INDEPENDENT SEISMIC REVIEW

of

Occidental Petroleum Building & Hammer Museum

10889 & 10899 Wilshire Boulevard
Los Angeles, CA

Prepared for:

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(source: King of Hearts)

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0.0 EXECUTIVE SUMMARY

This report presents the results of the independent seismic review of the 15-story Occidental Petroleum Building and the 3-story Hammer Museum located at 10889 and 10899 Wilshire Boulevard in Los Angeles, California. The buildings were evaluated for the ability to meet a seismic performance objective of substantial life safety in accordance with the University of California (UC) Seismic Safety Policy requirements for acquisitions by purchase.

The buildings are constructed adjacent to each other and occupy an entire city block bordered by Wilshire Boulevard to the south, Lindbrook Drive to the north, Glendon Avenue to the east and Westwood Boulevard to the west. The Occidental office building has 15-stories above-grade, a mechanical penthouse and two levels of subterranean parking. The building was constructed in 1961 and likely designed to the 1955 edition of the Uniform Building Code (UBC). The building was seismically strengthened in 1992 when a single-bay reinforced concrete shear wall was added in the east-west direction from the ground floor to the underside of the 3rd floor.

The museum has 3-stories above grade and four levels of subterranean parking. The museum was constructed in 1990 and designed to the 1985 edition of the City of Los Angeles Building Code. The museum is located to the north of the Occidental office building, separated by a 5" seismic joint.

Both buildings are of steel frame construction with metal deck and concrete fill roof and floors. The steel framing consists of wide flange steel beams, girders and columns. The foundation system of the Occidental office building consists of concrete tapered piles, pile caps and grade beams. The foundation of the museum consists of reinforced concrete spread footings and grade beams.

The lateral system of the Occidental tower consists of a combination of perimeter and interior steel moment frames with semi-rigid moment connections, interior concrete shear walls and perimeter concrete pier-spandrel system. The pier-spandrel system and the perimeter steel moment frames along the south, east and west elevations terminate at the third floor. Seismic forces in the north-south direction are transferred to the interior concrete shear walls that are continuous to the foundation. Seismic forces in the east-west direction are transferred to a long concrete shear wall along the north elevation of the building and the single-bay shear wall, added in 1992, along the south building elevation. The concrete shear walls are continuous to the foundation.

The lateral system of the Hammer museum consists primarily of reinforced concrete shear walls along the perimeter. There is a large opening in the roof and Gallery level diaphragm due to the courtyard.

The evaluation was performed in accordance with University of California Seismic Safety Policy requirements. The expected seismic performance of the buildings was determined by a limited site review of the structures, review of structural drawings, linear elastic and nonlinear dynamic analyses and a general seismic hazard analysis for the region.

The subject property is located in a region traversed by several active faults that are capable of producing moderate to large magnitude earthquakes. The closest major known surface fault to the building site is the Santa Monica Fault, which is approximately 0.6 mile away. The site is not located within a geologic zone where surface rupture, landsliding or liquefaction is likely to occur.

In accordance with the UC Seismic Policy for seismic rating III, a two tier evaluation was performed using the following performance criteria:

Evaluation Tier	Earthquake Hazard Level	Structural Performance Level	Nonstructural Performance Level
1	BSE-1 (10/50 – 475 yr)	Life Safety	Hazard Reduced
2	BSE-2 (2/50 – 2475 yr)	Collapse Prevention	Not Considered

<u>Hammer Museum:</u>

The following deficiencies were identified based on the results of the analysis:

- Pre-Northridge welded moment connections with limited ductility.
- Beams and columns of moment frame do not satisfy strong column/weak beam configuration. Strong column/weak beam configuration promotes yielding of the beam – a preferred behavior.
- The moment frame beams are connected to the weak axis of the column at many locations.

Despite these deficiencies, the results of the analysis indicate that the moment frames satisfy the Life Safety and Collapse Prevention performance criteria at BSE-1 and BSE-2 earthquake hazard levels, respectively. This is due to the stiffness of the perimeter concrete walls that attract the majority of the seismic forces. The steel moment frames ultimately function more as chord elements at the diaphragm discontinuity.

<u>Occidental Tower:</u>

Due to the hybrid nature of the seismic system (multi-system with different vulnerabilities and discontinuities), a nonlinear 3D computer of the building was developed to better capture the actual behavior of the building in an earthquake.

Nonlinear history response analyses were performed to capture higher mode and degradation effects that are not adequately accounted for in linear analyses. For these analyses, seven pairs of spectrally matched ground motion time history records for the BSE-2 earthquake hazard level were used.

The results of the nonlinear history response analyses of the existing building identified inadequate shear strength of concrete walls in north-south direction as the principal structural deficiency of the building. The results also indicate the formation of story mechanisms at the third through seventh floors. Based on these results, the existing building does not satisfy the requirements for UC seismic rating level III.

The building was also evaluated for UC seismic rating level IV, by applying a scaling factor to the matched ground motions to approximate the 975-year seismic hazard (BSE-C) for the site. The results indicate marginally acceptable performance. However, given the fact that the ground motions used was amplitude scaled and not spectrum matched, it is our professional opinion that the building satisfies the UC seismic rating level IV requirements. Further development of ground motions for the 225-year and 975-year seismic hazards is recommended and associated analysis peer reviewed to confirm compliance with seismic rating level IV.

A conceptual seismic strengthening scheme was studied and evaluated to mitigate the identified deficiencies and improve the building performance to UC seismic rating level III. The recommended strengthening scheme consists of:

- Strengthening existing interior concrete walls, located adjacent to the elevator banks, with fiber reinforced mesh from the foundation to the underside of the 12th floor; and
- Add new single-bay interior buckling restrained brace (BRB) frames at the east and west perimeter of the building from the foundation to the roof.

A seismic risk assessment considering building stability, site stability, seismic ground motion hazard and building damageability was performed for the existing complex. The on-line seismic risk assessment tool *SeismiCat,* developed by ImageCat, Inc., for screening of buildings for seismic risk, was used.

The Scenario Expected Loss (SEL) for ground shaking hazards having 10% probability of exceedance within a 50-year exposure period (BSE-1) was calculated. The SEL for the complex is 18%.

1.0 INTRODUCTION

1.1 General

This report presents the results of the independent seismic review of the 15-story Occidental Petroleum Building and the 3-story Hammer Museum located at 10889 and 10899 Wilshire Boulevard in Los Angeles, California. Figure 1.1 shows a vicinity map of the site.

Figure 1.1 - Vicinity map

The evaluation was performed in accordance with University of California Seismic Safety Policy requirements. The expected seismic performance of the buildings was determined by a limited site review of the structures, review of structural drawings, linear elastic dynamic analyses and a general seismic hazard analysis for the region.

A description of the construction of the buildings is provided in Section 2. Observations from the site visit are provided in Section 3. The likelihood of earthquake-induced site failure is discussed in Section 4. The criteria used in the evaluation of the buildings, the modeling and analysis assumptions, and a summary of the analytical results are discussed in Section 5.

This evaluation of the structural system represents the opinion of *Nabih Youssef Associates (NYA)* based on the available information. This review is not intended to preempt the responsibility of the original design consultants.

1.2 Evaluation References

The following documents and available information were examined in the evaluation:

- Structural drawings for *Kirkeby Center* as prepared by Brandow & Johnston, dated November 8, 1960.
- Architectural drawings for *Kirkeby Center* as prepared by Claude Beelman & Associates, dated November 8, 1960.
- Structural drawings for *Armand Hammer Museum of Art & Cultural Center* as prepared by John A. Martin & Associates, dated February 1, 1989.
- Structural drawings for *Occidental Petroleum Building* as prepared by John A. Martin & Associates (6305), dated June 5, 1991.
- Report of Ground Motion Hazard Analysis 10899 Wilshire Boulevard, Geocon West, Inc. (A9216-06-01), August 25, 2015.
- Report of Geotechnical Investigation Proposed Armand Hammer Museum, LeRoy Crandall and Associates (AE-88055), March 29, 1989.
- *Seismic Evaluation and Retrofit of Existing Buildings*, American Society of Civil Engineers, 41-13, 2013.
- *California Code of Regulations, Title 24, Part 2, Volume 2*, California Building Standards Commission, 2013.
- *University of California, Seismic Safety Policy*, August 25, 2011.
- State of California Seismic Hazard Zone, Beverly Hills Quadrangle, County of Los Angeles, March 25, 1999.

