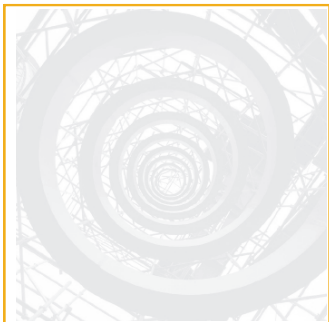

UCLA Seismic Rating Study

11075 Santa Monica Boulevard

Los Angeles, California

9/8/17

Englekirk
INSTITUTIONAL

September 8, 2017

Job No. 17-G132

UCLA Seismic Rating Study

11075 Santa Monica Boulevard

Los Angeles, California

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September 8, 2017

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1.0 INTRODUCTION

1.1 General

This report presents the findings of our seismic performance review of the 11075 Santa Monica Building, Los Angeles, California. The evaluation was based on a qualitative review of the structure using the American Society of Civil Engineer's *Seismic Rehabilitation of Existing Buildings,* ASCE 41-13 Tier 1 and Tier 2 Procedures, and the *2016 California Building Code (CBC*) Part 10, Chapter 3. An Earthquake Performance Level was assigned per the *UC Seismic Safety Policy*.

The location of the building is shown in Figure 1.1.

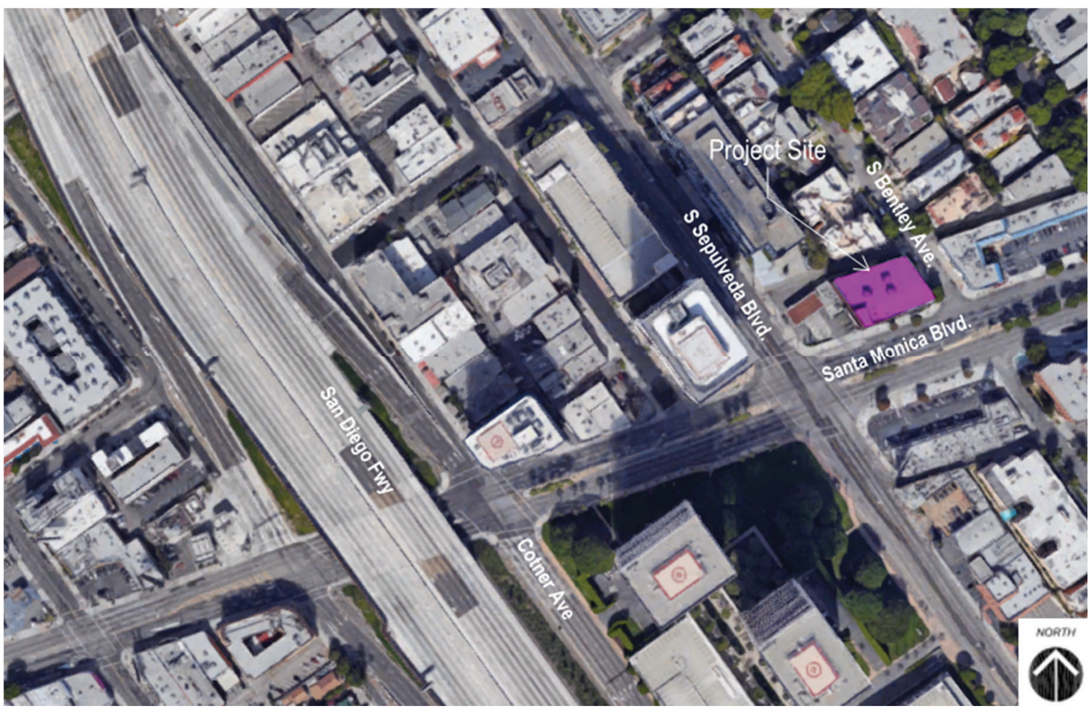


Figure 1.1 - Building Location - Source: Google Maps

1.2 Information Reviewed

The building owner undertook a search for record structural drawings, including at the City of Los Angeles Department of Building and Safety; no useful record drawings were located for the building. Thus, site visits were performed on July 24, 2017 and August 21, 2017 to review the structure and measure existing beam and column sizes that were exposed around the perimeter of the building at multiple floors.

