3. Structural

3.1 Tower Structure

EXECUTIVE SUMMARY

Scope: This report provides results of a seismic evaluation of the Board of Equalization Building at 450 N St Sacramento, California, performed by Buehler & Buehler Structural Engineers, Inc. (B&B) to supplement preliminary studies performed by Degenkolb Engineers dated 2006 [2]. The scope of this study is to evaluate the seismic performance of the 24-story tower's lateral force resisting system by conducting a three-dimensional nonlinear response history analysis (NRHA). The impetus of evaluation using NRHA is due to the presence of "Pre-Northridge" steel moment-resisting frame connections.

Seismic Evaluation Criteria: The seismic assessment is based upon a prescribed Earthquake Hazard Level and a specified structural performance level per the 2007 California Building Code (CBC) [9]. Two performance objectives are required by the 2007 CBC Section 3415: Level 1 and Level 2.

Performance Level	Hazard	Acceptance Criteria
Level 1	BSE-R (225 yr, 20%/50yrs)	Life Safety (S-3)
Level 2	BSE-C (975 yr, 5%/50yrs)	Collapse Prevention (S-5)

The Level 1 objective is to achieve Life Safety performance under an earthquake with a return period equal to 225 years. The Level 2 objective is to achieve Collapse Prevention performance under an earthquake with a return period equal to 975 years. As referenced in the CBC, the evaluation was performed in accordance with ASCE 41-06[8] "Seismic Rehabilitation of Existing Buildings" and was supplemented with information from FEMA 355D [5] "State of the Art Report on Connection Performance" related to similar beam-column connection assemblages. Seven pairs of scaled ground motions are used to estimate the seismic demand on the structure per ASCE 41-06 and ASCE 7-05 "Minimum Design Loads for Buildings and Other Structures" guidelines.

Building Description: The Board of Equalization Building is 24 stories tall with a typical floor area of 25,300 square feet. The building reaches a height over 350 feet above grade. The floor setbacks occur at the 22nd and 24th floors where a sloped mechanical penthouse and a roof top helicopter pad occur. The structure consists of steel beam and column framing. The floor structure is a composite metal deck at all floors and roof. Exterior skin of the structure consists of precast concrete and glass curtain wall construction. The foundation system consists of precast piles. A four-story parking structure is adjacent to the BOE building with a 5 inch seismic separation. (The parking structure evaluation is under a separate report.)

Lateral Force Resisting System: The lateral system is a steel moment-resisting frame in both directions, with shared columns at each corner of the building. The beams range from 33 to 36

inches deep. The columns are 24 inches deep in wide flange shapes, built-up I shape, or box columns. The beam-column connections are similar to "Pre-Northridge" Welded Unreinforced Flange (WUF) type with certain modifications per AISC 9th edition and the original design drawings.

Seismic Structural Evaluation and Conclusion:
A nonlinear response history analysis (NHRA) was performed. The results suggest that the building meets the 2007 CBC Level 1 and Level 2 criteria. According to the NHRA results, strengthening of the moment-resisting frame system is not required.

INTRODUCTION

This report provides results of a seismic evaluation of the Board of Equalization Building at 450 N St Sacramento, California, performed by Buehler & Buehler Structural Engineers, Inc. (B&B) to supplement preliminary studies performed by Degenkolb Engineers dated 2006 [2]. The scope of this study is to evaluate the seismic performance of the 24-story tower's lateral force resisting system by conducting a three-dimensional nonlinear response history analysis (NRHA). The impetus of evaluation using NRHA is due to the presence of "Pre-Northridge" steel moment-resisting frame connections.

GENERAL BUILDING DESCRIPTION

The building was designed using the 1988 Uniform Building Code (UBC) and was built in 1990. It is 24 stories tall with a typical floor area of 25,300 square feet. The building reaches a height over 350 feet above grade. The floor setbacks occur at the 22nd and 24th floors where a sloped mechanical penthouse and a roof top helicopter pad occur. The structure consists of steel beam and column framing. The floor structure is a composite metal deck at all floors and roof. Exterior skin of the structure consists of precast concrete and glass curtain wall construction. The foundation system consists of precast piles. A four-story parking structure is adjacent to the BOE building with a 5 inch seismic separation. The lateral system is a steel moment-resisting frame in both directions, with shared columns at each corner of the building. The beams range from 33 to 36 inches deep. The columns are 24 inches deep in wide flange shapes, built-up I shape, or box columns. The beam-column connections are similar to "Pre-Northridge" Welded Unreinforced Flange (WUF) type with certain modifications per AISC 9th edition and the original design drawings.

SCOPE OF STUDY

The seismic evaluation of the building was performed in accordance with the requirements set forth in the 2007 California Building Code (CBC) Title 24 Part 2 Section 3415.

ASCE 41-06 "Seismic Rehabilitation of Existing Buildings" (ASCE 41) was utilized to perform the seismic evaluation of the building. The seismic assessment was based upon a prescribed Earthquake Hazard Level and a specified structural performance level per 2007 CBC Table 3415.5. Two performance objectives are required by the 2007 CBC: Level 1 and Level 2.

Table 1. Performance Criteria per 2007 CBC Section 3415

Performance Level	Hazard	Acceptance Criteria
Level 1	BSE-R (225 yr, 20%/50yrs)	Life Safety (S-3)
Level 2	BSE-C (975 yr, 5%/50yrs)	Collapse Prevention (S-5)

The Level 1 objective is to achieve Life Safety performance under an earthquake with a return period equal to 225 years. The Level 2 objective is to achieve Collapse Prevention performance

under an earthquake with a return period equal to 975 years. As referenced in the CBC, the evaluation was performed in accordance with ASCE 41. The evaluation was supplemented with information from FEMA 355D and Degenkolb/CALTRANS (See Kim et al 2003) testing related to similar beam-column connection assemblages.

