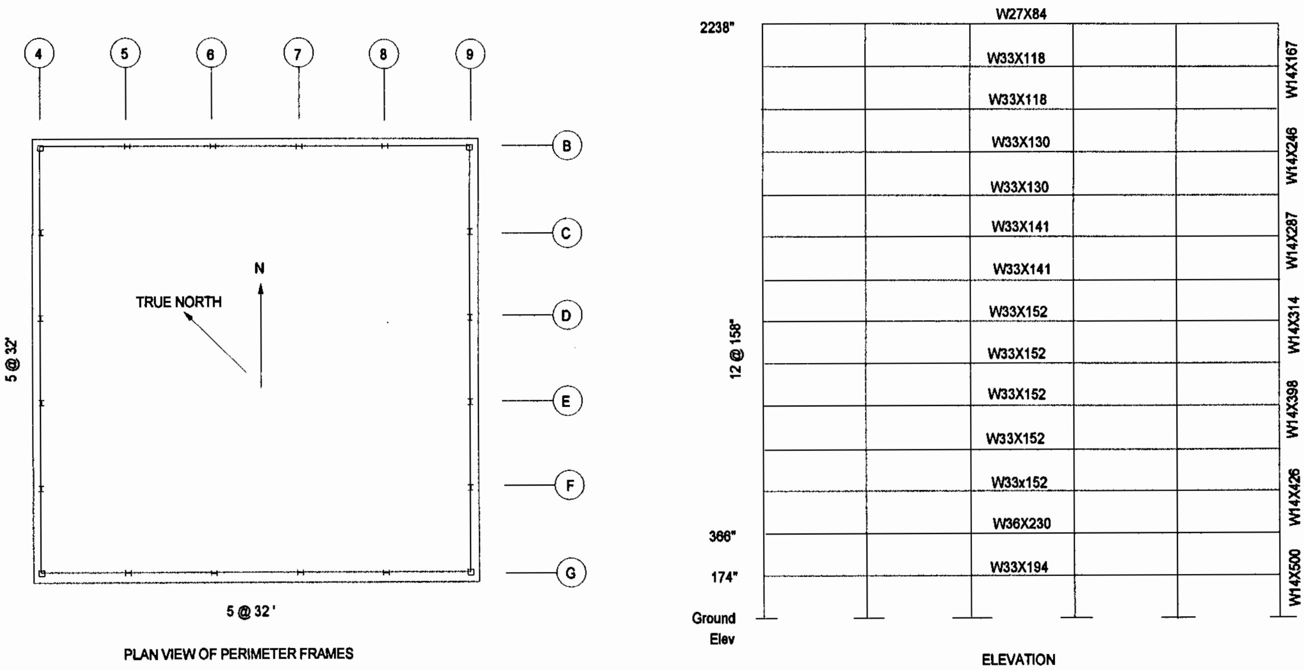


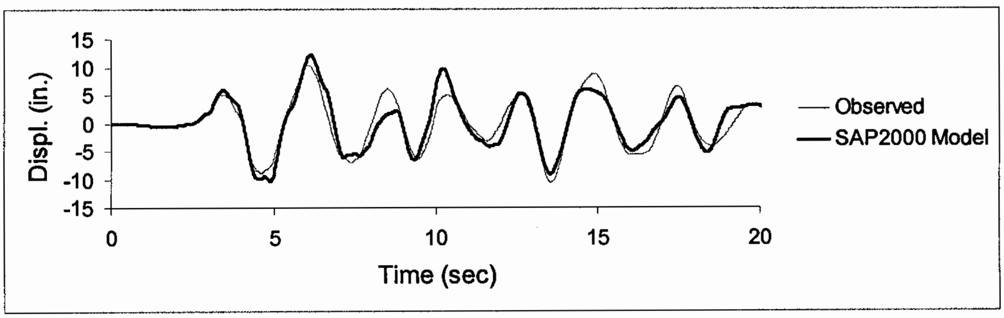
Figure 15. Plan and Elevation of Perimeter Frames of Blue-Cross Building

The typical floor system consists of about 2.5 inches of concrete fill over 3-inch 20-gage steel decking. The roof system is lighter with 2.25 inches of vermiculite fill on 3-inch 20-gage steel decking. 3 ksi concrete was specified for all deck fill. Exterior walls are composed of 6-inch 22 gage steel studs with 0.25 inch opaque glass and 2-inch precast panels. A total uniform load of 102.5 psf was used to calculate the building mass properties and axial load on columns. Strong motion data is available for 7 sensors: three each in the North-South and East-West directions, respectively, and one in the vertical direction. The instrumentation was located in the basement and on the sixth and twelfth floors. Recorded accelerations at the basement indicate that the building experienced a PGA of 0.41g in the NS direction and 0.32g in the EW direction.

