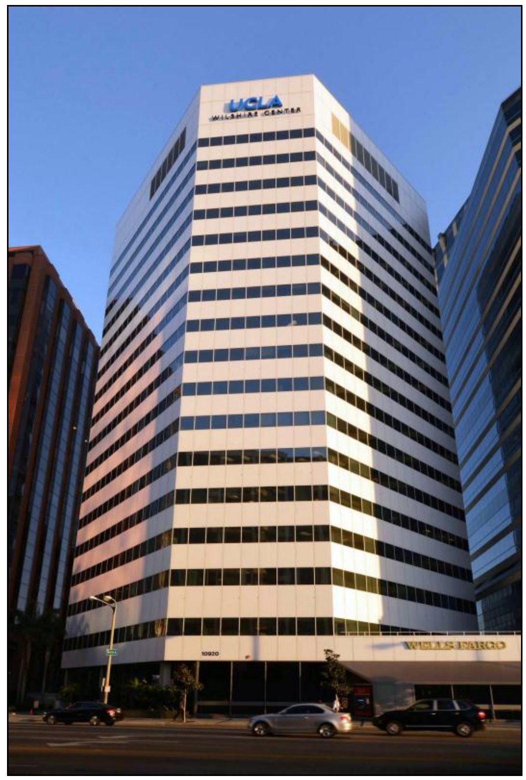
SEISMIC EVALUATION

of

Wilshire Center
10920 Wilshire
Los Angeles, CA

Prepared for:

UCLA Capital Programs
1060 Veteran Avenue
Los Angeles, CA



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0.0 EXECUTIVE SUMMARY

This report presents the results of the seismic evaluation of the 17-story office building with 4-levels of subterranean parking located at 10920 Wilshire Boulevard in Los Angeles. The office floors are typically octagon-shaped in-plan. The building was constructed in 1981 and designed to the 1976 edition of the Uniform Building Code (UBC).

The building is of steel frame construction with metal deck and concrete fill roof and floors. The foundation system consists of shallow spread footings supporting columns and continuous footing supporting retaining walls. The lateral-force-resisting system consists of perimeter welded steel moment frames with "pre-Northridge" moment connections.

A linear dynamic analysis of the building was performed to evaluate performance in accordance with University of California Seismic Safety Policy requirements. The welded beam-to-column moment connections and column splice connections were found to have inadequate strength. The results of the analysis indicate that the building does not satisfy the requirements for SPL rating IV and is assigned an SPL V rating.

Conceptual strengthening to mitigate the identified deficiencies and improve building performance to SPL IV includes the following:

- Retrofit all beam-to-column welded moment connections using proprietary slotted beam web connection by Seismic Structural Design Associates, Inc. (SSDA) or welded haunch at bottom flange. There are 24 beam-to-column moment connections per floor, and all connections from 2nd floor through the roof level (17 floors) require retrofit for a total of 408 connections.
- Enhance all column splice connections for shear by fillet welding around existing ¾" splice plate. There are 12 moment frame columns per floor. Columns are spliced at every other floor, thus there are 9 floors at which splices occur for a total of 108 splice connections.
- Enhance approximately 15% of column splice connections for flexure by welding steel plates to existing column flanges.

Cost range to retrofit: **LOW***

It is recommended that testing of the moment connection and column splice connection weld material be performed to determine material toughness and refine assessment of connection performance.

It is also recommended that nonlinear response history analysis (NLRHA) be performed to verify adequacy of proposed mitigation measures. Linear dynamic analyses used for this evaluation have limitations in capturing the realistic behavior of tall steel moment frame buildings – yield sequence, and strength and stiffness cyclic degradation.

* See U.C. Seismic Program Guidebook Version 1.3 or Section 5 for definition of cost range.

1.0 INTRODUCTION

1.1 General

This report presents the results of the seismic evaluation of the 17-story office building with 4-levels of subterranean parking located at 10920 Wilshire Boulevard in Los Angeles. 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 building was determined by a review of structural drawings, structural analysis and a general seismic hazard analysis for the region.

A description of the construction of the building is provided in Section 2. The likelihood of earthquake-induced site failure is discussed in Section 3. The criteria used in the evaluation of the building, analysis assumptions, and a summary of the results are discussed in Section 4. Conceptual strengthening approaches to mitigate the identified deficiencies and improve building performance are provided 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 Tishman Midvale Building, Erkel/Greenfield & Associates (79-11), September 12, 1979.