Public records of building permits at this address indicate that it was constructed in approximately 1982. Based on this, it is assumed that the 1979 Uniform Building Code, or its equivalent City of Los Angeles Building Code, formed the basis of the building's structural design.

1.3 Tasks Performed

The following tasks were performed as a part of our review of the building:

- Visit the site to measure existing beam and column sizes at exposed locations.
- Develop a structural analysis model of the building to reflect the existing seismic system based on results from the site visit.
- Obtain response spectra parameters from the *United States Geological Survey*.
- Perform an ASCE 41 Tier 1 seismic evaluation to identify key potential deficiencies in the building.
- Perform an ASCE 41 Tier 2 analysis to study steel moment frame connections that predate the 1994 Northridge Earthquake and other potential deficiencies identified in the Tier 1 review.
- Based on the results of the analysis, prepare an evaluation of the anticipated seismic performance of the existing structure and provide a seismic rating based on the *UC Seismic Safety Policy*.

2.0 BUILDING DESCRIPTION

2.1 General Building Description

The building has a rhomboid shape with an approximate footprint of 13,100 ft^2. Although complete record drawings were not available for the building, a floor plan of the at-grade parking level is shown in Figure 2.1, based on information obtained from public records. A representative photograph of the building's exterior elevation is shown in Figure 2.2.

The existing building consists of one story of subterranean parking, one level of at-grade parking, and three floors of office space. Based on visual observations, it appears that the on-grade parking level and the first floor of office space consist of reinforced concrete beams and slabs supported by concrete columns along the interior of the building and concrete columns and concrete masonry unit (CMU) walls along the exterior. Subsequent levels consist of steel beams and columns supporting sawn lumber joists or engineered wood members.

2.2 Lateral System

The lateral framing system for the building, based upon observations made during our site visit, consists of reinforced concrete (RC) diaphragms that span to CMU walls at the first elevated level (first floor of office space). Subsequent levels consist of flexible wood plywood panel diaphragms that span to steel moment frames along the north, south and east perimeters and a CMU wall along the west perimeter. The CMU wall is continuous to the foundation. The steel moment frame columns are supported at the first elevated level by concrete transfer beams and columns along the south perimeter and by CMU walls adjacent to the steel column bases at the north and east perimeters. At the south perimeter, the CMU shear walls are directly adjacent to the concrete beam and columns supporting the moment frames.