EVALUATION METHODOLOGY

The preliminary results presented in Degenkolb's draft report dated January 2006 recommends using nonlinear response history analysis (NRHA) to determine whether the building would be adequate for the established acceptance criteria. Building on the results of the preliminary Degenkolb study, B&B established a mathematical model incorporating the nonlinear load-deformation characteristics of the individual components of the building. In this case, the most critical component of the building was the steel moment frame connection. The connections were similar to "Pre-Northridge" era called Welded Unreinforced Flange (WUF), with few, but notable, differences as discussed in the following sections. Although earlier research studies show strong evidence to suggest the WUF connection would provide acceptable seismic behavior, during the 1994 Northridge Earthquake, WUF connections performed poorly. After the earthquake, several publications pointed out the potential reasons behind the brittle behavior of these connections.

In order to incorporate the existing connection characteristics to the mathematical model, the individual components were evaluated to estimate the behavior of the connection. The existing connections were investigated using the existing weld procedures and existing connection detail drawings used for the original construction of the building. The connection load-deformation parameters were estimated through available literature that involves similar beam-to-column tests. The findings were compared with the component characteristics for Pre-Northridge connections that are given in ASCE 41 and FEMA 355D.

The model then was subjected to earthquake shaking represented by ground motion time histories in accordance with ASCE 41.

The components of the building were checked against Level 1 for life safety and Level 2 for collapse prevention performance criteria, per the requirements of the 2007 CBC.

Seismic Evaluation Site Parameters

Two response spectra were created at level BSE-R (20%/50yrs) and BSE-C (5%/50yrs) using ASCE 41 guidelines. Spectral acceleration values were first calculated per ASCE 7-05 using 2002 USGS maps. The acceleration values were adjusted to Soil Type D. The parameters were further modified from the 2%/50yrs basis of 2002 USGS maps to the levels needed for this study: 20%/50yrs and 5%/50yrs.

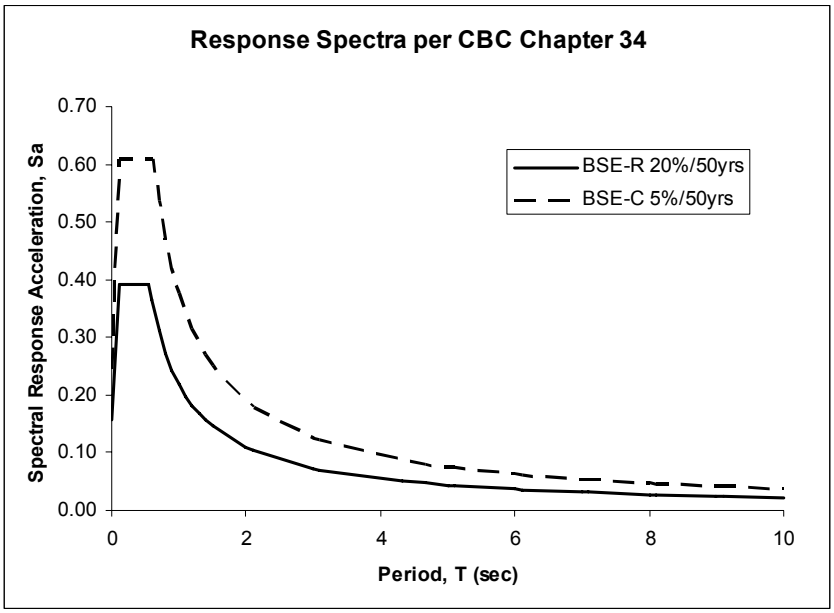


Figure 1. Response Spectra

The seismic demands on the building were determined by nonlinear response history analysis procedures using several ground motions. Procedures for selecting and scaling ground motion records for a site-specific hazard are described in ASCE 7-05 Chapter 16. The site-specific response spectra and matching ground motions used for this study were established by a geotechnical engineer for another building within a few hundred yards of BOE building site. Please refer to the geotechnical report by J.P. Singh & Associates from 2006 [1]. The site-specific response spectrum contained in the report fairly matched the BSE-C response spectrum (see Figure 2).

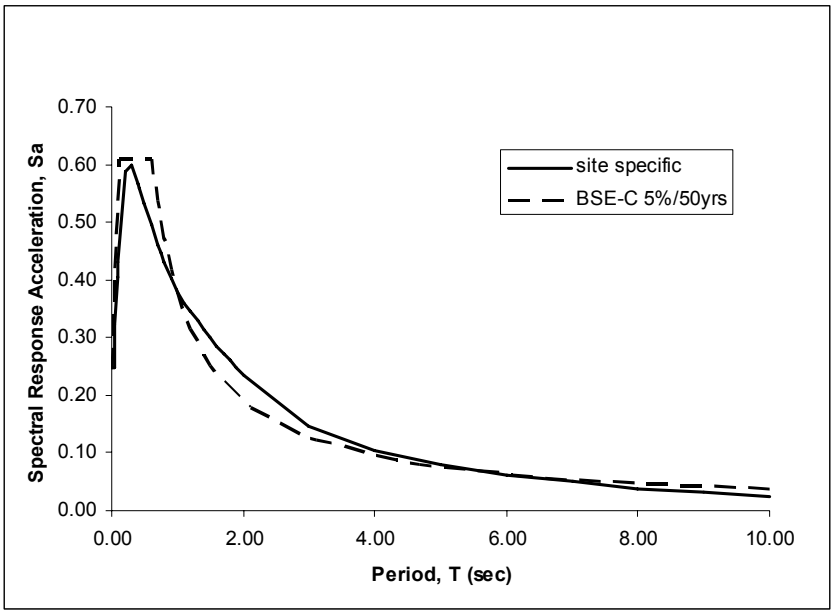


Figure 2. Response Spectra Comparison: Site-specific versus Code-based.

Figure 3 shows the response spectra of one of the pairs of scaled ground motions. As required by ASCE 7-05, the square root of the sum of the squares (SRSS) rule was applied to the pair to compare the SRSS amplitude with 1.3 times the site-specific response spectra. Similarly, all other 6 ground motions satisfied the criteria set forth in the ASCE 7-05.