- *Report of Geotechnical Investigation Proposed Armand Hammer Museum,* LeRoy Crandall & Associates (AE-88055), April 6, 1988.
- *Seismic Evaluation and Retrofit of Existing Buildings*, American Society of Civil Engineers, 41-17, 2017.
- *California Code of Regulations, Title 24, Part 2, Volume 2*, California Building Standards Commission, 2010.
- *University of California, Seismic Safety Policy*, May 19, 2017.
- State of California Earthquake Zone of Required Investigation, Beverly Hills Quadrangle, California Geological Survey, January 11, 2018.

2.0 BUILDING DESCRIPTION

2.1 General

The Wilshire Center is located at 10920 Wilshire Boulevard between Wilshire Boulevard, Ashton Avenue, Westwood Boulevard and Midvale Avenue. The building was constructed in 1981 and likely designed to the 1976 edition of the Uniform Building Code (UBC).

The office floors are typically octagon-shaped in-plan with overall dimensions of approximately 151′ by 162′. Figure 2.1 shows the framing plan of the typical office floor. The floor-to-floor height of the typical office floor, 1st floor, 2nd floor, and 17th floor is 12′-6″, 18′-0″, 14′-0″ and 14′-6″, respectively.

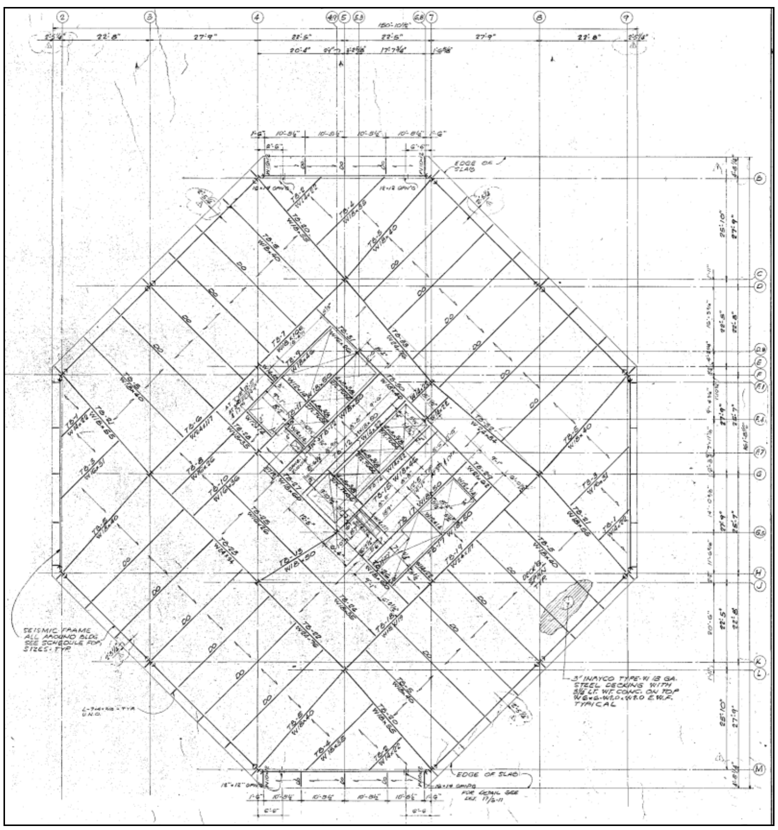

Figure 2.1 – Framing Plan for Typical Office Floor

2.2 Gravity System

The roof and typical floors are constructed of 3″ deep 18 GA metal deck with 3¼″ light weight concrete fill spanning to steel wide flange beams and girders supported by steel wide flange columns that are continuous to the foundation. The columns are spliced at every other floor. The foundation system consists of shallow concrete spread footings supporting columns and continuous strip footings supporting concrete retaining walls. A 4″ thick reinforced concrete slab-on-grade forms the basement floor.

2.3 Lateral System

The lateral-force-resisting system consists of the metal deck and concrete fill roof and floors acting as structural diaphragms to transfer seismic inertial forces to perimeter welded steel moment frames that are continuous to the foundation. At street level and below perimeter reinforced concrete retaining walls also resist seismic loads.

The moment frame beams are W36 sections and the columns are W14 sections and built-up I-shapes spliced with partial penetration welds at the flanges and bolted web. Figure 2.2 shows a detail of the column splice connection. The beams and columns consist of ASTM A36 steel. The proportioning of the frame members typically do not satisfy strong column-weak beam joint checks.

The moment frames have beam-to-column-flange connections that are typical "pre-Northridge" welded moment connection which consists of field-welded full-penetration joints of beam flange to column flange with bolted shear tab. Figure 2.3 shows a detail of the beam-to-column-flange connection.