2.0 BUILDING DESCRIPTION

2.1 General

The Occidental Petroleum building and the Hammer Museum are constructed adjacent to each other and occupy an entire city block bordered by Wilshire Boulevard to the south, Lindbrook Drive to the north, Glendon Avenue to the east and Westwood Boulevard to the west. The Occidental building has 15-stories above-grade, a mechanical penthouse and two levels of subterranean parking. The building was constructed in 1961 and likely designed to the 1955 edition of the Uniform Building Code (UBC). The building was seismically strengthened in 1992 when a single-bay reinforced concrete shear wall was added in the east-west direction from the ground floor to the underside of the 3rd floor.

The museum has 3-stories above grade and four levels of subterranean parking. The museum was constructed in 1990 and designed to the 1985 edition of the City of Los Angeles Building Code. The museum is located to the north of the Occidental building. Figure 2.1 shows an aerial view of the buildings.

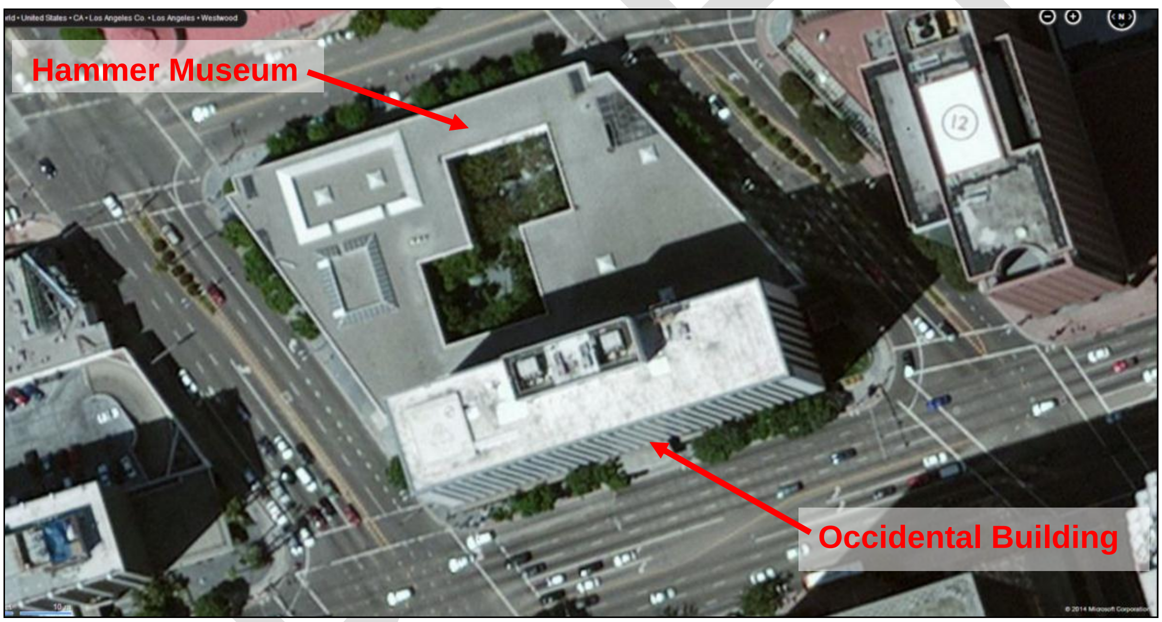

Figure 2.1 – Aerial view of building

The Occidental building is rectangular in-plan with overall dimensions of approximately 251′-8″ by 63′-0″. The 2nd floor is a partial floor in-plan creating a tall-story at the southeast corner of the 1st floor. The floor-to-floor height of the typical floor is 13′-0″. The 2nd and 15th floors have floor-to-floor height of 12′-0″ and 15′-6″, respectively.

The museum is irregular shaped in-plan with a large courtyard and overall dimensions of approximately 340′ by 182′. The floor-to-floor height of the P1, Garden and Gallery levels is 7′-9″, 17′-6″ and 22′-0″, respectively. A 5″ wide joint separates the museum and office building.

2.2 Gravity System

Occidental Tower:

- The penthouse and main roof are constructed of metal deck with 4″ lightweight concrete fill that span to wide flange steel beams and girders.
- The typical floors are constructed of metal deck with 2½″ lightweight concrete fill that span to wide flange steel beams and girders.
- The upper parking level and 1st floor are constructed of 4½″ one-way reinforced concrete slab that span to wide flange steel beams and girders.
- The wide flange steel girders that are supported by wide flange steel columns that are typically continuous to the foundation.
- The foundation system consists of Raymond step-taper piles with reinforced concrete pile caps and grade beams.

Hammer Museum:

- The roof and floors are constructed of 3″ metal deck with 3¼″ to 5″ lightweight concrete fill that span to wide flange steel beams and girders.
- The wide flange steel girders that are supported by wide flange steel columns that are typically continuous to the foundation.
- The foundation system consists of reinforced concrete spread footings and grade beams. A 6″ thick concrete slab on grade forms the P5 level.

2.3 Lateral System

Occidental Tower:

- The metal deck with concrete fill roof and floors act as structural diaphragms to transfer seismic forces to a combination of perimeter and interior steel moment frames with semi-rigid moment connections, interior concrete shear walls and perimeter concrete pier-spandrel frames.
- The semi-rigid moment connections consist of steel tee-shapes bolted to the top and bottom flange of the beams and the flange of the columns. Figure 2.2 shows a detail of the semi-rigid moment connection.
- The pier-spandrel frames are supported by, but eccentric to the steel framing. The concrete spandrels are connected to the wide flange steel beams with ⅞″ diameter steel J-bolts. The concrete piers are connected to the wide flange steel columns by steel stirrups that wrap around the steel column and are embedded in the pier. Figure 2.3 shows the detail of the spandrel to steel beam connection. Figure 2.4 shows the detail of the pier to steel columns connection.
- The pier-spandrel frames and the perimeter steel moment frames along the south, east and west elevations terminate at the third floor. Seismic forces in the north-south direction are transferred to the interior concrete shear walls. Seismic forces in the east-west direction are transferred to a long concrete shear wall along the north elevation of the building and the single-bay shear wall, added in 1992, along the south building elevation. The concrete shear walls are continuous to the foundation.

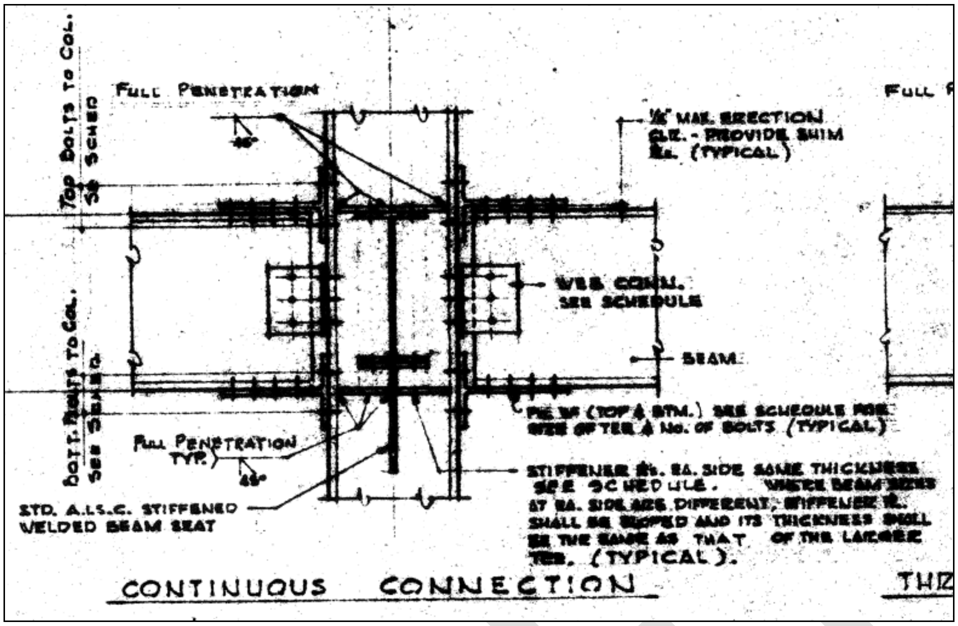

Figure 2.2 – Detail of semi-rigid moment connection

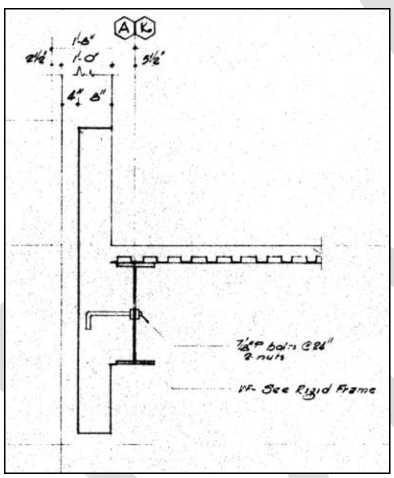

Figure 2.3 – Detail of concrete spandrel to steel beam connection

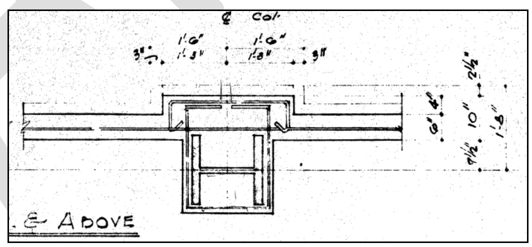

Figure 2.4 – Detail of concrete pier to steel column connection

Hammer Museum:

- The metal deck with concrete fill roof and floors act as structural diaphragms to transfer seismic forces to perimeter reinforced concrete shear walls that are continuous to the foundation.
- The roof and Gallery level diaphragms have large openings for the courtyard. Limited steel moment frames and collectors/chords have been provided along the perimeter of these openings.

3.0 FIELD OBSERVATIONS

3.1 General

A site visit was made on June 18, 2014. The majority of the structural components was covered and was not visually observable. Observation was limited to the visible areas of the structures. Photographs were not allowed.

3.2 Structural Observations

- In general, the building appeared to be in good condition; there were no signs of significant structural cracking, spalling or deterioration of the structural framing.
- No permanent offset of the building that would indicate structural distress was observed.

3.3 Nonstructural Observations

- No potential falling hazards that pose a life-safety threat to building occupants were observed in occupied or common areas within the buildings.

4.0 EARTHQUAKE INDUCED SITE FAILURE

4.1 Geologic Hazard

The likelihood of earthquake-induced site failure is discussed below. An extensive report on the seismic hazards for this area has been published in *Seismic Hazard Report for the Beverly Hills 7.5-Minute Quadrangle, Los Angeles County*.

The natural soils underlying the site consists of alluvial fan deposits the non-marine Lakewood Formation consisting primarily of cohesive clays and silts with some silty sand and clayey sand. Based on site specific soil shear wave velocity for the upper 30 meters of soil (Vs30) of 318 meters/second, the soil at the site is classified as Site Class "D".

4.1.1 Ground Fault Rupture

Ground fault rupture is the direct manifestation of the movement along a fault, projected to the ground surface. It consists of a concentrated, permanent deformation of the ground surface, which in major earthquakes can extend many miles along the trace of the fault. This deformation can be in either horizontal and/or vertical direction. A ground-surface rupture involving more than a few inches of movement within a concentrated area can result in major damage to structures that cross it.

The subject building is not located at a site subject to the jurisdiction of the Alquist-Priolo Special Studies Zone Act (this Act prohibits the location of most structures for human occupancy across the traces of active faults and thereby mitigates the hazard of fault rupture). Further, the closest identified active fault to the site is the Santa Monica fault, which is approximately 0.6 mile away. The potential for ground surface rupture is low.

4.1.2 Landsliding

A landslide is the downhill movement of masses of earth under the force of gravity. Earthquakes can trigger landslides in areas that are already landslide prone. Landslides are most common on slopes of more than 15 degrees and can generally be anticipated along the edges of mesas and on slopes adjacent to drainage courses.

The subject building is located on a gently sloping piece of land and examination of the *Beverly Hills Seismic Hazard Zone Map* indicates that the site is not located in an area recognized by the State of California where historic occurrence of landslide movement, or local topographic, geological, geotechnical and subsurface water conditions indicate a potential for permanent ground displacement. Therefore, the potential for landsliding is very low.

4.1.3 Liquefaction

Liquefaction is the sudden loss of bearing strength that can occur when saturated, cohesionless soils (sands and silts) are strongly and repetitively vibrated. Damage from liquefaction results primarily from horizontal and vertical displacement of the ground. These displacements occur because sand/water mixtures in a liquefied condition have virtually no strength and provide little or no resistance to compaction, lateral spreading, or down slope movement. This movement of the land surface can damage buildings, and buried utilities, such as gas mains, water lines and sewers, particularly at their connection to the building.

The geotechnical report indicates that the natural soils are dense and stiff and are not subject to liquefaction.

5.0 BUILDING PERFORMANCE IN EARTHQUAKES

5.1 Evaluation Criteria

The University of California (UC) policy requires that all buildings considered for purchase must be evaluated for the ability to meet a seismic performance objective of substantial life safety. As such, the UC Seismic Safety Policy requires buildings acquired by purchase to have an earthquake damageability of Level III or better. The University may purchase a building rated at Performance Level IV provided a seismic retrofit, and an estimate of the cost, is developed to achieve a Performance Level rating of III. By definition, Level III seismic performance is equivalent to the performance of Occupancy Category I-III for existing buildings as established in Chapter 34 of the 2013 California Building Code (CBC). The CBC uses, by reference, the methodology and procedures of ASCE 41, *Seismic Evaluation and Retrofit of Existing Buildings*. ASCE 41 is a national standard for the seismic evaluation and retrofit of existing buildings.

The buildings were evaluated in accordance with Section 3417 of the 2013 CBC with modified earthquake hazard levels per the UC Seismic Safety Policy. A two tier evaluation was performed using the performance criteria specified in Table 3417.5 for Occupancy Categories I-III. The criteria used are presented in Table 5.1.

Table 5.1 Seismic Performance Criteria for Seismic Rating III

Evaluation Tier	Earthquake Hazard Level	Structural Performance Level	Nonstructural Performance Level
1	BSE-1 (10/50 – 475 yr)	Life Safety	Hazard Reduced
2	BSE-2 (2/50 – 2475 yr)	Collapse Prevention	Not Considered

5.2 Seismic Hazard

The response acceleration parameters for the BSE-1 and BSE-2 earthquake hazard level from USGS, adjusted for the site soil conditions, are:

Earthquake Hazard Level	S_{XS}	S_{X1}
BSE-1 (10/50 – 475 yr)	1.49g	0.82g
BSE-2 (2/50 – 2475 yr)	2.24g	1.24g

A conditional mean spectrum for the BSE-2 hazard was also developed for the site. Due to the close proximity of faults that govern the seismic hazard at the site, fault normal and fault parallel spectra were developed. Figure 5.1 shows a plot of the general and conditional mean response spectra for the site.

The conditional mean spectra were used to spectrally match recorded ground motion "seed" records. These matched time histories were used to evaluate the seismic performance of the Occidental Tower building. Seven pairs of recorded ground motion records were selected and are listed below:

- 1989 Loma Prieta, Saratoga – West Valley College
- 1994 Northridge, 17645 Saticoy Street
- 1994 Northridge, Rinaldi Receiving Station

- 1994 Northridge, Sylmar Converter Station
- 1999 Chi-Chi, Taiwan, TCU065
- 1999 Duzce, Turkey, Bolu
- 2010 Darfield, New Zealand, HORC

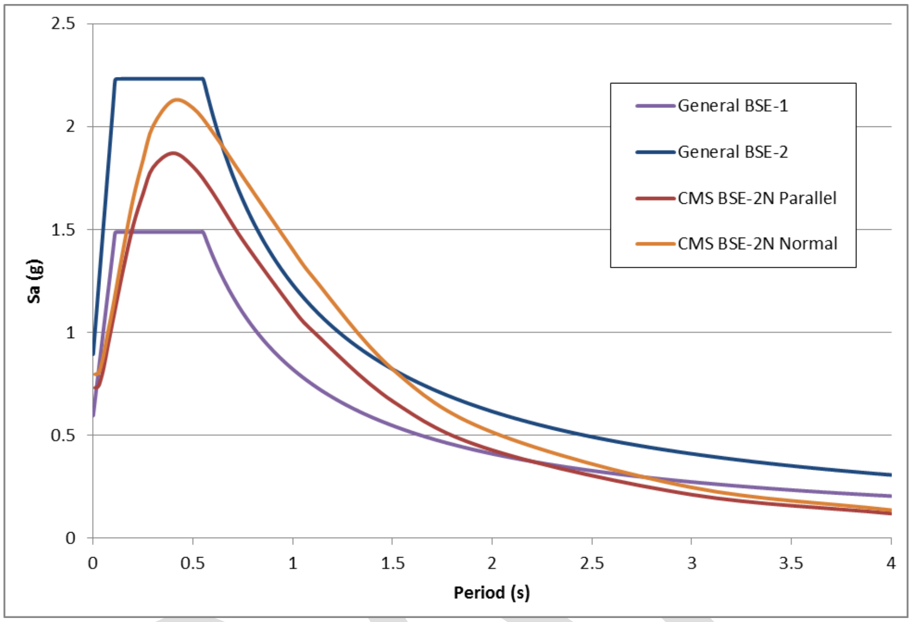

Figure 5.1 – Plot of general and conditional mean response spectra for the site

Figures 5.2 and 5.3 shows a plot of the fault normal target and seed record spectra, and fault normal and parallel, target and average of matched seed record spectra, respectively.