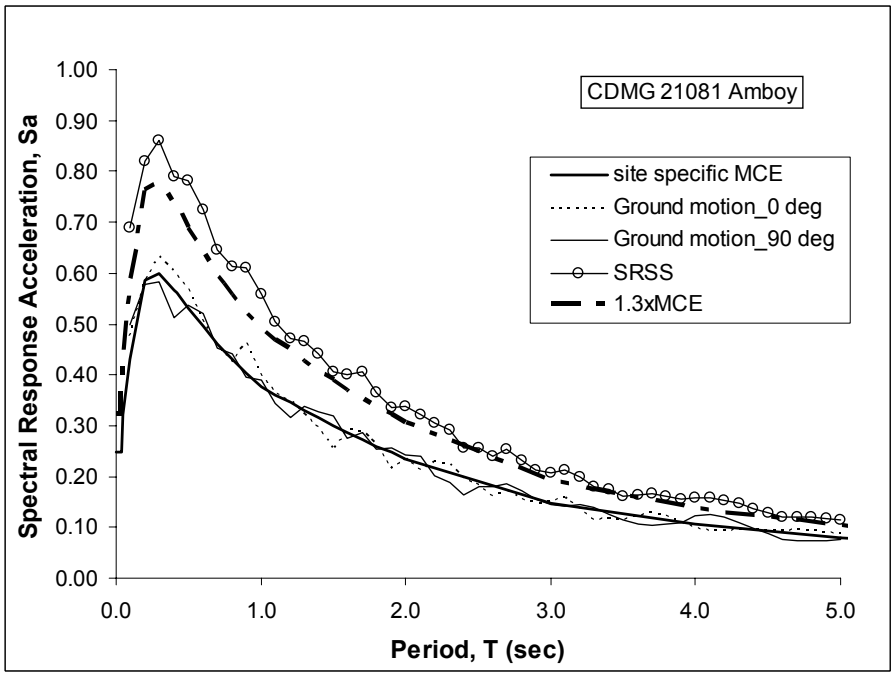


Figure 3. Spectral-matched Ground Motion Pairs per ASCE 7-05.

Table 2 shows the seven pairs of scaled ground motions provided in the geotechnical report that were used to estimate the seismic demand on the structure per ASCE 41 and ASCE 7-05 guidelines.

Table 2. Selection of Seed Time Histories for Developing Spectra Matched Time Histories

Earthquake	Magnitude	Magnitude Recording Station	Distance (km)
1987 Whittier Narrows	5.99	CDMG 24401 San Marino	15.9
1989 Loma Prieta	6.93	CDMG 4738 Gilroy #3	12.8
1989 Loma Prieta	6.93	CDMG 57382 Gilroy #4	14.3
1989 Loma Prieta	6.93	CDMG 57066 Agnews Hospital	24.5
1989 Loma Prieta	6.93	CDMG 47524 Hollister	27.9
1992 Landers	7.28	CDMG 21081 Amboy	69.2
1992 Landers	7.28	CDMG 22074 Yermo FS	23.6

"Pre-Northridge" Connections and Factors Affecting Their Performance

The welded unreinforced flange (WUF) connections have complete joint penetration (CJP) groove field welds connecting the beam flanges to the column, and a shear tab is shop-welded to the column with fillet welds and bolted to the beam web for transfer of shear force. As discussed next, the original design of the BOE building utilizes WUF connections with certain enhancements that improve the performance.

Effect of Weld Electrode

The type of weld filler metals used in the connections play an important role on connection performance. The filler metal with high toughness, as measured by Charpy V-Notch (CVN) test, is one of several contributing factors to achieve good connection behavior. Based on the existing original weld procedure documents, the flanges of the BOE building steel moment connections were welded with E70T-1 FCAW filler metal. This filler metal material meets a moderate toughness requirement. These types of filler metals are specified when notch-toughness is required above the 20ft-lbs at temperature ranges of -20°F to 0°F. Current moment frame connection standards require slightly higher toughness values. It should be noted that at the time of design and construction the "typical" practice of a Pre-Northridge connections was to use E70T-4 FCAW electrodes that did not meet a minimum required CVN toughness, and those electrodes have been shown to have low notch-toughness. This low notch-toughness is one of the factors shown to contribute to connection fracture. It has been well documented through full scale testing that using a higher CVN tough electrode improves the connection behavior. FEMA 355D lists results from beam-column tests (with W36 beams) both with and without notch-tough filler metals. The typical "Pre-Northridge" connection tests with E70T-4 (with W36x150 beams and W14x257 columns) showed little or even no ductility. The ones with high notch-tough

electrodes fractured at 0.009 radians and 0.017 radians. These values are still low ductility compared to today's standards and requirements, but they are better than "typical" Pre-Northridge connections. It is concluded that it is likely that the BOE building steel moment connection welds perform better that "typical" Pre-Northridge connection. As a result, use of E70T-1 electrode in the original construction will likely help the connection to reach 0.009 radian rotation.

Effect of specified versus actual beam strength

The connection performance depends on the relative strength between the beam, the column, and the panel zone. Therefore, it is important to identify the expected yield stress of the materials used in the structure to estimate the capacities of the steel sections. The properties of structural steel have changed significantly over the years. Until about the Northridge earthquake, mild structural steel was sold as ASTM A36 steel with a specified minimum yield stress of 36 ksi. The specified minimum yield stress is a statistical requirement, since the mean yield stress and the actual expected yield stress of the vast majority of specimens must be significantly larger than the minimum value. The seismic performance of a connection depends on the actual yield stress of the material used. By the mid 1980s, tests were showing the average yield stress for mild steel was nearly 47 ksi. By the time of Northridge earthquake, the expected yield stress was commonly larger than 50 ksi.