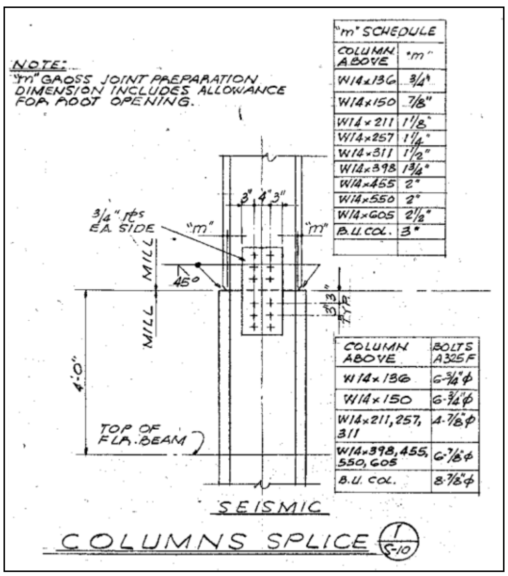

Figure 2.2 – Column Splice Detail

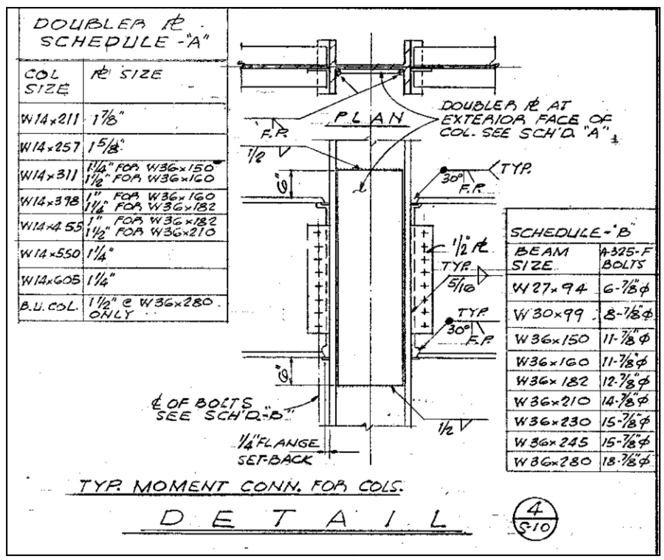

Figure 2.3 – Moment Connection Detail

3.0 EARTHQUAKE INDUCED SITE FAILURE

3.1 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*.

Site-specific information on subsurface soil conditions was not available for this review. Notes on the structural drawing indicate that the soil at the site consists of shale.

3.1.1 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). The closest identified active fault to the site is the Santa Monica fault, which is approximately 0.5 mile away. The potential for ground surface rupture is low.

3.1.2 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 relatively flat site and is not adjacent to steep slopes. Therefore, the potential for landsliding is very low.

3.1.3 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.

Geotechnical report for a nearby site (10889 Wilshire Blvd) indicates that the underlying soils in the area are dense and stiff and are not subject to liquefaction.

4.0 BUILDING PERFORMANCE IN EARTHQUAKES

4.1 Evaluation Criteria

The building was evaluated in accordance with the University of California (UC) seismic safety policy for Seismic Performance Level (SPL) Rating IV. This level of seismic performance is equivalent to the performance of Risk Category I-III for existing buildings as established in Chapter 3 of the 2016 California Existing Building Code (CEBC). The CEBC uses, by reference, the methodology and procedures of ASCE 41-17, *Seismic Evaluation and Retrofit of Existing Buildings*. ASCE 41-17 is national standard for the seismic rehabilitation of buildings.

The building was evaluated per Section 317 of the 2016 CEBC with modified earthquake hazard levels per the UC Seismic Safety Policy. A two tier evaluation was performed using the performance criteria specified in Table 317.5 for Occupancy Categories I-III. The criteria used are presented in Table 4.1.

Table 4.1 Seismic Performance Criteria

Evaluation Tier	Earthquake Hazard Level	Structural Performance Level	Nonstructural Performance Level
1	BSE-1E (20/50 – 225 yr)	Life Safety	Hazard Reduced
2	BSE-2E (5/50 – 975 yr)	Collapse Prevention	Not Considered

4.2 Analysis Assumptions

A three-dimensional computer model of the building was developed using ETABS 2017, developed by Computers & Structures, Inc. The model included all elements that significantly contribute to the lateral force resistance of the building: these include the metal deck with concrete fill roof and floors, and perimeter steel moment frames. The roof and floors were modeled as semi-rigid diaphragms. The seismic base of the building was assumed to be at street level. The columns and walls typically have pinned supports at the basement level. Figure 4.1 shows a plot of the ETABS model.