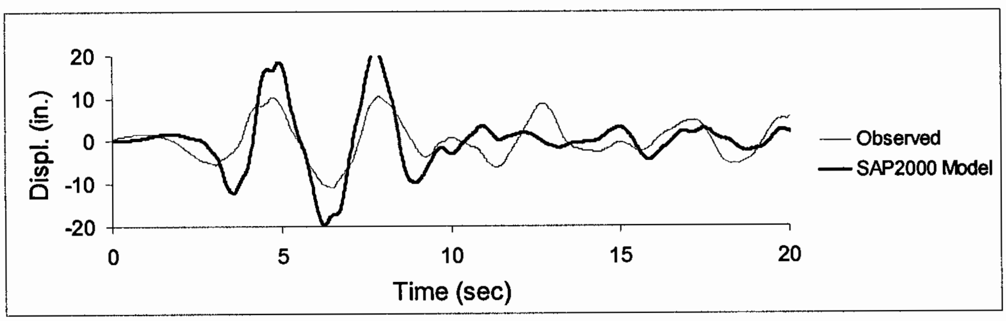
Approximately 12% of the connections on the west perimeter of the North-South frame fractured during the earthquake. Connection fractures on the remaining three sides were less than half this number. The stronger component of the Northridge earthquake was oriented in this direction. Damage consisted 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 reported in Uang et al. (1995).

Calibration of Observed Response

A three-dimensional model of the building was developed using SAP2000. Both the exterior moment frames and the interior gravity frames were included in the model. Bare section properties without consideration for composite floor action were used to model the members. The building was subjected to both recorded components of the earthquake at the basement level. Results of the simulation in both directions are shown in Figure 16.



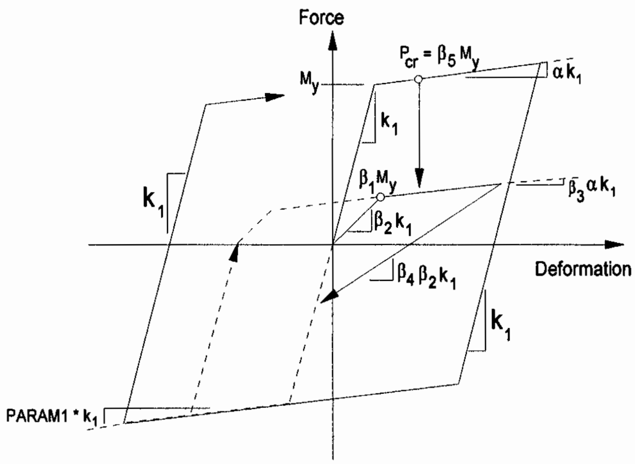
(a) East-West Direction



(b) North-South Direction

Figure 16. Simulation of Observed Roof Response During Northridge

As observed in Figure 16, the correlation in the NS direction is poor. This is to be expected since the building suffered numerous weld fractures in this direction. Hence, a detailed inelastic analysis was carried out on a two-dimensional frame using an advanced hysteresis model to represent the behavior of the fracturing weld connection. Figure 17 shows the weld-fracture model used in the simulation. This model is incorporated in the IDASS computer program (Kunnath, 1995). The weld-fracture model was first calibrated using available experimental data from tests conducted at the University of Berkeley (Popov et al., 1996). It was assumed that the welds fractured at approximately 70% of the specified yield strength (31.5 ksi). Results of the simulation in the NS direction using this model is shown in Figure 18. Some improvement in the simulation is observed though all features of the response could not be calibrated.