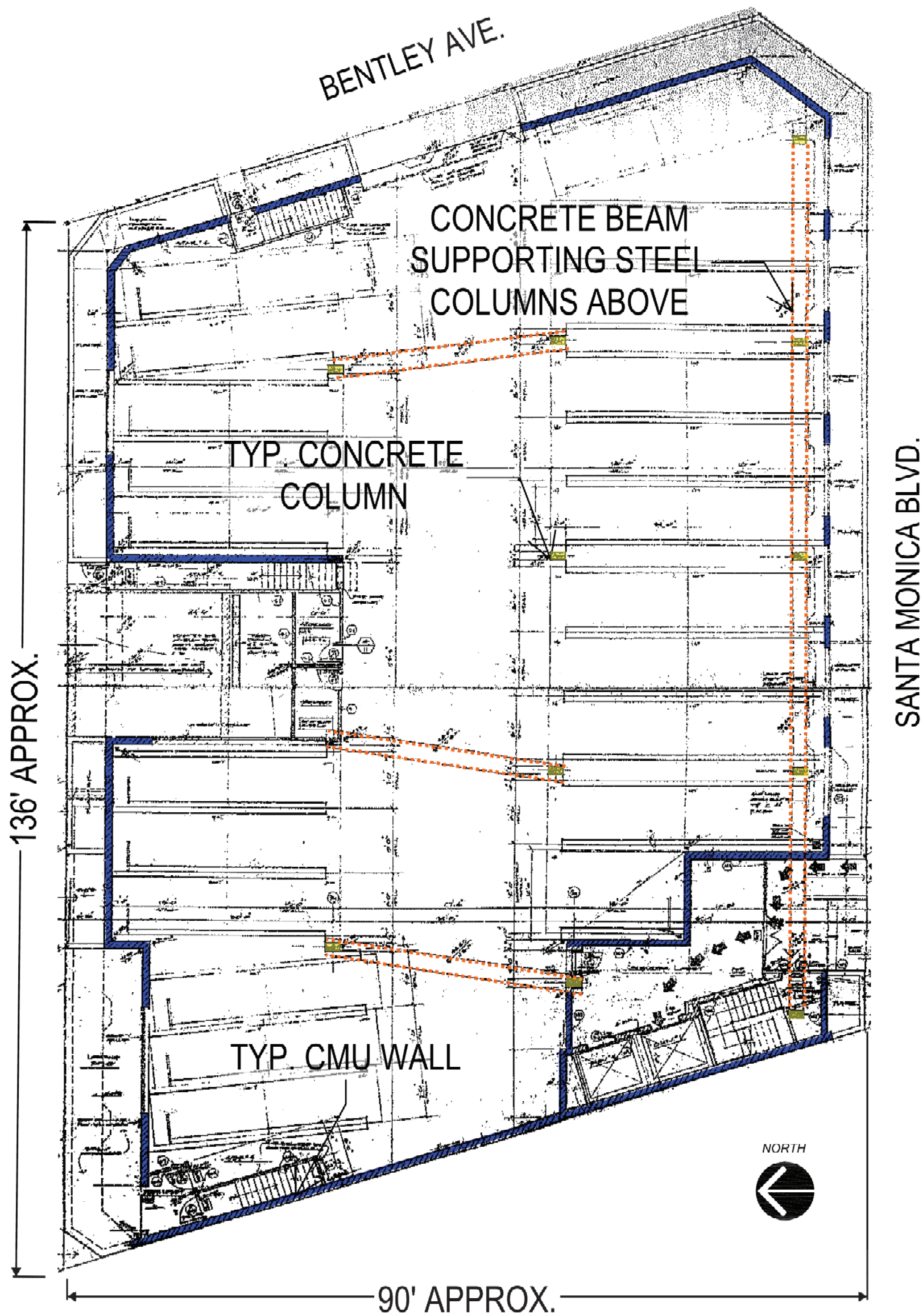


Figure 2.1 - At-Grade Parking Level Floor Plan

Figure 2.2 - Partial South Elevation

3.0 SEISMIC EVALUATION METHODOLOGY

An ASCE 41-13 *Tier 1 Screening* forms the basis of the first step of the seismic evaluation and consists of checklists that allow identification of potential deficiencies of the building based on the lateral force resisting system type. It provides a qualitative review of the structure's performance under an established performance level.

ASCE 41-13 defines *Basic Performance Objectives for Existing Buildings (BPOE)* depending on the building *Risk Category* based on Table 1604.5 of CBC 2016. A BPOE of *Life Safety (S-3)* under the BSE-1E hazard level per Table 301.1.4.2 of CBC 2016 has been defined for this structure. An explanation of hazard levels can be found in Section 4.

Tier 1 checklists applicable to this structure were completed for the appropriate performance level to assist in developing an opinion about the seismic performance of the building. Checklist items were marked as compliant, non-compliant, unknown or not applicable. Section III.B.3, Footnote 2, of the *UC Seismic Safety Policy* excludes non-structural components from the determination of the seismic performance rating.

Potential deficiencies discovered in a Tier 1 evaluation require further study through a Tier 2 deficiency-based evaluation. Additional analysis and evaluation of each potential deficiency were completed in accordance with Tier 2 procedures to either confirm the deficiency or demonstrate the adequacy of the structure as it relates to the potential deficiency. For instance, structural analysis models were developed using the structural analysis software package *SAP2000* to study the steel moment frames with fully restrained connections that predate the 1994 Northridge Earthquake.