The building was analyzed using the linear dynamic procedure of ASCE 41-17, where modal spectral analysis is performed using linearly elastic response spectra that are not modified to account for anticipated nonlinear response. The procedure produces displacements that approximate maximum displacements expected during the design earthquake, but internal forces exceed those that the building can sustain because of anticipated inelastic response of components and elements. These forces are evaluated using acceptance criteria that include modification factors.

The default soil profile, Class D, was used since site specific soil data was not available. The response acceleration parameters for the BSE-1E and BSE-2E earthquake hazard level, adjusted for the site soil conditions, are:

Earthquake Hazard Level	S_{XS}	S_{X1}
BSE-1E (20/50 – 225 yr)	0.895g	0.515g
BSE-2E (5/50 – 975 yr)	1.547g	0.946g

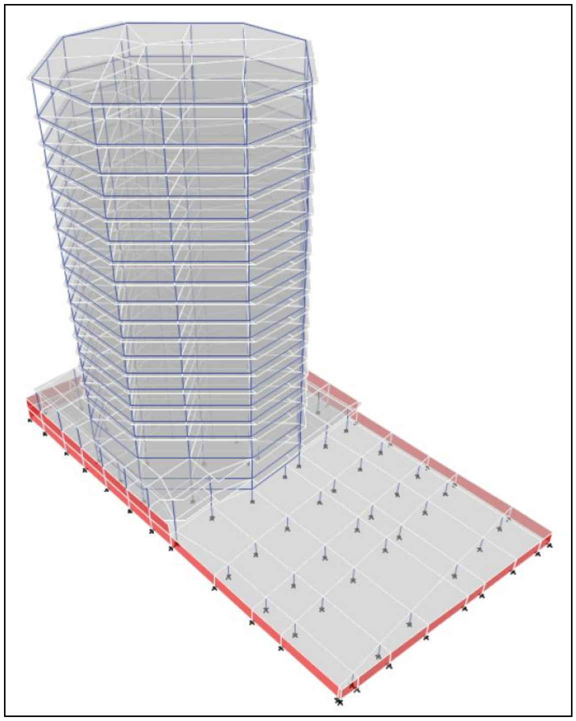

Figure 4.1 – Plot of ETABS Model

4.3 Evaluation of Building Performance

Modal analyses were performed to determine the dynamic characteristics of the building. The results indicate that the lateral force resisting elements generally provide a regular response. Table 4.2 summarizes the fundamental periods. Figure 4.2 through 4.4 shows plots of the fundamental mode shapes.

Dynamic analyses were performed to establish likely earthquake demand on individual structural components and global response. The demands on individual components were evaluated using ASCE 41-17 acceptance criteria for Life Safety and Collapse Prevention performance. Table 4.3 presents a summary of the seismic base shear in north-south and east-west direction for the different hazard levels considered.

Global inter-story drift response results indicate potential soft story condition at the base of the tower. Figure 4.5 shows a plot of the maximum inter-story drift for the BSE-2E hazard. The maximum drifts are between 2.5%-2.8%.

The moment frame beams, columns, welded moment connections and column splice connections were evaluated based on internal forces from the analyses, expected strength of the element and appropriate element modification factor. Member demand-to-capacity ratio (DCR) including element modification factor is a commonly used metric to assess the adequacy of the member; members with DCR less than 1.0 are considered adequate.

The results indicate that the moment frame beams and columns provide adequate performance at BSE-1E and BSE-2E hazards. The results also indicate that the welded beam-to-column moment connections and column splice connections have DCRs > 1.0. Table 4.4 provides a summary of member demand-to-capacity ratios.