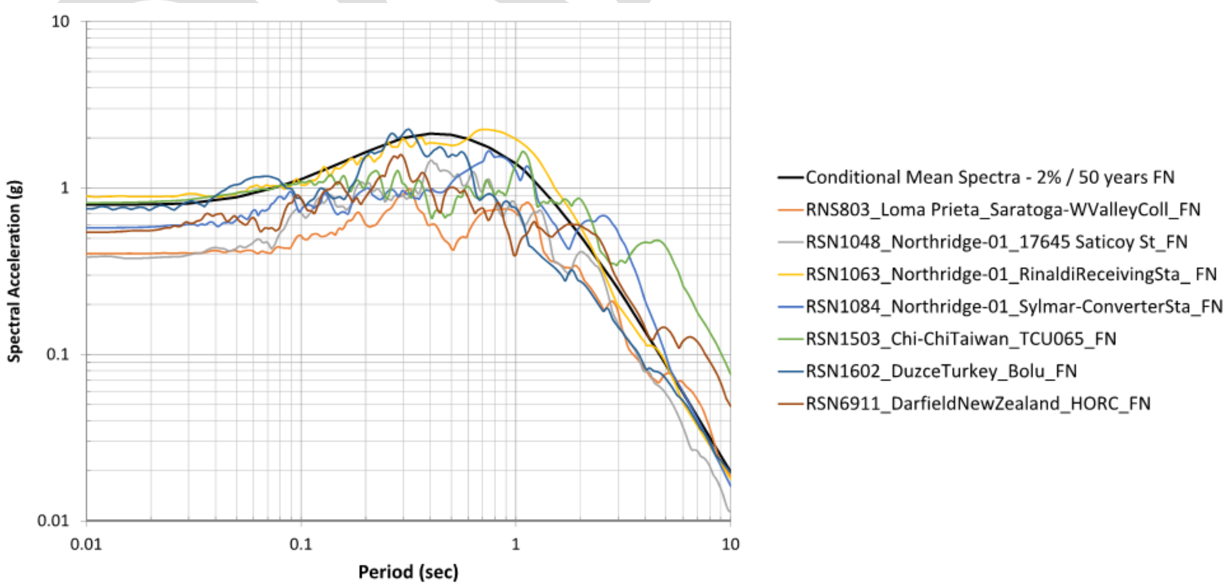

Figure 5.2 – Plot of fault normal target and seed record spectra

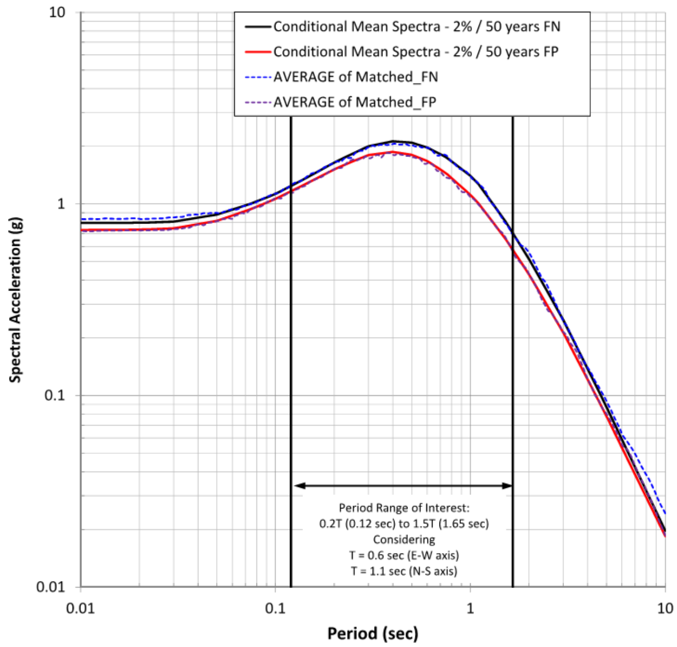

Figure 5.3 – Plot of fault normal and parallel, target and averaged matched seed spectra

5.3 Analysis and Modeling Assumptions

Occidental Tower:

The nonlinear dynamic procedure of ASCE 41 was used to analyze the seismic performance of the Occidental Tower building. A three-dimensional computer model of the building was developed using *Perform-3D*, developed by Computers & Structures, Inc. The program is capable of performing nonlinear analysis and performance assessment of buildings.

The seismic base of the building was assumed to be at street level. The model included all elements that significantly contribute to the lateral force resistance of the building, these include the steel beams and columns of the tower moment frames with semi-rigid moment connections, the concrete walls, and concrete piers and spandrels. Figure 5.4 shows a plot of the model.

The steel beams and columns, and concrete spandrels are modeled as elastic frame elements with flexural hinges at their ends. The concrete piers are modeled as elastic frame elements with flexural and shear hinges. The concrete walls are modeled as shell elements with elasto-plastic shear properties and fiber elements to capture the contribution of the steel reinforcement. The hinge properties were determined using the procedures of ASCE 41, Supplement No. 1.

The floor and roof levels were assumed to act as a rigid diaphragm. Story and roof masses were lumped at the story/roof center of mass. The self-weight of the structural members was included in the model. Additional dead and live loads were assigned as point loads in the model.

Modal analysis was performed to determine the dynamic response characteristics of the building. The fundamental periods of vibration for the building are summarized in Table 5.2. Figures 5.5 through 5.7 show the plots of the mode shapes.

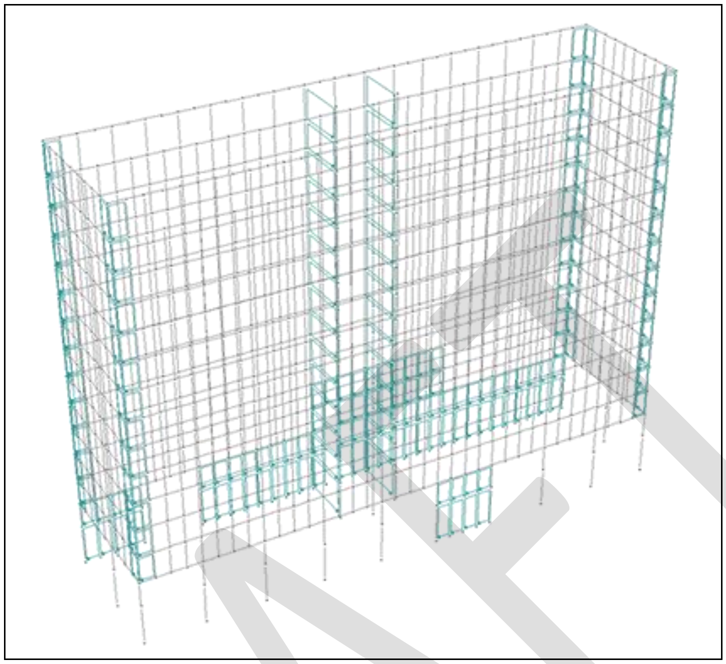

Figure 5.4 – Plot of Perform-3D model of Occidental Tower building

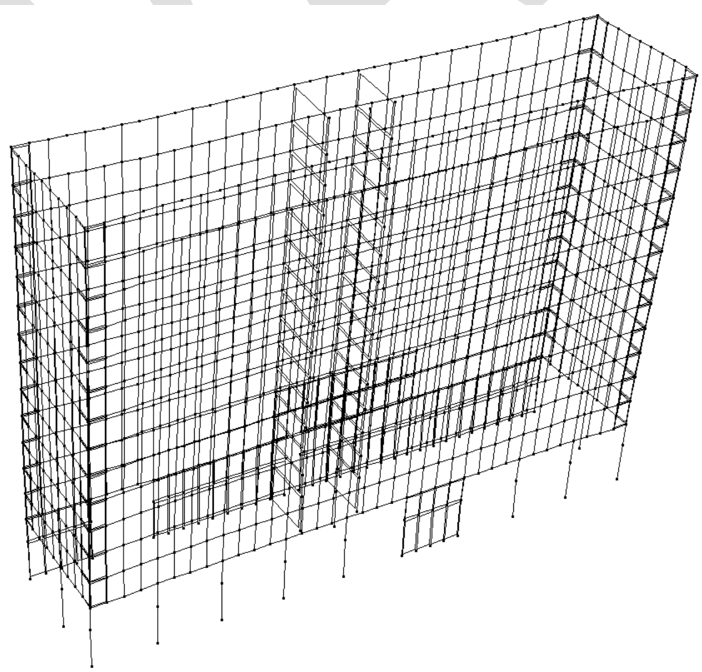

Figure 5.5 – Plot of north-south translational mode

Table 5.2 Vibration Period for Occidental Tower

Period of Vibration (sec.)		
Translation (N-S)	Torsion	Translation (E-W)
1.07	0.97	0.63