Based on these findings and using tables in AISC Seismic Provisions for Structural Steel Buildings 13th Edition [6], an expected yield strength value of 54 ksi was used in the nonlinear analysis of the BOE building where A36 rolled mild steel was specified.

Effect of Panel Zone Yielding

Panel zone yielding has been shown by researchers to provide considerable ductility in inelastic cyclic deformation. Panel zone yielding is a yield mechanism; it is not a failure mode. Experiments suggest that specimens designed to have weak panel zones may have less plastic rotational capacity than those specimens designed for a balanced yielding of the beam and the panel zone. At the same time, many experiments show that reduced ductility may be expected if the panel zone is so strong that no panel zone yielding occurs. The BOE building has ASTM A572 Grade 50 columns which mean the minimum panel zone yield stress is 50 ksi. No doubler plates were used in the building to increase the capacity of any panel zones. The typical beams consists of ASTM A36 material, however, as discussed in previous sections, the beams are likely to yield at 54 ksi (although specified as 36 ksi nominal) – generating a higher beam yield moment capacity. This may force the panel zone to yield before the beam flanges. The nonlinear response history analysis modeling allows for the appropriate levels of yielding occurring separately in the panel zone and in the beam.

Effect of Beam Depth

Less ductility is expected from deep beams than from shallow beams. The BOE building's lateral resisting steel moment frame beams are "deep" beams typically ranging from W33 to W36.

Plastic Rotation Capacity

Plastic rotation capacities of the moment connections at the moment-resisting frames were determined using FEMA 355D and Degenkolb/CALTRANS test results (See Kim et al 2003) along with ASCE 41-06.

FEMA 355D "State of the Art Report on Connection Performance"

For sections up to W36 (for steels with larger yield to tensile stress ratios), FEMA 355D suggests a mean plastic rotation capacity (θ_{pmean}) and standard deviation (σ_p) as follows:

$$\theta_{pmean} = 0.011 \text{ radians} \qquad \sigma_p = 0.007 \text{ radians}$$

The tests indicated in FEMA355D with notch-tough electrode show flange fracture at 0.009 radians. Therefore, where I-flange columns and beams occur at a connection, the plastic rotation corresponding to flange fracture was established as 0.009 radians. It should be noted that the plastic rotation capacity where a beam connects to a boxed column has a different capacity limit due to its lack of an effective panel zone, and that behavior is discussed under the CALTRANS tests section.

The test results in FEMA 355D indicated that when flanges fracture in the connection, the loads were redistributed to the shear tab. The moment capacity of the connection was significantly reduced. The rotational capacity and standard deviation of the connection after flange fracture was defined as follows:

$$\theta_{gmean} = 0.043\text{-}0.0006\ d_b \qquad \sigma_g = 0.011 + 0.0004\ d_b$$

Degenkolb/CALTRANS Tests

Ideally for a nonlinear, the force-deformation relationship of each component is determined experimentally. Therefore a literature search was performed. The full-scale testing performed for CALTRANS District 4 Headquarters Building in Oakland, California (CALTRANS Tests) was selected as a reference. The results of these tests were reported in detail at Kim, et al (2003) [3]. This study was selected for the similarities of sizes of the beams and the columns which were tested compared to the sizes of the BOE frame members. Furthermore, the type of moment connections tested was similar in nature with the BOE moment type connections. The selected tested specimens consisted of the following:

- Prototype EC01: a moment connection between a box column, BC 18x18x257, and a W33x118 beam. This is a corner column connection.
- Prototype EC03: a moment connection between a WTM 27x14x281column and a W36x210 beam. This is an intermediate frame connection.

Although Prototype EC01 and EC03 fairly represent the corner and middle framing conditions in the BOE framing, a few differences from the BOE were noted. The type of weld used in the test was reported as E70T-4, whereas the BOE connections used a better performing electrode called E70T-1 (see "Effect of Weld Electrode" section for discussion). Also, the tested beams were identified as ASTM A572 Grade 50, whereas the BOE beams consisted of A36 beams. As mentioned in the section "Effect of specified versus actual beam strength," the difference in yield strengths may not be significant between ASTM 572 Grade 50 and ASTM A36.

The loading protocol used in these tests was reported as "cyclic" loading and consisted of symmetric and stepwise-increasing displacements at the end of the beam.

The test results indicated poor performance of these connections. Prototype EC01 experienced fracture at the beam top flange at a story drift ratio of 0.76%. The beam bottom flange fracture occurred at 0.91%. The connection showed almost no plastic energy dissipation before fracture at the beam welds. Prototype EC03 exhibited fracture at the beam top flange and beam bottom flange at a story drift ratio of 0.58% and 1.38%, respectively. Fracture of the shear tab occurred approximately at a story drift ratio of 3%.

Nonlinear modeling for BOE building NRHA used these test results to calibrate the connection behavior.

Acceptance Criteria: Plastic Rotation Angle

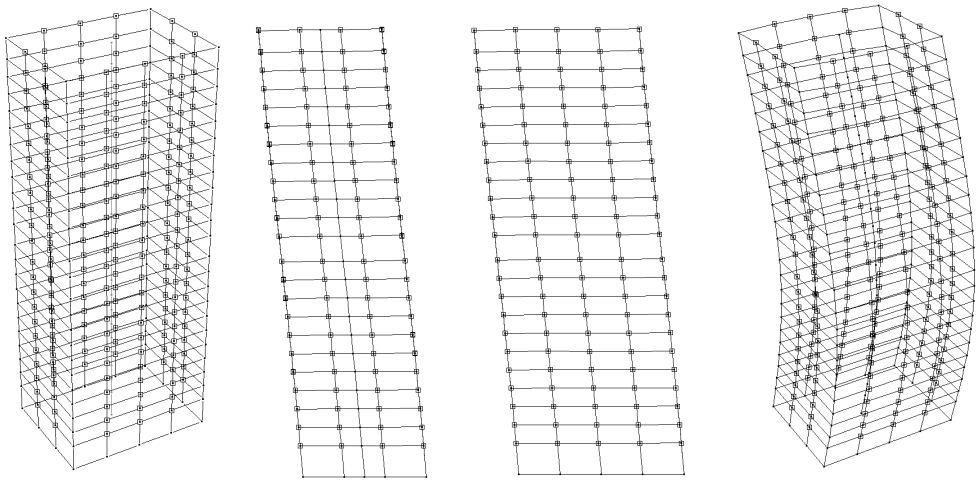
Acceptable performance limits for various components and actions were based on ASCE 41. Life safety (LS) and Collapse Prevention (CP) limits were determined using ASCE 41 Table 5-6. See Table 3 for a summary of acceptable plastic rotations for various components of the moment frame system.