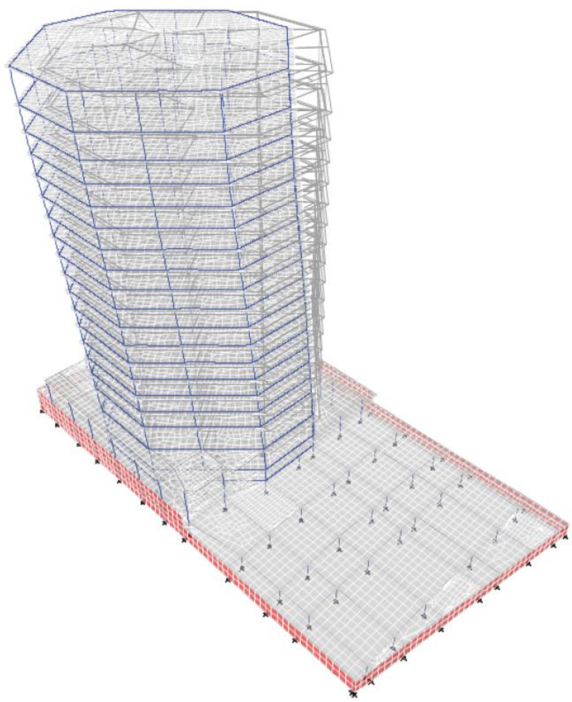

Figure 4.2 – NW-SE Translational Mode

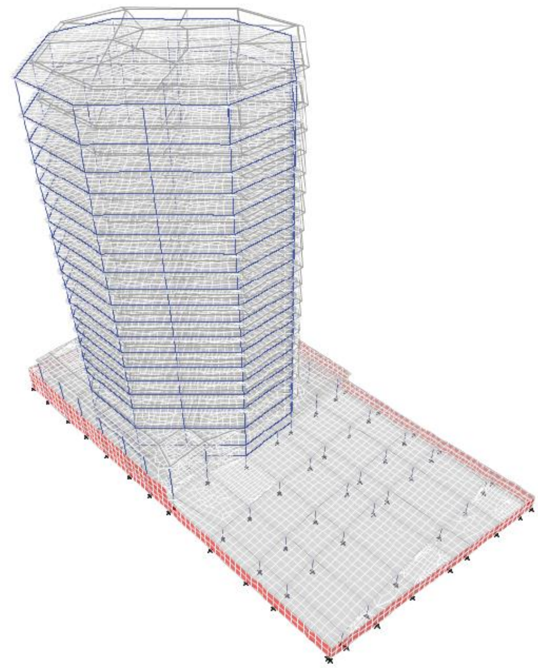

Figure 4.3 – NE-SW Translational Mode

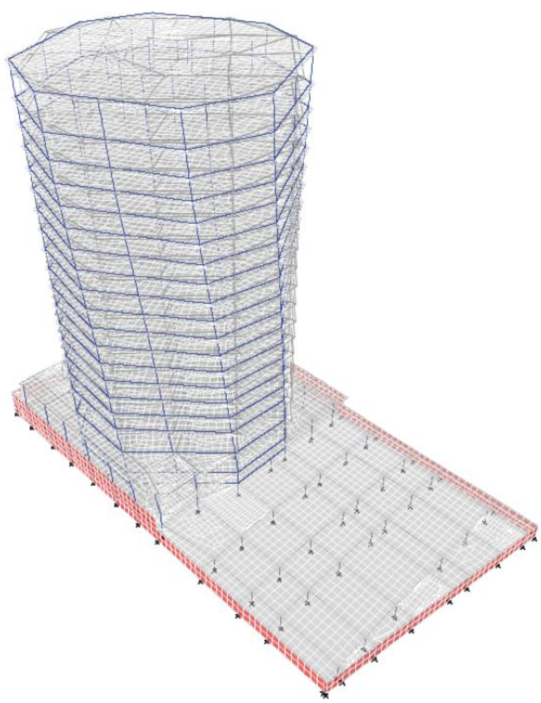

Figure 4.4 – Torsional Mode

Table 4.2 Fundamental Periods

Fundamental Mode	Period (sec.)
NW-SE	4.6
NE-SW	4.6
Torsion	2.5

Table 4.3 Seismic Base Shear

Seismic Hazard	E-W	N-S
BSE-1E	0.102W	0.098W
BSE-2E	0.183W	0.178W

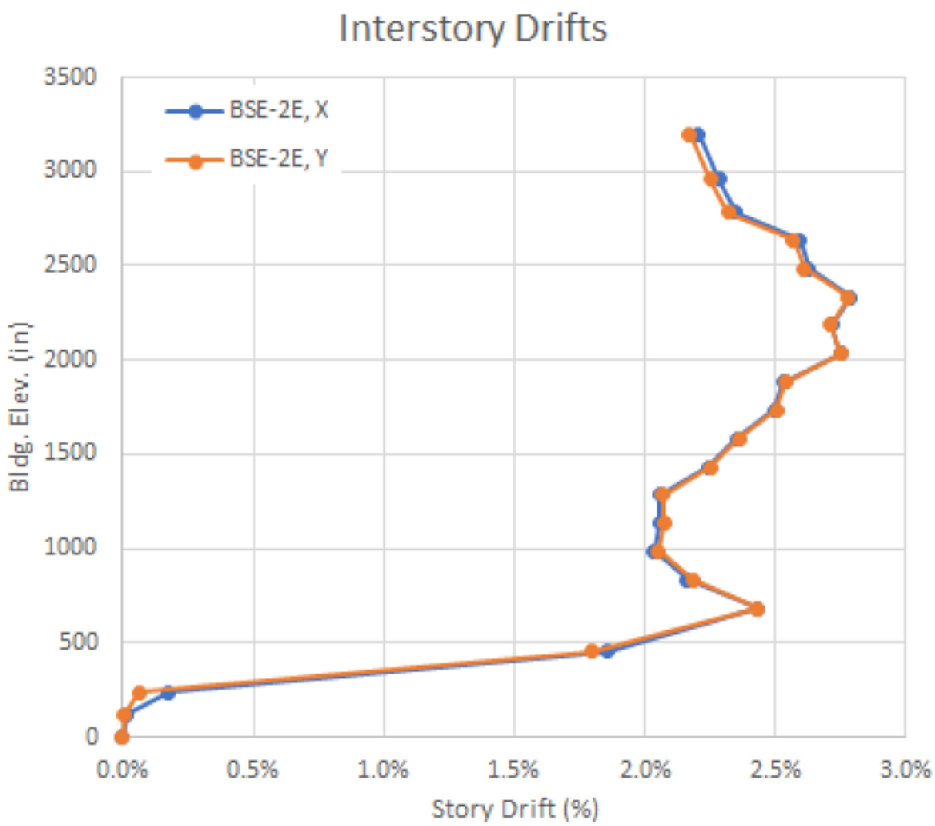

Figure 4.5 – Maximum Inter-Story Drift for BSE-2E

Table 4.4 Summary of Member Demand-to-Capacity Ratio

Element/Member	BSE-1E		BSE-2E	
	Max. DCR	% w/DCR>1.0	Max. DCR	% w/DCR>1.0
Beam	0.64	NA	0.91	NA
WUF Connection - V	1.26	4	1.61	91
WUF Connection - M	1.66	19	1.62	91
Column – V	0.50	NA	0.70	NA
Column - M	0.56	NA	0.81	NA
Column Splice – V	-	-	2.36	39
Column Splice - M	-	-	1.94	15

The column splice connections have inadequate shear strength to develop the full flexural capacity of the columns.

The building was also evaluated to SPL V requirements: life safety performance at ⅔BSE-1E and collapse prevention performance at ⅔BSE-2E. The results show that a limited number (~10%) of the beam-to-column welded connections and column splice connections are slightly overstressed for collapse prevention performance.

4.4 Conclusion

A linear dynamic analysis of the building was performed to evaluate performance in accordance with U.C. Seismic Safety Policy. The results of the analysis indicate that the building does not satisfy the requirements for SPL rating IV and is assigned an SPL V rating.

It is noted that linear dynamic analyses have limitations in assessing seismic performance of tall steel moment frame buildings. It is recommended that nonlinear response history analysis be performed to verify rating.

5.0 RECOMMENDATIONS

Conceptual strengthening to mitigate the identified deficiencies and improve building performance to SPL IV includes the following:

- Retrofit all beam-to-column welded moment connections using proprietary slotted beam web connection by Seismic Structural Design Associates, Inc. (SSDA) or welded haunch at bottom flange. There are 24 beam-to-column moment connections per floor, and all connections from 2nd floor through the roof level (17 floors) require retrofit for a total of 408 connections. Figure 5.1 shows the location of the beam-to-column moment connections on the framing plan for the typical floor. Figure 5.2 shows a conceptual detail for the retrofit of the beam-to-column moment connection using SSDA slotted beam web. Figure 5.3 shows a conceptual detail for the retrofit of the beam-to-column moment connection using welded haunch at bottom flange.
- Enhance all column splice connections for shear by fillet welding around existing ¾" splice plate. There are 12 moment frame columns per floor. Columns are spliced at every other floor, thus there are 9 floors at which splices occur for a total of 108 splice connections. Figure 5.4 shows a conceptual detail for the column splice connection retrofit.
- Enhance approximately 15% of column splice connections for flexure by welding steel plates to existing column flanges.

Cost range to retrofit: **LOW***

It is recommended that testing of the moment connection and column splice connection weld material be performed to determine material toughness and refine assessment of connection performance.

It is also recommended that nonlinear response history analysis (NLRHA) be performed to verify adequacy of proposed mitigation measures. Linear dynamic analyses used for this evaluation have limitations in capturing the realistic behavior of tall steel moment frame buildings – yield sequence, and strength and stiffness cyclic degradation.

Conceptual strengthening to improve building performance to SPL III requires retrofit of all beam-to-column welded moment connections, enhancement of all moment frame column splice connections, and supplementing the existing steel moment frames (will require foundation work) with steel braced frames or viscous dampers. Cost range to retrofit: **MEDIUM/HIGH***

* Per U.C. Seismic Program Guidebook Version 1.3:

- Low cost-range corresponds to a complete retrofit cost less than $50 per square foot (sf).
- Medium cost-range corresponds to a complete retrofit cost greater than $50/sf and less than $200/sf.
- High cost-range corresponds to a complete retrofit cost greater than $200/sf and less than $400/sf.
- Very High cost-range corresponds to a complete retrofit cost greater than $400/sf.