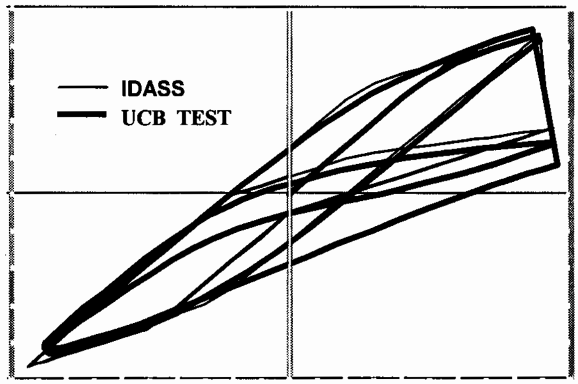


Figure 17. Calibration of Weld Fracture Model with Test Data

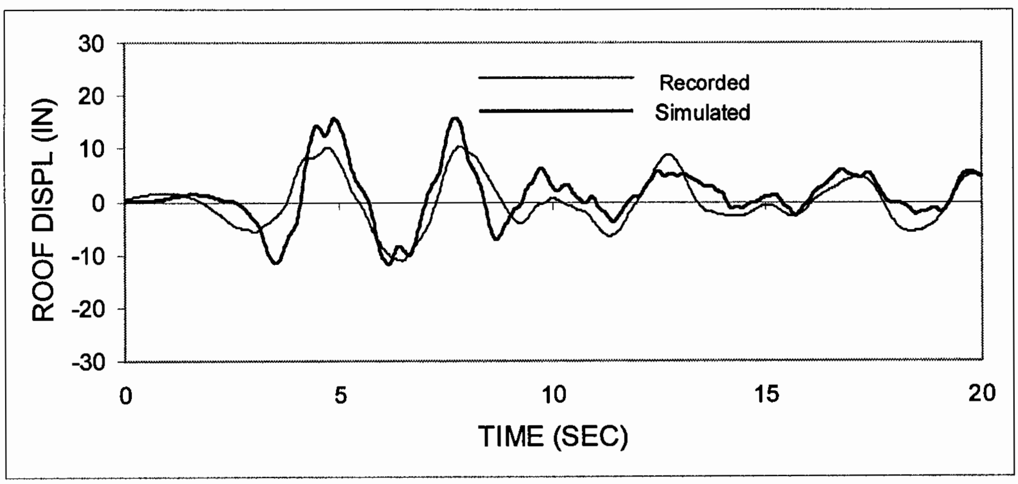


Figure 18. Simulation of Observed Roof Response Using Weld Fracture Model

Evaluation of FEMA-273 Analytical Procedures

FEMA-273 based evaluations of the building were carried out on a two-dimensional model of the building with the assumption that there was no potential for weld fracture. Results are presented only for the linear and nonlinear static procedures. Figure 19 shows the m-factors for both beam and columns as a function of the story level for BSE-2 loading. Exterior columns are distinguished from interior columns because the peak axial stresses occur in the exterior columns and typically govern the acceptance criteria for the building. Figure 20 displays the results of the nonlinear static analysis wherein an inverted triangular loading pattern was used. Post yield stiffness of both beams and columns was assumed to be 2% of the initial elastic stiffness. All plots also include the FEMA acceptance criteria for immediate occupancy, life safety and collapse prevention.