Table 3. ASCE 41 Acceptable Performance Limits

Component	*LS*	*CP*
Panel Zone	12 $q_y \approx 0.032$ rad	12 $q_y \approx 0.032$ rad
Beam: W33	0.0323 – 0.00045d ≈ 0.016 rad	0.0430-0.00060d ≈ 0.021 rad
Beam: W36	0.0323 – 0.00045d ≈ 0.015 rad	0.0430-0.00060d ≈ 0.020 rad
Columns	1.2 q_y (conservative)	1.2 q_y (conservative)
Columns: WTM24x192 and WTM24x207	0.96 q_y (conservative)	0.96 q_y (conservative)

ANALYSIS RESULTS

Three-dimensional nonlinear response history analysis was performed using the Perform-3D (version 4) computer program. Inelastic properties of the beam, column, and panel zones were modeled. The masses were calculated for each story and uniformly distributed within the story diaphragm. The model incorporated P-delta effects and 25% of the design live load was included in the loading per the ASCE 41 guidelines. A modal analysis was done to evaluate the fundamental modal behavior of the structure. Figure 4 shows the first three model shapes of the lateral moment-resisting frames. The fundamental period was found to be 4.63 seconds in the North/South direction.



(a) (b) (c) (d)

Figure 4. Mode Shapes: (a) Undeformed, (b) First Mode, (c) Second Mode, and (d) Third Mode.

The structure was subjected to seven pairs of ground motions. Then the same pairs of ground motions were rotated 90 degrees and applied to the structure. The resulting maximum story drift drifts for each story are shown in Figure 5 and Figure 6. Per ASCE 41 and ASCE 7-05, the average of the maximum drift values at each story (from the seven separate analyses) were calculated and are presented in the figures as dashed lines. The story drifts shown on these figures indicate total rotation, including both elastic and plastic rotation contributions from elements. It was observed that the maximum total story drift rotation occurred near mid height of the structure with absolute maximum amplitude below 0.77% drift with an average drift of 0.64%.