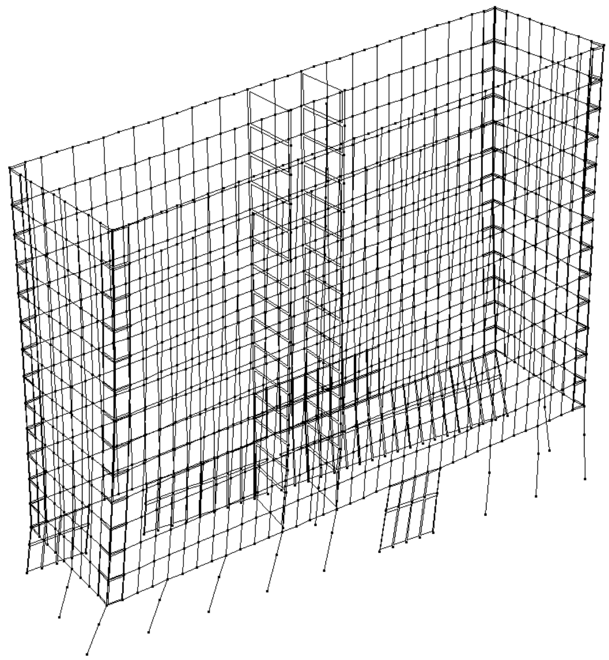

Figure 5.6 – Plot of torsional mode

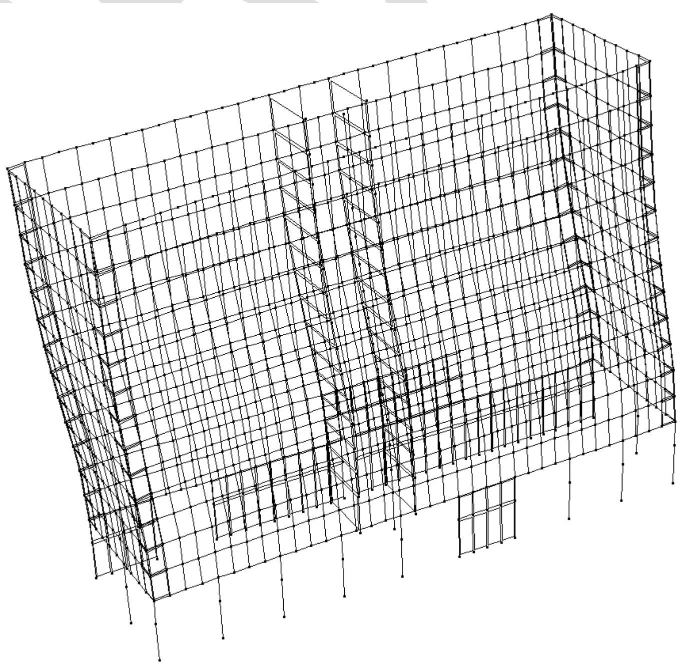

Figure 5.7 – Plot of east-west translational mode

Hammer Museum:

The linear dynamic procedure of ASCE 41 was used to analyze the seismic performance of the building. A three-dimensional finite element computer model of the building was developed using the computer program ETABS. The model was developed to study the overall distribution of the lateral force and the drift response. The structural model included concrete shear walls and steel moment frames. The metal deck with concrete fill floors and roof were modeled as semi-rigid diaphragms.

Modal analyses were performed to determine the dynamic response characteristics of the building. The fundamental periods of vibration for the building are summarized in Table 5.3.

Table 5.3 Vibration Period for Hammer Museum

Period of Vibration (sec.)		
Translation (N-S)	Torsion	Translation (E-W)
0.254	0.141	0.134

Figures 5.8 to 5.11 show plots of the three-dimensional computer model and the first three mode shapes of the Hammer Museum.

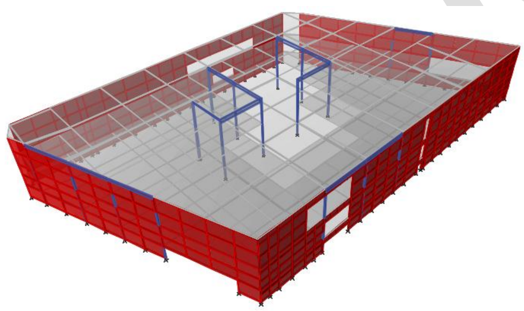

Figure 5.8 – Plot of Hammer Museum model

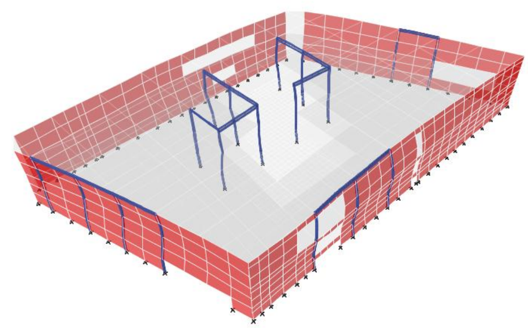

Figure 5.9 – Translation (N-S) mode response

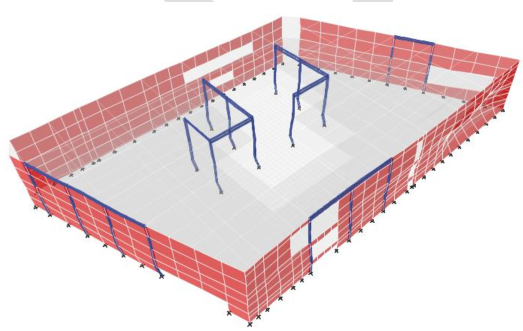

Figure 5.10 – Torsional mode response

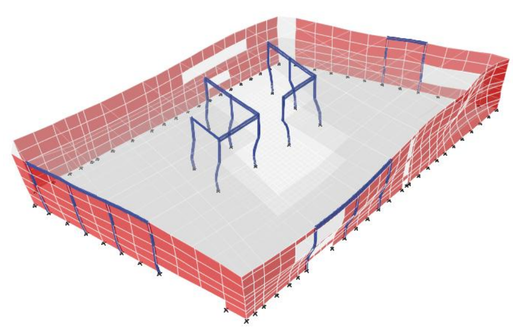

Figure 5.11 – Translation (E-W) mode response

5.4 Evaluation of Building Performance

Occidental Tower:

Nonlinear history response analyses were performed to capture higher mode and degradation effects. Seven pairs of spectrally matched ground motion time history records for the BSE-2 earthquake hazard level were used.

The results of the nonlinear history response analyses indicate formation of story mechanisms at the 3rd through 7th floors due to excessive overstress of the concrete shear walls resulting in excessive inter-story drifts. The average maximum inter-story drift in the north-south direction at the 4th floor is approximately 6% with maximum inter-story drift of approximately 10%. Generally accepted drift limits for collapse prevention performance at the 2,475-year seismic hazard is average maximum inter-story drift less than 3%, maximum inter-story drift less than 4.5%.

Figure 5.12 shows a plot of the deformed shape for the Sylmar record. Figure 5.13 shows a plot of the maximum inter-story drifts for all BSE-2 records.