Note this range includes all construction costs, including code upgrades (e.g., ADA, fire and life safety, mechanical, electrical, plumbing) triggered by the seismic retrofit. Cost range is based only on the engineer's rough estimate (no input from a professional cost estimator).

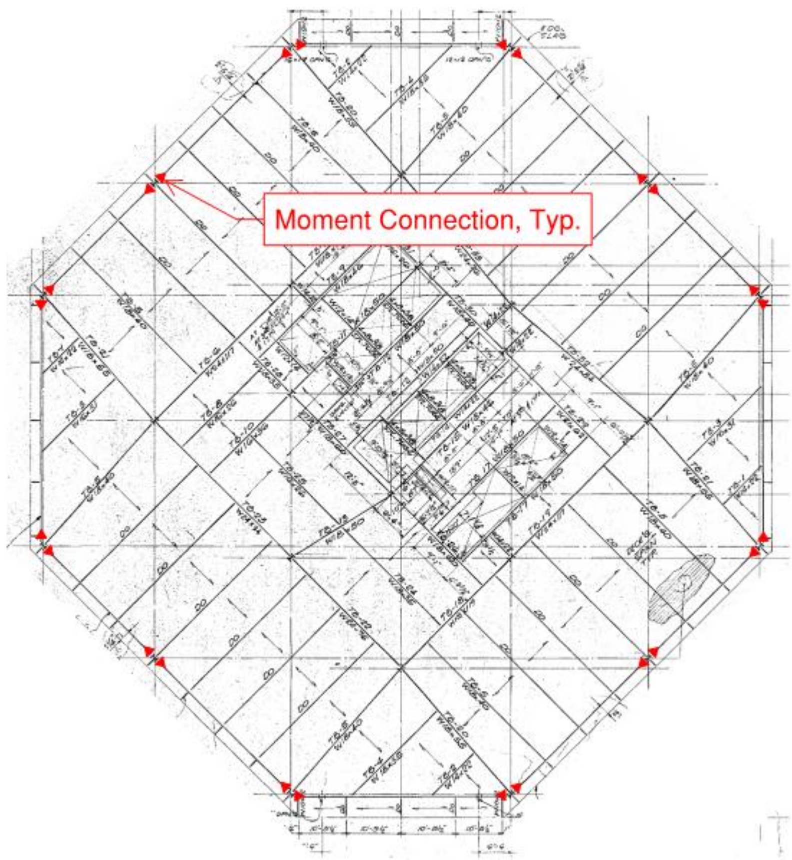

Figure 5.1 – Typical Floor Plan with Location of Moment Connection Indicated

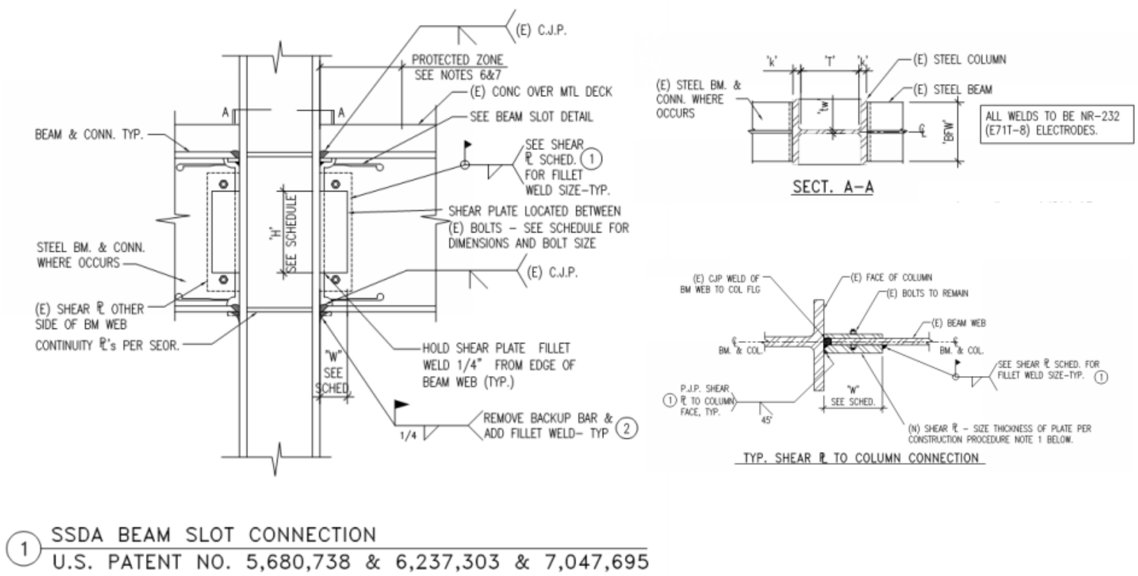

Figure 5.2 – Conceptual Detail of Moment Connection Retrofit using Slotted Beam Web