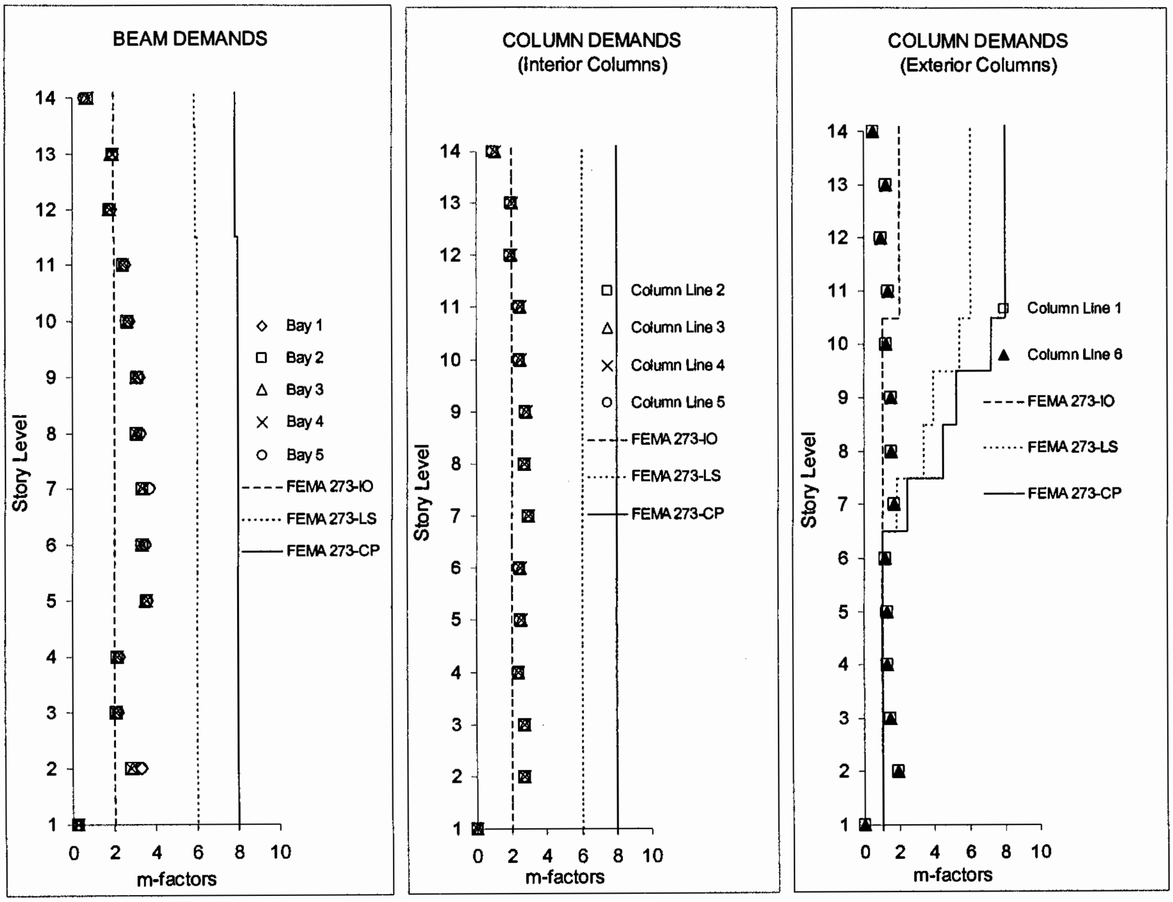


Figure 19. Comparison of Critical Component Demands using LSP with FEMA-273 Criteria for BSE-2 Loading

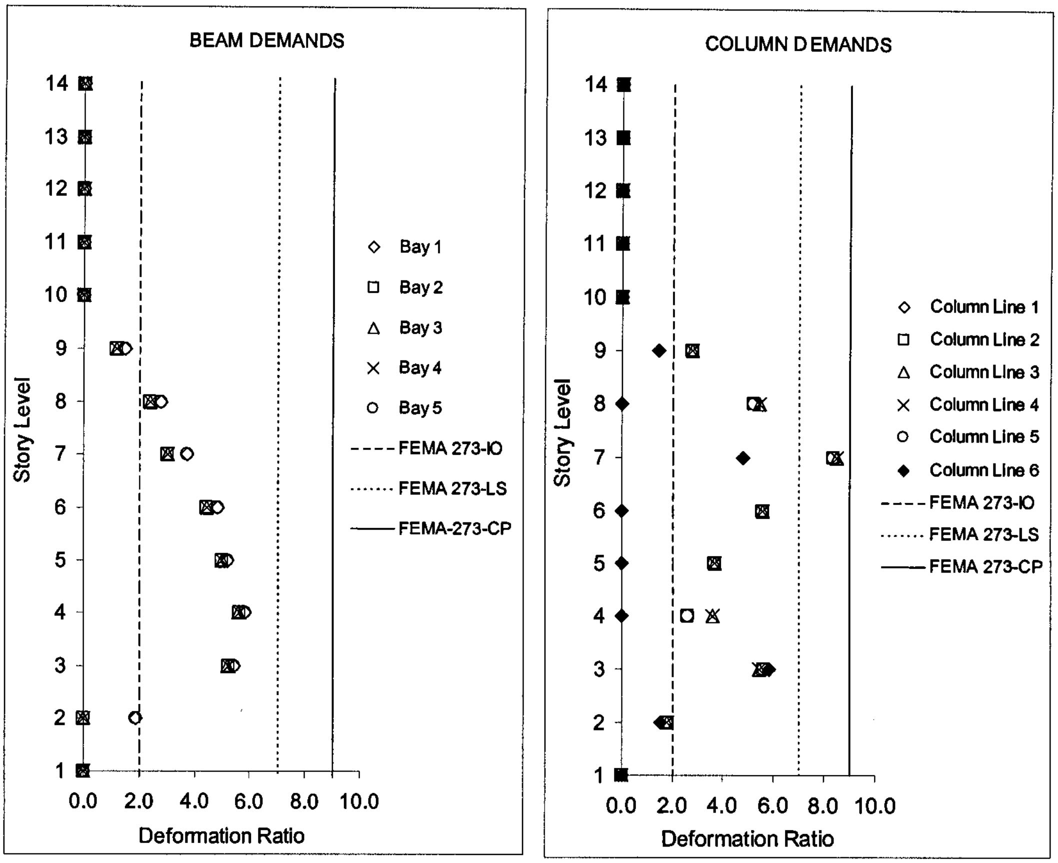


Figure 20. Comparison of Critical Component Demands using NSP with FEMA-273 Criteria for BSE-2 Loading

Figure 21 displays the FEMA evaluation of the welded connections. The criteria for potential connection failure is a function of the panel zone strength and the depth of the beam. Results are shown for three cases: LSP and NSP based on BSE-2 loading and NDP using the actual recorded time history. The observed connection fractures in the two perimeter frames in the NS direction is also shown for comparison. It is to be noted that the perimeter framing system is essentially symmetric, hence computer based simulations will result in identical damage scenarios in both the EW and NS directions. However, observed damage in the frames in the EW direction were substantially less. This is indicative of the fact that weld fractures possibly occurred at stresses below the yield stress of the material.