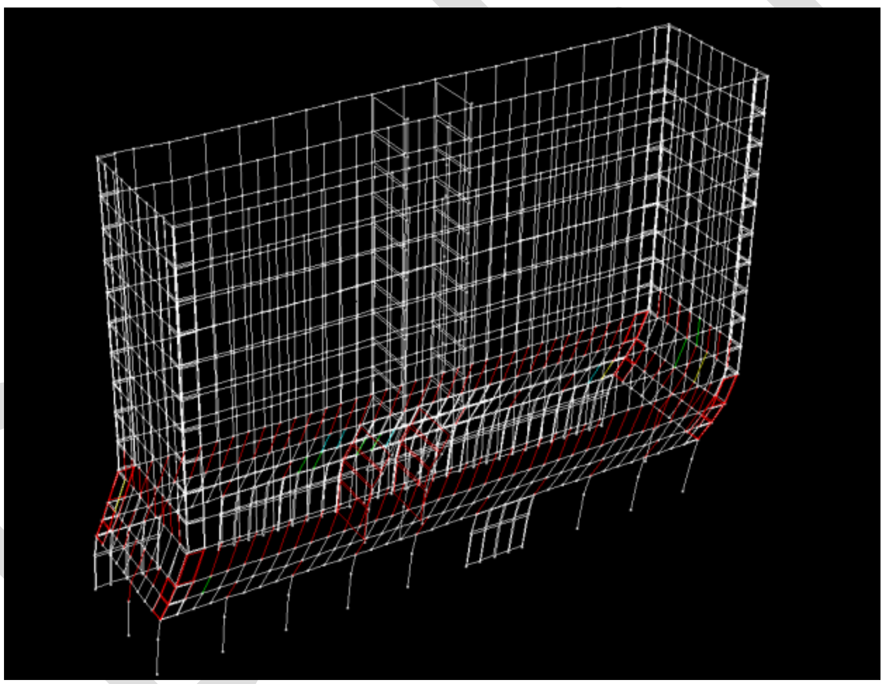

Figure 5.12 – Plot of deformed shape for Sylmar record (BSE-2)

The building was also evaluated for UC seismic rating level IV, by applying a scaling factor of 0.727 (provided by Geocon West) to the matched ground motions to approximate the 975-year seismic hazard (BSE-C) for the site.

The results of the 975-year seismic hazard analyses indicate slight overstress of the interior shear walls in the north-south direction, average maximum demand-to-capacity ratio of 1.1; and average maximum inter-story drift less than 2% with a maximum inter-story drift of 5.4% for the Sylmar Earthquake record and maximum residual drift of 1.2%. Figure 5.14 shows a plot of the deformed shape for the Sylmar record. Figure 5.15 shows a plot of the maximum inter-story drifts for all BSE-C records.

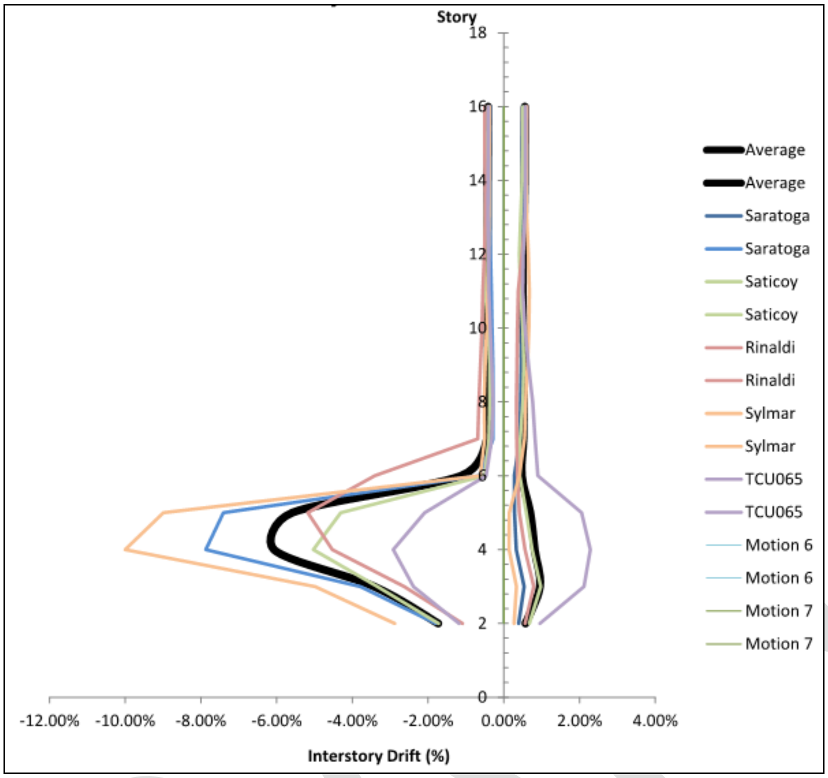

Figure 5.13 – Plot of average and maximum inter-story drifts (BSE-2)

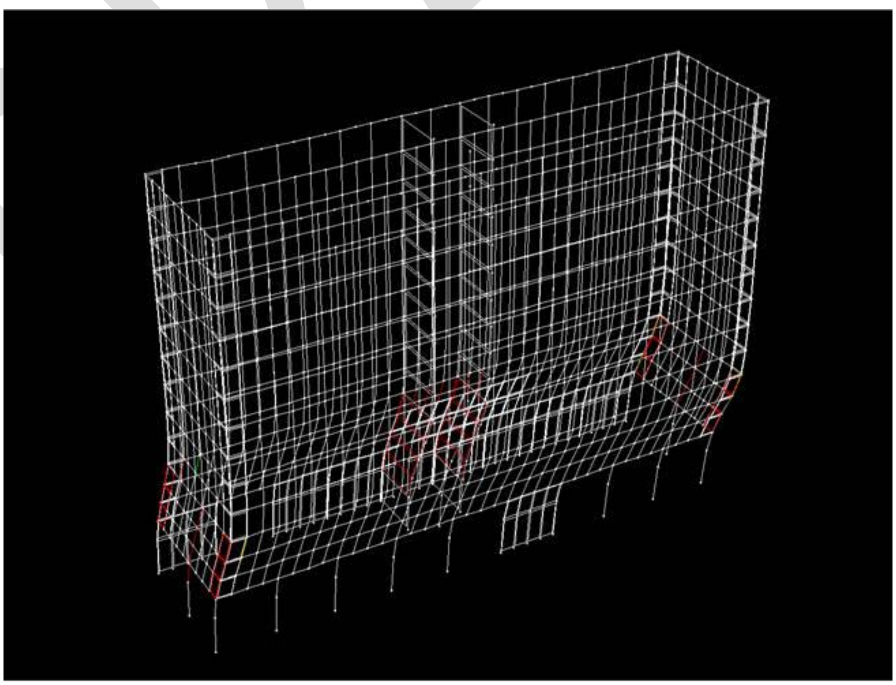

Figure 5.14 – Plot of deformed shape for Sylmar record (BSE-C)

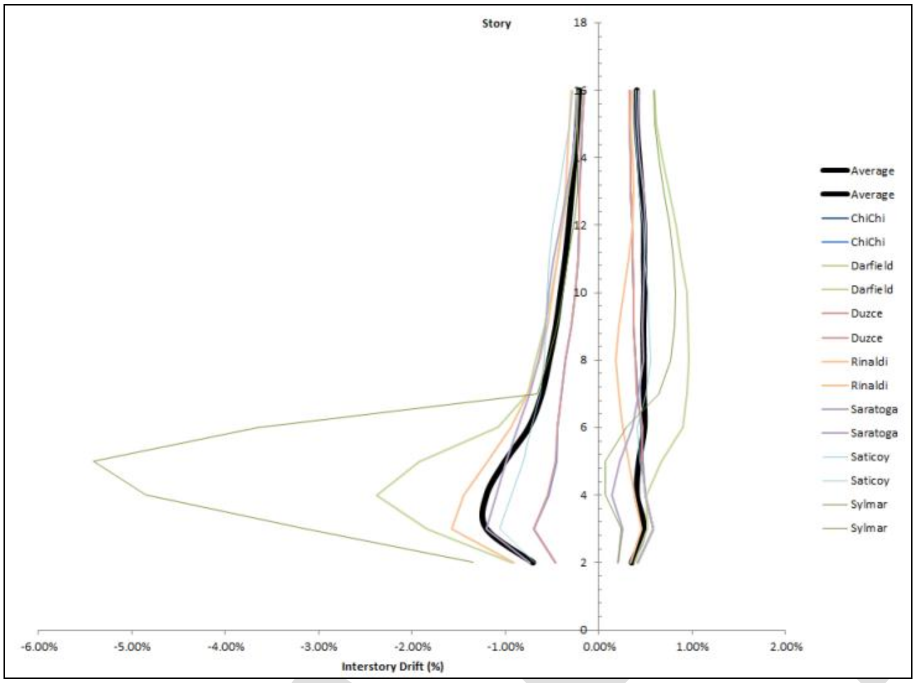

Figure 5.15 – Plot of average and maximum inter-story drifts (BSE-C)

Hammer Museum:

The building has long reinforced concrete shear walls along the perimeter. A few bays of steel moment frames are provided at the roof level along the perimeter of the courtyard opening and adjacent to the perimeter concrete shear walls.