Based upon the results of the Tier 1 review, Tier 2 analyses and our field observations, a seismic performance level was determined for the building based on the UC Seismic Safety Policy Table A.1 shown below in Table 3.1

Table 3.1 Determination of Expected Seismic Performance Level (UC Seismic Safety Policy)

Table A.1. Determination of Expected Seismic Performance Level[1] Based on the Edition, California Code of Regulations, Part 10, California Building Code (CBC) (current edition)

Definitions based upon California Building Code (CBC) requirements for seismic evaluation of buildings using Risk Categories of CBC Table 1604A.5, depending on which applies, and performance criteria in CBC Table 317.5[2]	Expected Seismic Performance Level [1]
A building evaluated as meeting or exceeding the requirements of CBC Part 10 Chapter 3 for Risk Category IV performance criteria with BSE-1N and BSE-2N hazard levels replacing BSE-R and BSE-C as given in Chapter 3.	I
A building evaluated as meeting or exceeding the requirements of CBC Part 10 Chapter 3 for Risk Category IV performance criteria.	II
A building evaluated as meeting or exceeding the requirements of CBC Part 10 Chapter 3 for Risk Category I-III performance criteria with BSE- 1N and BSE-2N hazard levels replacing BSE-R and BSE-C respectively as given in Chapter 3; alternatively, a building meeting CBC requirements for a new building.	III
A building evaluated as meeting or exceeding the requirements of CBC Part 10 Chapter 3 for Risk Category I-III performance criteria.	IV
A building evaluated as meeting or exceeding the requirements of CBC Part 10 Chapter 3 for Risk Category I-III performance criteria only if the BSE-R and BSE-C values are reduced to 2/3 of those specified for the site.	V
A building evaluated as not meeting the minimum requirements for Level V designation and not requiring a Level VII designation.	VI
A building evaluated as posing an immediate life-safety hazard to its occupants under gravity loads. The building should be evacuated and posted as dangerous until remedial actions are taken to assure the building can support CBC prescribed dead and live loads.	VII

Notes:

1. Expected seismic performance levels are indicated by Roman numerals I through VII. Assignments are to be made following a professional assessment of the building's expected seismic performance as measured by a CSE's experience or referenced technical standard and earthquake ground motions. Equivalent Arabic numerals, fractional values, or plus or minus values are not to be used. These assignments were prepared by a task force of state agency technical personnel, including the California State University, the University of California, the California Department of General Services, the Division of the State Architect, and the Administrative Office of the Courts. The levels apply to structural and non-structural elements of the building as contained in Chapter 3, CBC Part 10 requirements. These definitions replace those previously used by these agencies.

2. Chapter 3 of the California Building Code Part 10, current edition, regulates existing buildings. It uses and references the American Society of Civil Engineers Standard *Seismic Rehabilitation of Existing Buildings, ASCE-41-13*. All earthquake ground motion criteria are specific to the site of the evaluated building. The CBC definitions for earthquake ground motions to be assessed are paraphrased below for convenience:

3. BSE-2N, the 2,475-year return period earthquake ground motion, or 150% of the Maximum Considered Earthquake ground motion for the site.

   BSE-C, the 975-year return period earthquake ground motion.

   BSE-1N, two-thirds of the BSE-2N, nominally, the 475-year return period earthquake ground motion. BSE-R, the 225-year return period earthquake ground motion.

   Risk Category is defined in the CBC Table 1604A.5. The risk category sets the level of required seismic building performance under the CBC. Risk Category IV includes acute care hospitals, fire, rescue and police stations and emergency vehicle garages, designated emergency shelters, emergency operations centers, and structures containing highly toxic materials where the quantities exceed the maximum allowed quantities, among others. Risk categories I-III includes all other building uses that include most state-owned buildings.