Summary

Calibration of the three-dimensional model was achieved in the East-West direction using bare section properties without composite floor action. The response in the North-South direction was overestimated since the SAP2000 run was based on linear elastic behavior. Post-earthquake investigation of the building revealed numerous weld fractures in the NS direction which explains the lack of correlation in this direction. An enhanced weld fracture model was used to improve the simulation in the NS direction. FEMA-273 based analyses were carried out on the structure assuming no weld fractures. Results of the evaluation reconfirm previous evaluations that LDP based on a response spectrum approach is less conservative than LSP. In fact, results of LSP produce the most conservative demand estimates. There was no correlation in critical demand regions (story levels) between linear and nonlinear methods: NSP predicted maximum beam demands in the lower half of the building while LSP and LDP indicates critical demands in the middle third of the building.

Finally, the potential for connection failure was investigated using FEMA-273 criteria. Both LSP and NSP for BSE-2 loading indicated that CP criteria was not satisfied at several levels across the mid-height of the building.

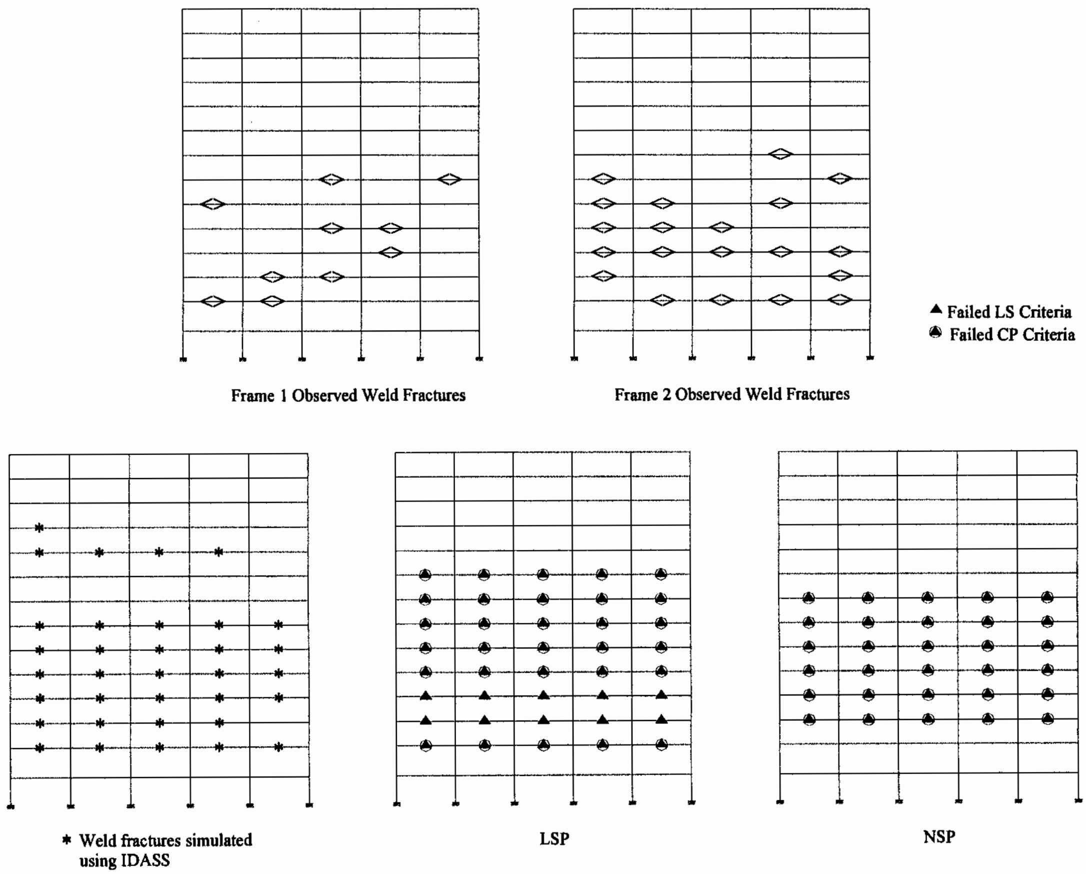


Figure 21. Observed vs. Predicted Connection Fractures Using Various Analysis Methods