The steel moment frames have welded Pre-Northridge moment connections that consist of field-welded full-penetration joints of the frame beam flanges to the column flanges and bolted web. Many steel moment frame buildings with this type of connection were damaged during the 1994 Northridge Earthquake – most of the observed damage occurred at or near the welded moment connection. Subsequent research indicate that the these connections have inherent flaws due to their configuration that concentrates stress and strain demands, construction practice, and welding material and processes (flux-cored arc welding process).

Prior to the 1994 Northridge earthquake, this type of moment connection was standard practice and most steel moment frame buildings constructed between 1970 and 1994 utilized this connection. The use of this type of connection is prohibited by current building codes for new construction.

The total calculated seismic weight of the building was approximately 20,600 kips. The seismic base shears for the BSE-1 and BSE-2 earthquake hazard levels are:

Earthquake Hazard Level	North-South	East-West
BSE-1 (10/50 – 475 yr)	18,000 kips (0.87W)	16,400 kips (0.80W)
BSE-2 (2/50 – 2475 yr)	27,000 kips (1.31W)	24,600 kips (1.19W)

A summary of the maximum member demand-to-capacity ratio (DCR) for the Collapse Prevention performance at BSE-2 earthquake hazard level is provided in Table 5.4.

Table 5.4 Summary of Maximum DCR for Collapse Prevention Performance Level

Component	Max.	Avg.
Concrete Shear Walls	0.61	-
Moment Frame Columns	0.51	-
Moment Frame Beams	0.87	-

Figure 5.16 shows a plot of the maximum inter-story drift ratios from the dynamic analysis at each story for the BSE-2 earthquake hazard level. As can be seen, the maximum inter-story drifts are well within acceptable levels at all floors.

The following deficiencies were identified based on the results of the analysis:

- Pre-Northridge welded moment connections with limited ductility.
- Beams and columns of moment frame do not satisfy strong column/weak beam configuration. Strong column/weak beam configuration promotes yielding of the beam – a preferred behavior.
- The moment frame beams are connected to the weak axis of the column at many locations.

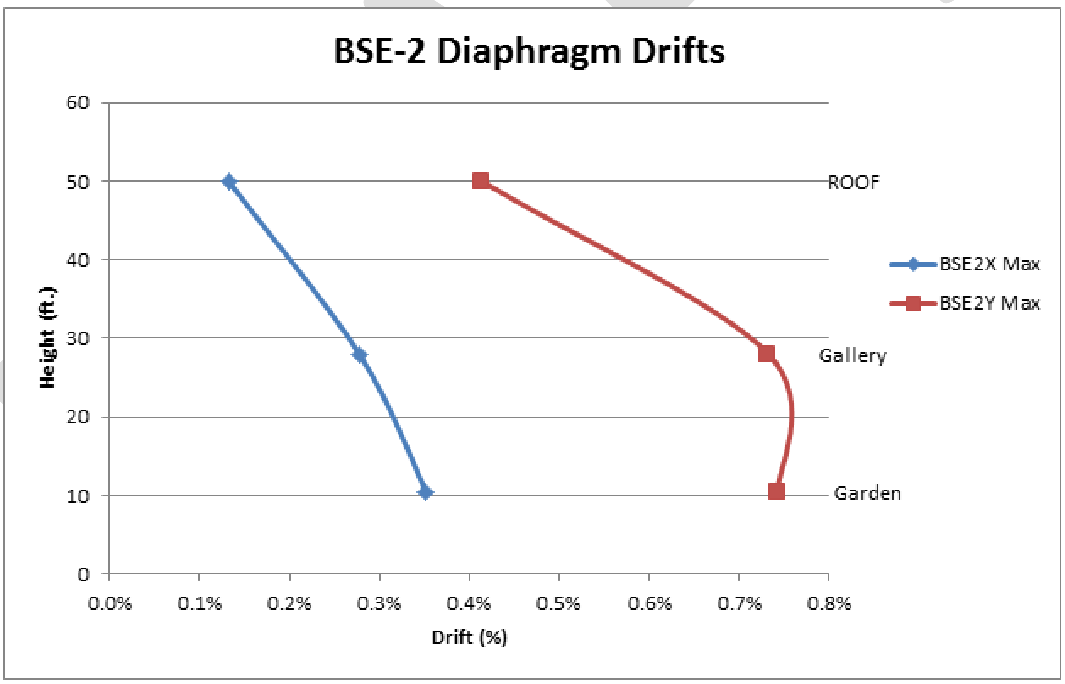

Figure 5.16 – Plot of Maximum BSE-2 Inter-story Drifts for Hammer Museum

Despite these deficiencies, the results of the analysis indicate that the moment frames satisfy the Life Safety and Collapse Prevention performance criteria at BSE-1 and BSE-2 earthquake hazard levels, respectively. This is due to the stiffness of the perimeter concrete walls that attract the majority of the seismic forces. The steel moment frames ultimately function more as chord elements at the diaphragm discontinuity.

5.5 Conclusion

The results of the analysis indicate that the Occidental Tower does not satisfy the requirements for UC seismic rating level III. In addition, based on the results and the fact that the ground motions used to evaluate the existing building for the UC seismic rating level IV performance was amplitude scaled and not spectrum matched, it is our professional opinion that the building satisfies the UC seismic rating level IV requirements. Further development of ground motions for the 225-year and 975-year seismic hazards is recommended and the associated analysis peer reviewed to confirm compliance with seismic rating level IV.

The results of the analysis indicate that the Hammer Museum satisfies the requirements for UC seismic rating level III.

6.0 RECOMMENDATIONS

The evaluation of the Occidental Tower identified inadequate shear strength of concrete walls in north-south direction as the principal structural deficiency.

A conceptual seismic strengthening scheme was studied and evaluated to mitigate the identified deficiencies and improve the building performance to UC seismic rating level III. A description of the proposed strengthening scheme and analyses is provided in the following section.

6.1 Recommended Strengthening Scheme

The recommended seismic strengthening scheme to achieve a UC seismic rating Level III consists of:

- Strengthening existing interior concrete walls, located adjacent to the elevator banks, with fiber reinforced mesh from the foundation to the underside of the 12th floor; and
- Add new single-bay interior buckling restrained brace (BRB) frames at the east and west perimeter of the building from the foundation to the roof.

Figure 6.1 shows the location of the proposed strengthening on a typical floor plan. Figure 6.2 shows an isometric view of the computer model with the proposed strengthening indicated.

The results of the analyses indicate no overstress and average maximum inter-story drift less than 2% with a maximum inter-story drift less than 3%. Figure 6.3 shows a plot of the deformed shape. Figure 6.4 shows a plot of the maximum inter-story drifts for all BSE-2 records.