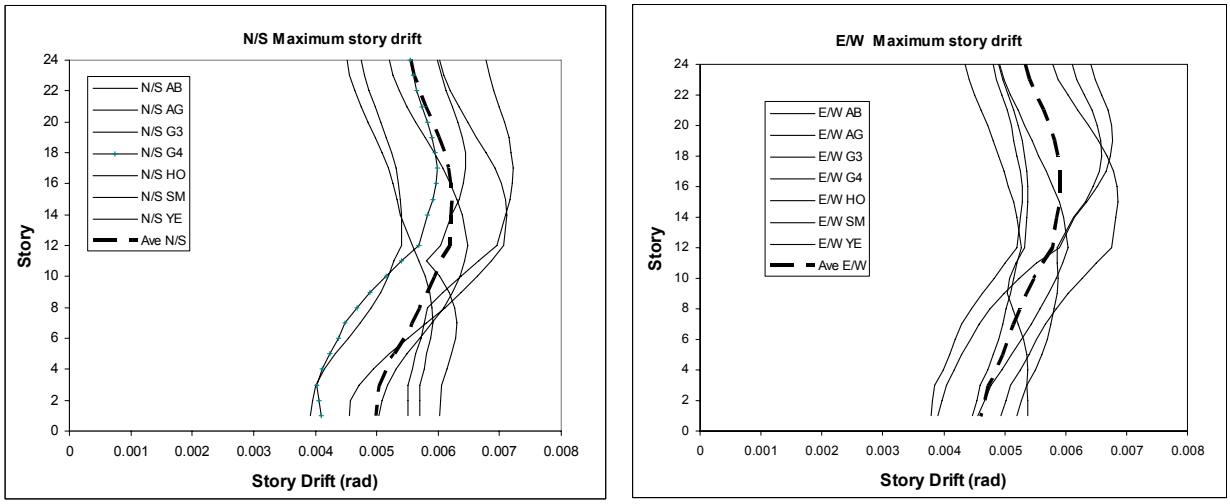


Figure 5. Maximum story drift in two principal directions

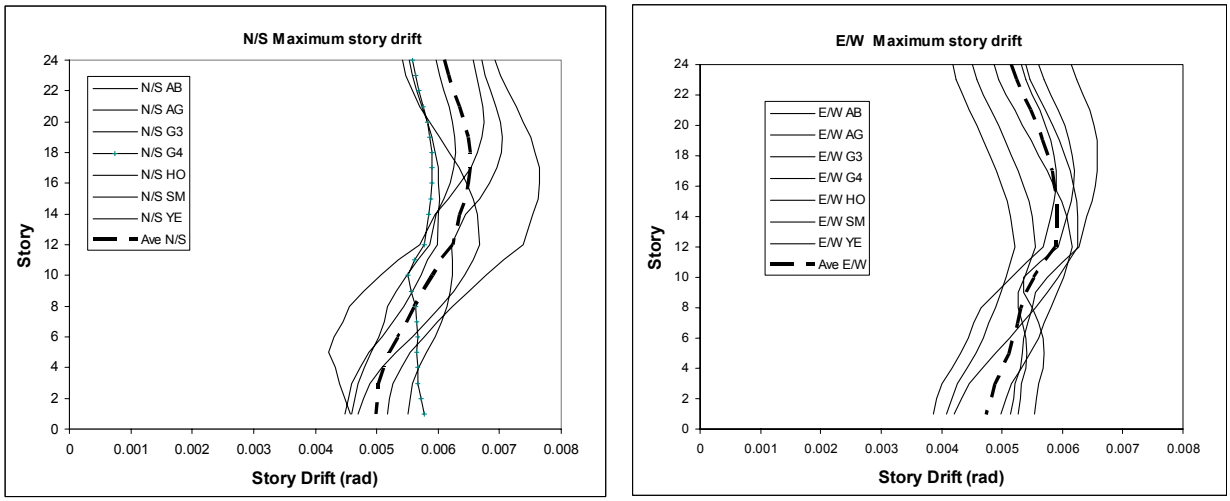


Figure 6. Maximum story drift in two principal directions – ground motion rotated 90deg

Although the resulting drift was found to be well below the acceptance criteria, the measured drift exceeded the bottom flange fracture capacity per the CALTRANS test data. The building response to 7 pairs of ground motions in 0 and 90 degrees (total 14 ground motions) resulted in fractured connections at several locations of the building. Results from one of the response history runs is shown in Figure 7. As indicated on the figure, the corner connections fractured before the intermediate connections.

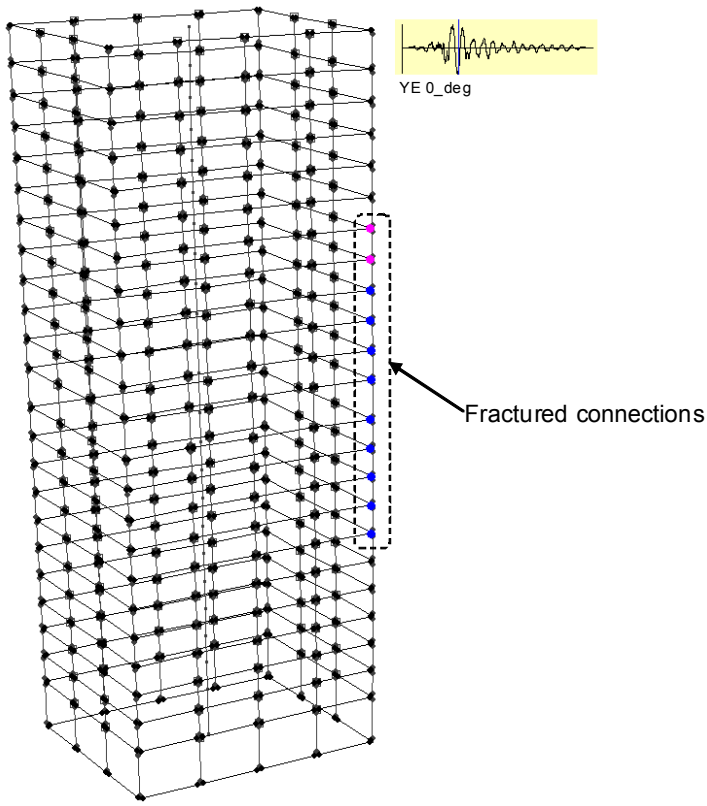


Figure 7. Deformed Shape with fractured connections

Figure 8 shows the fractured connection locations (indicated with a large circle) for all ground motions. The distribution of the fractured connections shows expected correlation with the higher mode behavior of the building. Also as expected, the corner moment frame connections sustained the most damage. Since the corner columns consist of heavy boxed shapes, there was no panel zone to contribute to the overall joint rotation. This resulted in concentration of the plastic rotation at beam ends and eventually getting an early fracture at the beam flanges where the weld occurs. As seen in the figure, the fractured corner connections occur from the 8th floor thru the 22nd floor.