TWO-STORY CITY HALL, BIG BEAR LAKE

The fourth and final building evaluated in this project is the two-story Big Bear civic center and city hall. There are no recording instruments in the building but free field records very close to the site indicate a peak ground acceleration of 0.48g in the east-west direction and 0.54g in the north-south direction during the Big Bear earthquake of 1992. No damage to the structure was reported based on visual external examination of the building. A detailed investigation of the building was not carried out until after Northridge. After weld fracture problems were identified following the Northridge event, a closer inspection of the 2-story civic center revealed severe cracking in several welded connections. A complete retrofit of the building has since been carried out. This building is included in the study since it represents a typical yet complex system with steel and timber members.

Building Details

Lateral resistance in the Big Bear building is provided by a set of moment frames. The main lateral force resisting system of the structure is not well defined. The building was completed in 1987 and was designed based on 1985 UBC provisions. The floor system consists of wood floor joists with plywood cover. The dead load of the floor was estimated at 35 psf. The total building weight was calculated to be approximately 450 kips including structural steel and plywood walls. Additional lateral stiffness is provided by plywood walls that infill the moment frames. Structural drawings show that the plywood diaphragms, reinforced with 2"x6" studs at 24 inch on center, are connected to the steel framing system through bolts at the upper and lower beams thereby participating in the initial response and contributing to the overall structural stiffness. No connections are provided between the wall diaphragm and the columns.

Simulation of Building Response

Several computer models of the building were investigated: a reduced 3D model in which only the main framing systems in each direction were included; a complete 3D model with all moment frames; a 3D model with diagonal braces to simulate the stiffness of the plywood wall panels; and two dimensional models in each direction of the building. The final model selected for preliminary evaluation was the full three-dimensional model of the building. The model was subjected to the free-field accelerations that were recorded near the site. The SAP2000 model of the building is shown in Figure 22. Note that the structure contains a few isolated single bay moment frames. Since these frames contribute to the overall stiffness of the building, they were included in the analysis. Most of these isolated frames are connected to the rest of the building through wooden beams and diaphragms or steel beams with simple shear connections. A rigid diaphragm assumption was used to tie all frames together.

The results of the seismic analysis of the building model subjected to bi-directional components of the free-field motion indicated several regions of high stress. Demand to capacity ratios for a typical frame in each building direction is shown in Figure 23. The peak ratios based on a yield stress of 36 ksi is 1.71for columns and 1.26 for beams. These ratios may be not appear to be critical considering factors such as material over-strength and redundancy. However, as identified in the analysis of the 13-story frame, it is possible that brittle fracture was initiated before yielding of the connections.

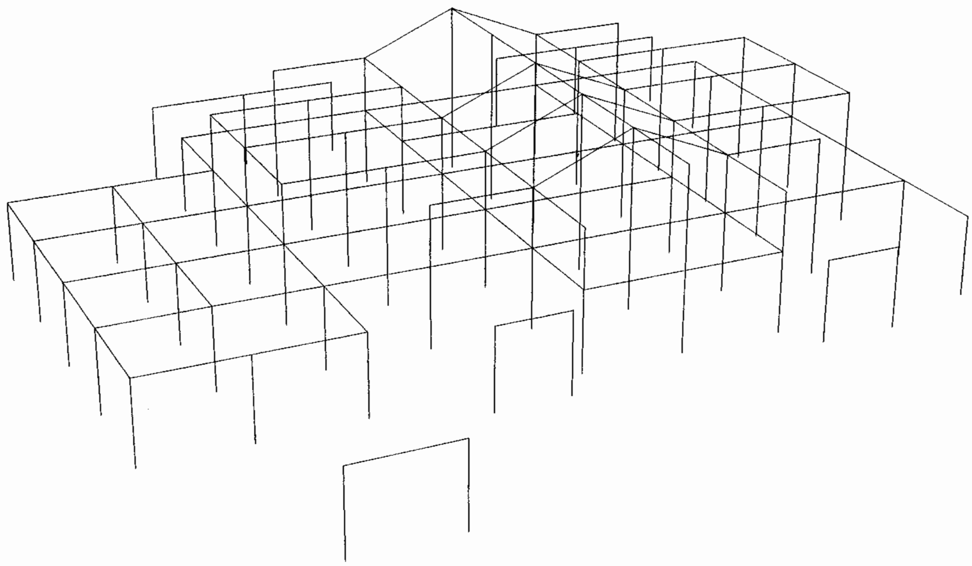
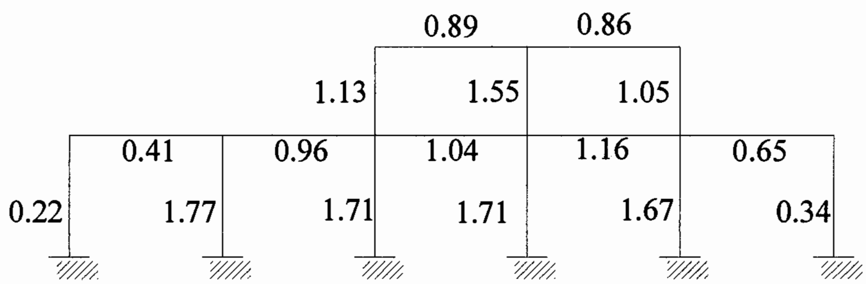
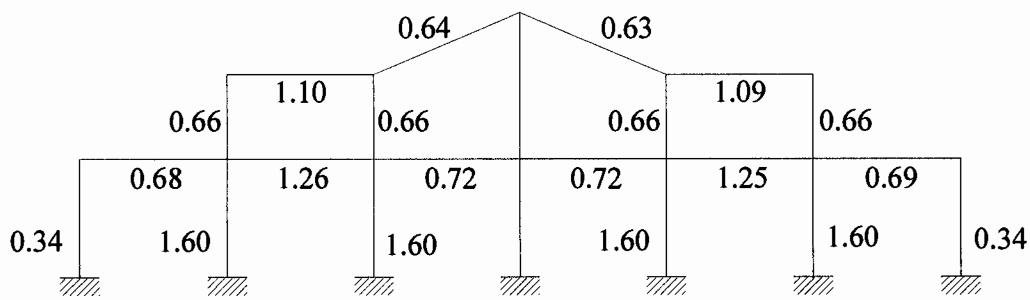