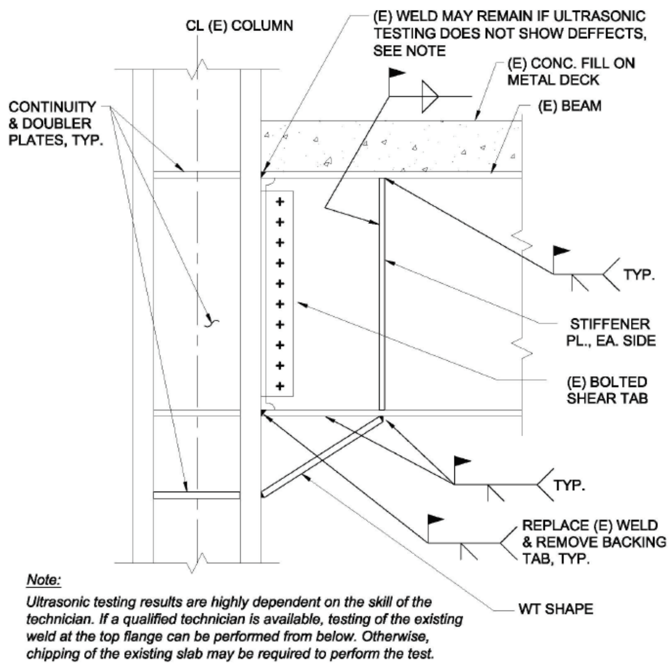

Figure 8.4.6-2: Welded Haunch at Bottom Flange of Existing Beam
(adapted from AISC Design Guide 12)

Figure 5.3 – Conceptual Detail of Moment Connection Retrofit using Welded Haunch at Bottom Flange

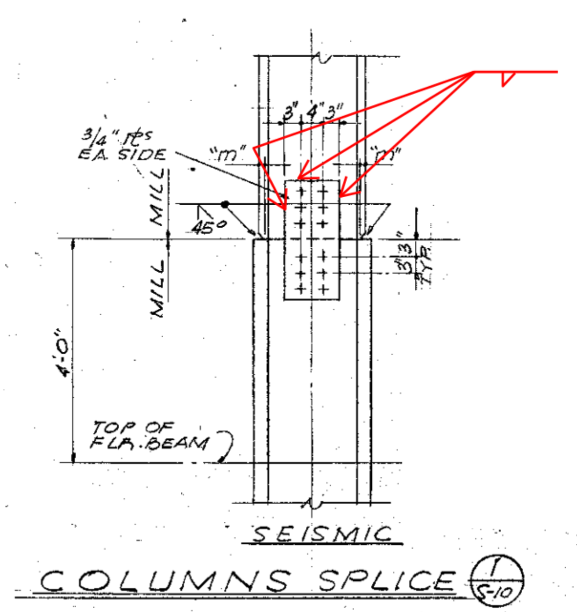

Figure 5.4 – Column Splice Enhancement for Shear

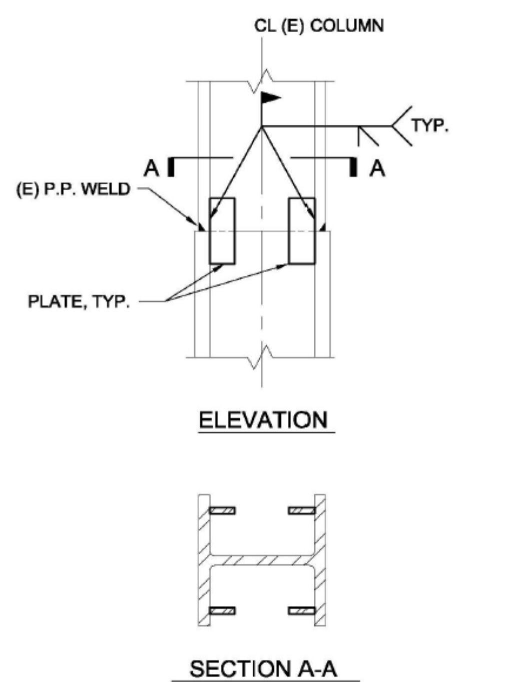

Figure 8.4.7-1: Welded Splice Upgrade at Existing Column

Figure 5.5 – Column Splice Enhancement for Flexure