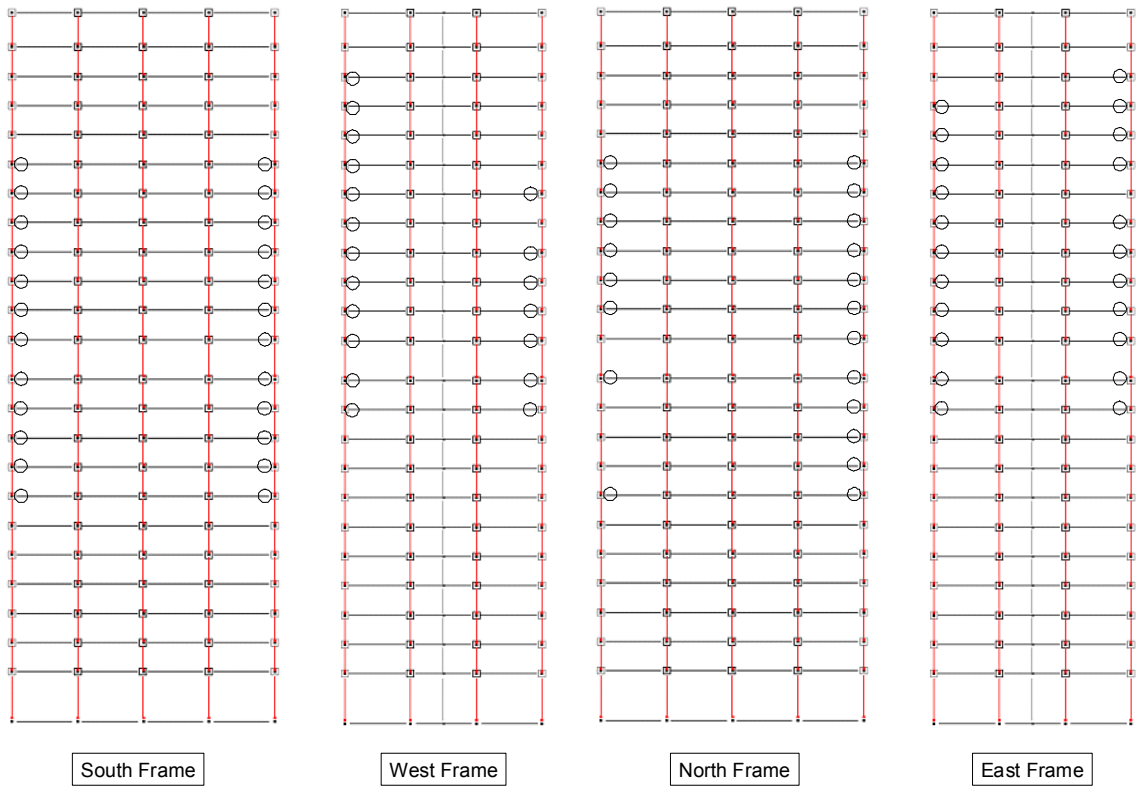


Figure 8. Connection fractures at beam flanges for all ground motions

The intermediate moment frame connections exhibited better performance than the ones at the corner due to rotation contribution from the panel zones as well as better capacity data from available tests. Although the wide flange column sections had relatively weak panel zones at these connections, all connections at the intermediate locations stayed below the fracture rotation limit.

Although the fractured connections at the corners lost approximately 80% of their strength capacities, the residual strength and the ductility from the shear tab showed enough reserve characteristics to satisfy both the Life Safety and Collapse Prevention acceptance criteria. Figure 9 shows a collection of all ground motion results indicating the maximum time history result of relative beam rotations ratio to beam flange fracture rotations. The values shown above 1.0 indicate fracture in the beam flange. As seen in the figure, in most ground motions demands, the building sustained further damage at the beam ends after beam flange fracture. The maximum rotation at the beam end relative to the beam flange fracture rotation stayed below 3.0. The reserve capacity of the intermediate moment frames helped delay the spreading of the connection fractures throughout the building.

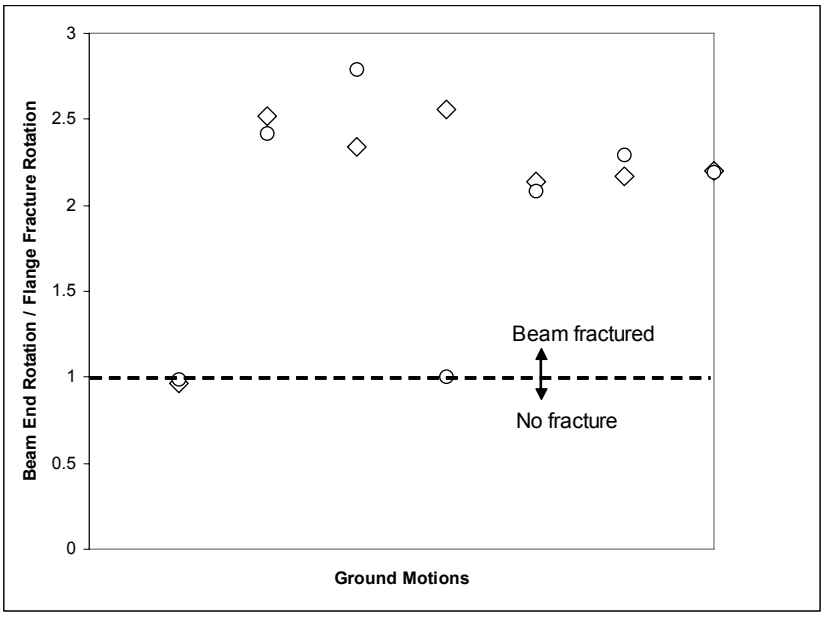


Figure 9. Time history plot of relative beam rotations divided by beam flange fracture rotations

Level 2 performance level requires achieving Collapse Prevention performance under a ground motion with 20% probability of exceedance in 50 years (BSE-C). Since only one level of ground motion data was available and since that data compared well with the BSE-C magnitude, the component rotations were conservatively compared with the Life Safety instead of Collapse Prevention acceptance criteria (see Figure 10). Figure 10 shows the Life Safety (LS) acceptance criteria ratio. For all ground motion cases, the maximum ratio stayed below 1.0. Therefore, the building actually satisfies the Life Safety performance level for the BSE-C scenario.

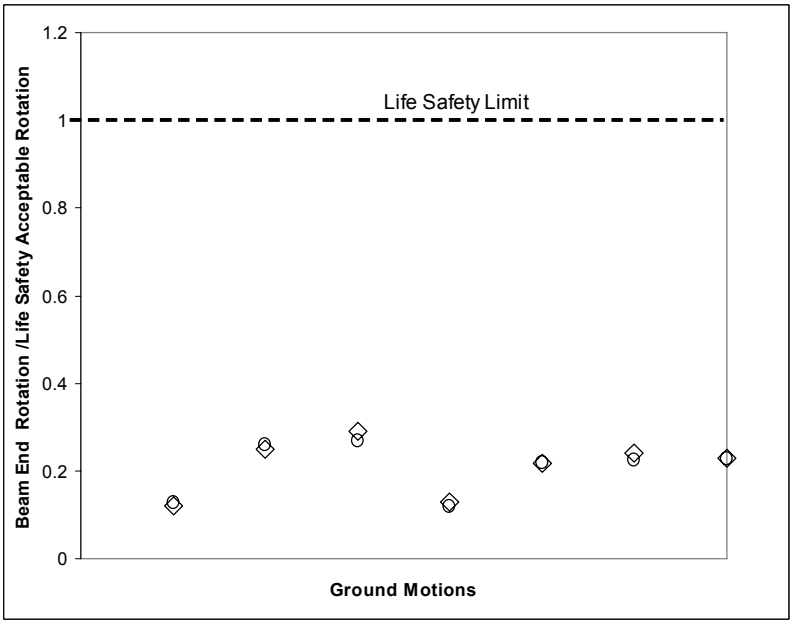


Figure 10. Life Safety (LS) acceptance criteria ratio

Most of the panel zones remained elastic during the response history demand. There are few locations where yielding was observed. Figure 11 shows the panel zone yield shear strain ratio. The values shown above 1.0 indicate panel zone yielding. It should be noted that if ground motions scaled to the MCE-based response spectrum were to be used, then the demand to the members would be higher. This increase would end up with yielding most of the panel zones. However, per CBC Chapter 34 requirements, a reduced demand was used (5%/50yrs compared to traditional MCE of 2%/50yrs). Therefore, this reduced demand resulted in fewer number of yielded panel zones.