Figure 22. SAP2000 Model of 2-Story Big Bear Civic Center



(a) Typical NS Frame



(b) Typical EW Frame

Figure 23. Demand to Capacity Ratios in Critical Frames of Big Bear Civic Center Subjected to Recorded Free Field Motions at Nearby Site

An additional factor not considered in the evaluation that may have played a significant role in the seismic forces experienced by the structure is the presence of plywood diaphragm walls. In order to consider the effects of the wood diaphragms, a separate set of analyses was carried out to identify the increase in stiffness due to the presence of these elements. Test data obtained from the city of Los Angeles (Nghiem, 2000) based on experiments conducted at the University of California, Irvine, was used to calibrate the approximate stiffness of a steel moment frame with plywood diaphragm walls. Figure 24 shows the conceptual process of calibrating the brace elements used to represent the walls. The equivalent shear stiffness of the frame with the brace element was simulated to represent the steel frame with the actual plywood wall. Membrane elements were used to model the plywood panel.

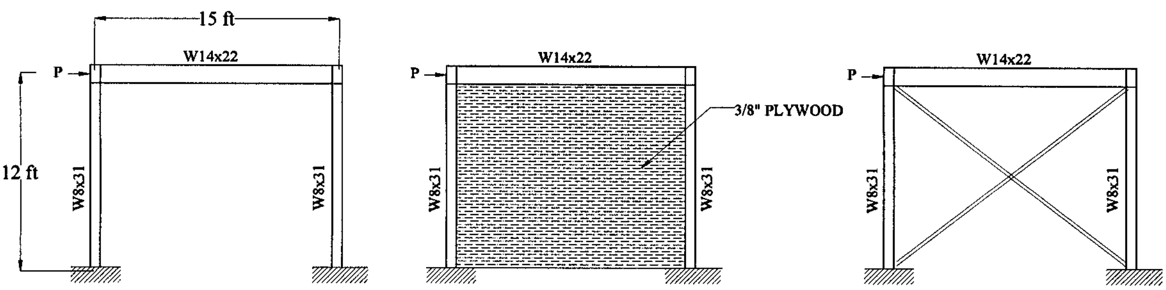


Figure 24. Modeling the Effect of Wood Diaphragm Walls

The calibrated brace element was then used in a new analysis of the building. It was observed that the model with the brace elements resulted in a 50% increase in shear in the EW direction and more than 30% increase in the NS direction. However, the DCR values in the beams and columns were reduced since it was not possible to limit the brace forces. In an actual event, the wall panels will most likely separate from the frame and the forces will be carried by the main framing system. Hence, the results of the evaluation with the equivalent braces in the inelastic range is not valid. The objective of analyzing the building with the wall panels was to ascertain the initial stiffness of the system and the resulting increase in shear forces.