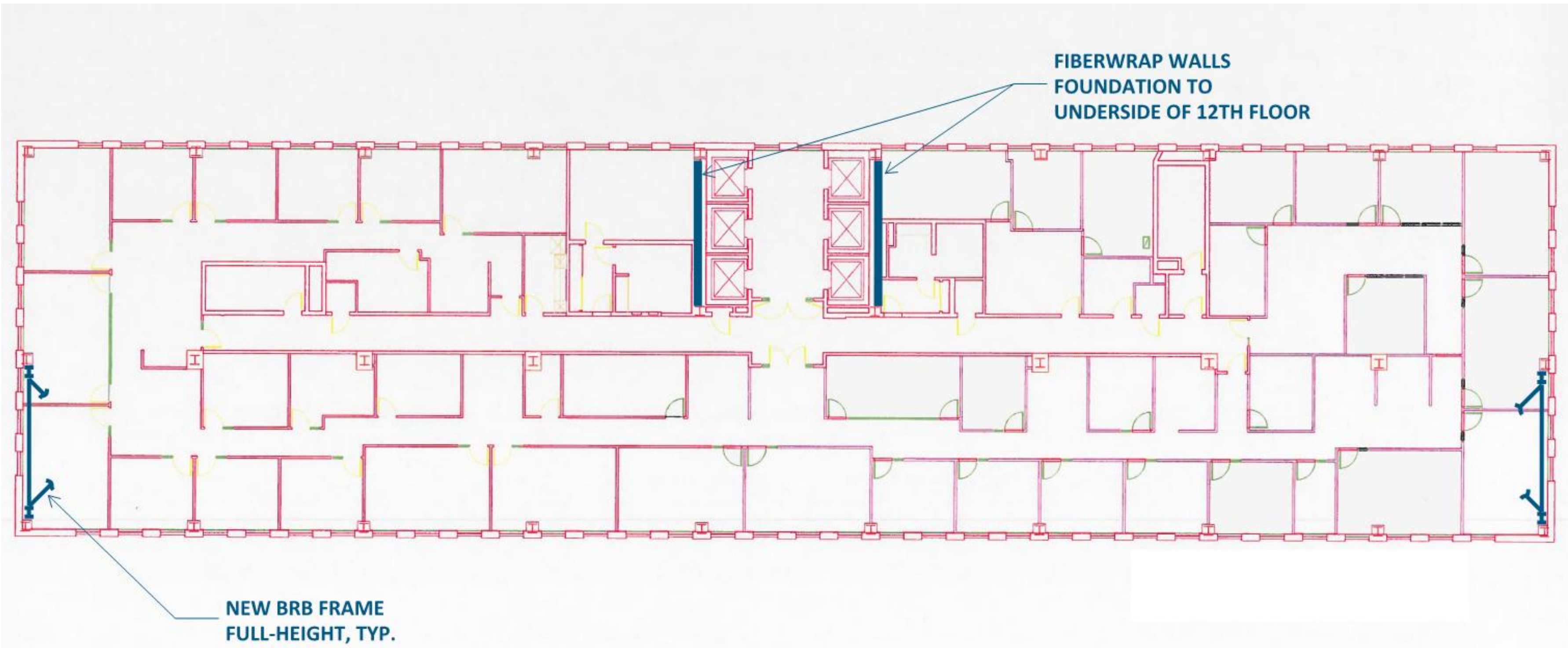

Figure 6.1 – Typical floor plan of proposed seismic strengthening

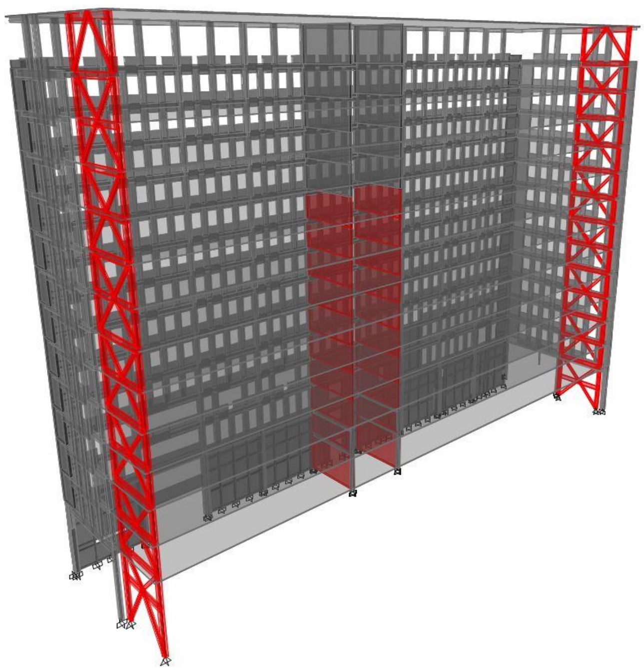

Figure 6.2 – Isometric view of proposed seismic strengthening

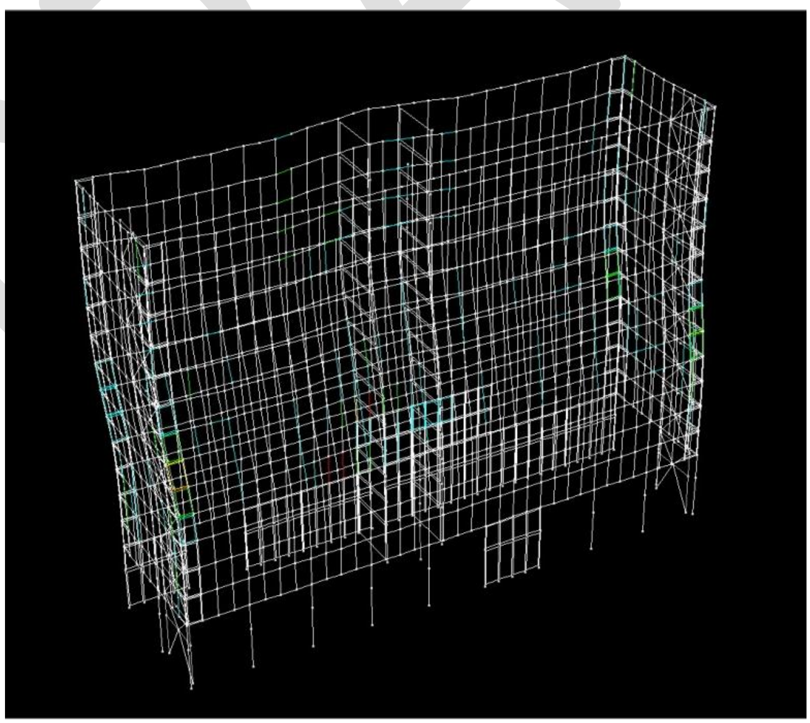

Figure 6.3 – Plot of deformed shape for Sylmar record (BSE-2)

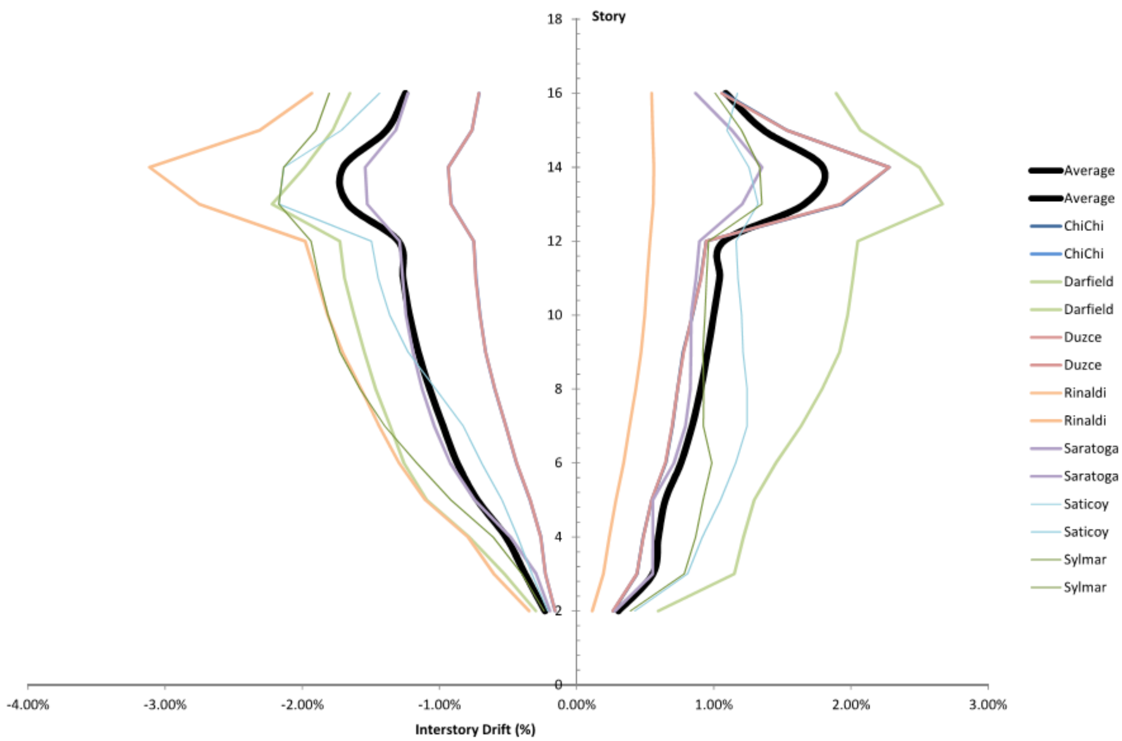

Figure 6.4 – Plot of average and maximum inter-story drifts (BSE-2)