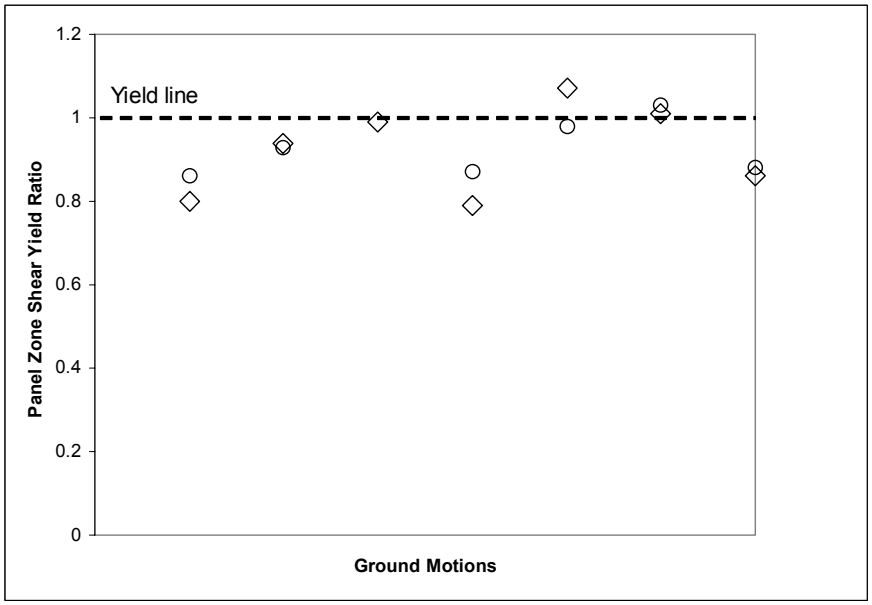


Figure 11. Panel Zone Shear yield strain ratio

Columns satisfied the acceptance criteria as set forth in ASCE Table 5-6 in this report. Additional detailed discussions can be found in Degenkolb's report regarding the column splices and anchorage of columns to the foundations. It was stated in that report that these connections were found conservatively sufficient. These results are neither investigated nor discussed in this evaluation. For pile foundation design for uplift and downward forces, also refer to Degenkolb report.

CONCLUSION

This report provides results of a seismic evaluation of the Board of Equalization Building at 450 N St Sacramento, California, to supplement preliminary studies performed by Degenkolb Engineers dated 2006. The scope of this study is to evaluate the seismic performance of the 24-story tower's lateral force resisting system by conducting a three-dimensional nonlinear response history analysis (NRHA). The impetus of evaluation using NRHA is due to the presence of "Pre-Northridge" steel moment frame connections. The lateral steel moment frame system was modeled in a three-dimensional nonlinear response history analysis using PERFORM 3D software. The connection behavior was calibrated using FEMA355D, ASCE 41-06 and existing testing results from a Degenkolb/CALTRANS study. The structure was subjected to seven pairs of scaled ground motions using a geotechnical report for a nearby site.

The building response to the seven pairs of ground motions resulted in fractured connections at several locations of the building. The corner moment connections above the 8th floor and below the 22nd floor suffered beam flange fractures. The intermediate moment frame connections exhibited better performance than the ones at the corner. This difference was due to rotation contribution from the panel zones as well as better capacity data from available tests.

The fractured connections at the corners lost approximately 80% of their strength capacities. The moment frames at the intermediate bays survived without fracture. The remaining 20% residual strength of the corner connections and the available ductility capacity from the shear tab showed enough reserve characteristics to satisfy the Life Safety and Collapse Prevention acceptance criteria for the BSE-C hazard defined in Chapter 34 of the 2007 CBC.

The results of the evaluation suggest that the building meets the 2007 CBC Level 1 and Level 2 performance criteria. According to the nonlinear response history analysis results, strengthening of the moment resisting frame connections is not required.

REFERENCES

[1] JP Singh & Associates, "Geotechnical Investigation and Seismic Design Criteria 6th and L Street, Sacramento, California" Job No: 157.001.01, August 16, 2006.

[2] Degenkolb, "Seismic Hazard Evaluation, Board of Equalization Building 450 N St, Sacramento, California, 90% Draft Report" January 2006, Degenkolb Job Number A5123024.00.

[3] Kim, T., Stojadinovic, B., Whittaker, A., S., 2003 "Physical and Numerical Performance Evaluation of Steel Moment Resisting Frames. Part 1: Existing Building Connection Studies" Report to California Department of Transportation, Sacramento, California.

[4] Federal Emergency Management Agency (FEMA), FEMA 350, 2000, "Recommended Seismic Design Criteria For New Steel Moment-Frame Buildings".

[5] Federal Emergency Management Agency (FEMA), FEMA 355D, 2000, "State of the Art Report on Connection Performance".

[6] American Institute of Steel Construction (AISC), AISC 360-05, 2005. "Steel Construction Manual, 13th Edition" Chicago, IL.

[7] American Society of Civil Engineers (ASCE), ASCE 7-05, 2005, "Minimum Design Loads for Buildings and Other Structures", Reston, VA.

[8] American Society of Civil Engineers (ASCE), ASCE 41-06, 2006, " Seismic Rehabilitation of Existing Buildings"

[9] 2007 California Building Code (CBC), California Code of Regulations, Title 24, Part 2, California building Standards Commission.