Finally, the 3D SAP2000 model shown in Figure 22 was analyzed for FEMA-273 loading. Both BSE-1 and BSE-2 hazard levels were considered. Analyses were carried out using LSP and NSP. Beam and column demands for BSE-2 loading are shown in Figure 25. Additional investigation of connection behavior at these loading levels indicate no likelihood of connection fracture. While a few connections did not pass Immediate Occupancy criteria, all connections passed both LS and CP criteria for both loading levels. The demands resulting from the Big Bear earthquake were more severe than those determined from FEMA-273 loading.

Figure 26 shows the regions that were upgraded following the Northridge earthquake. A number of these locations coincide with the regions of maximum stress identified in Figure 23. The NS frame shown in Figure 23 lies along line 3 and the EW frame lies along B. The stress ratios along frame line B were generally higher than those along frame line D. Correlations between stress ratios and potential damage may not be consistent if welds fractured prior to yielding.

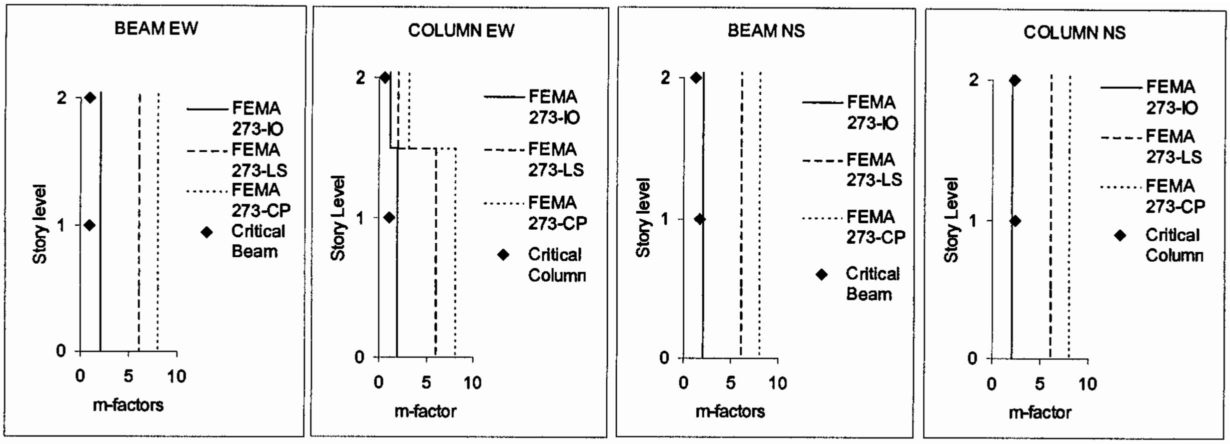


Figure 25. Comparison of Demand Estimates Using LSP with FEMA-273 Acceptance Criteria for BSE-2 Loading

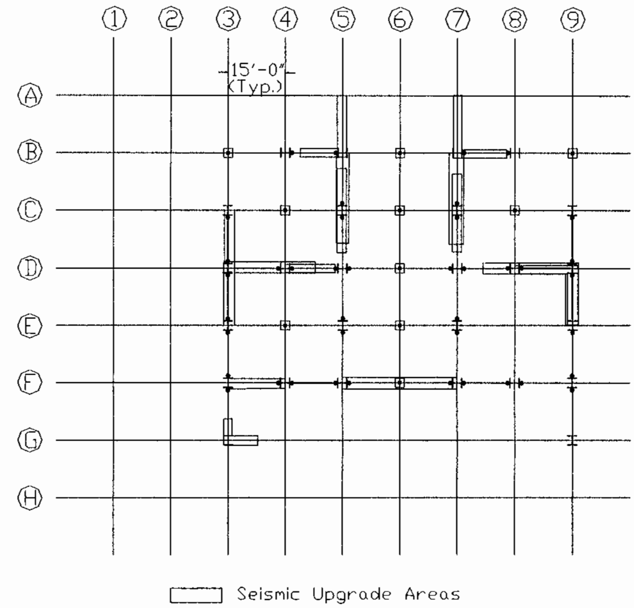


Figure 26. Plan View of Building Showing Locations of Connection Damage

Summary

Results of the evaluation of the 2-story Big Bear building were less conclusive given the complexity of the structural system. The influence of the wood wall panels in inducing more severe seismic demands in the initial phase of the response was demonstrated by utilizing equivalent diagonal brace elements. The actual response of the system in the presence of the walls could not be ascertained given the limitations of the computational tools available. The FEMA-273 criteria for connection failures appears to be inadequate for beams with limited depth. The current criteria based on beam depth was incapable of predicting any of the observed weld fractures. Additional studies of the building are still underway.

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