4.0 SITE SEISMICITY

4.1 Ground Motion Estimates

A geotechnical report was not provided for review. Site geotechnical conditions were assumed to be consistent with Site Class D (ASCE 41, Section 2.4.1.6.1), and ground motion information was obtained from probabilistic hazard mapping software developed by the United States Geological Survey (USGS).

Part 10 of the California Building Code (CBC) regulates existing buildings. CBC Chapter 3 references American Society of Civil Engineers *Seismic Evaluation and Retrofit of Existing Buildings*, ASCE-41, as the standard for evaluating existing buildings. The CBC definitions for earthquake ground motions to be assessed are summarized below for convenience.

- BSE-2: the 2,475-year return period earthquake ground motion, or the 150% of the Maximum Considered Earthquake ground motion for the site.
- BSE-C: the 975-year return period earthquake ground motion.
- BSE-1: two-thirds of the BSE-2, nominally, the 475-year return period earthquake ground motion.
- BSE-R: the 225-year return period earthquake ground motion. This is similar to BSE-1E per ASCE 41-13

CBC Table 317.5 identifies the earthquake hazard to be used when seismically evaluating a building. Spectral accelerations were obtained from the USGS for the BSE-R hazard level (i.e., Basic Safety Earthquake-1, BSE-1E). Figure 4.1 shows the design spectra for the BSE-1E hazard level. The BSE-1E hazard level corresponds to an earthquake with an average return period of 225 years or 20% probability of exceedance in 50 years. BSE-1E spectral accelerations are used to construct the response spectra for investigating whether the building satisfies the ASCE 41 Life Safety performance objectives, as specified in CBC Table 301.1.4.2.

Spectral ordinates for the BSE-1E hazard are:

- S_{XS} = 0.917g
- S_{X1} = 0.512g

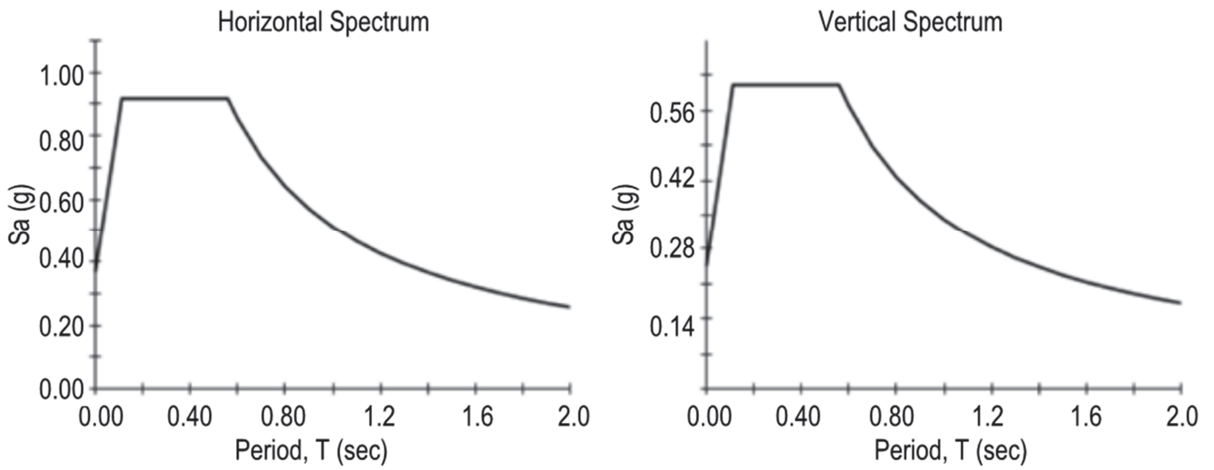


Figure 4.1 - Design Spectra for the BSE-1E Hazard Level per USGS

Based on the 0.2 second and 1.0 second spectral accelerations, ASCE 41-13 Table 2-4 identifies the level of seismicity at this site as “High.”

4.2 Seismic or Geotechnical Hazards

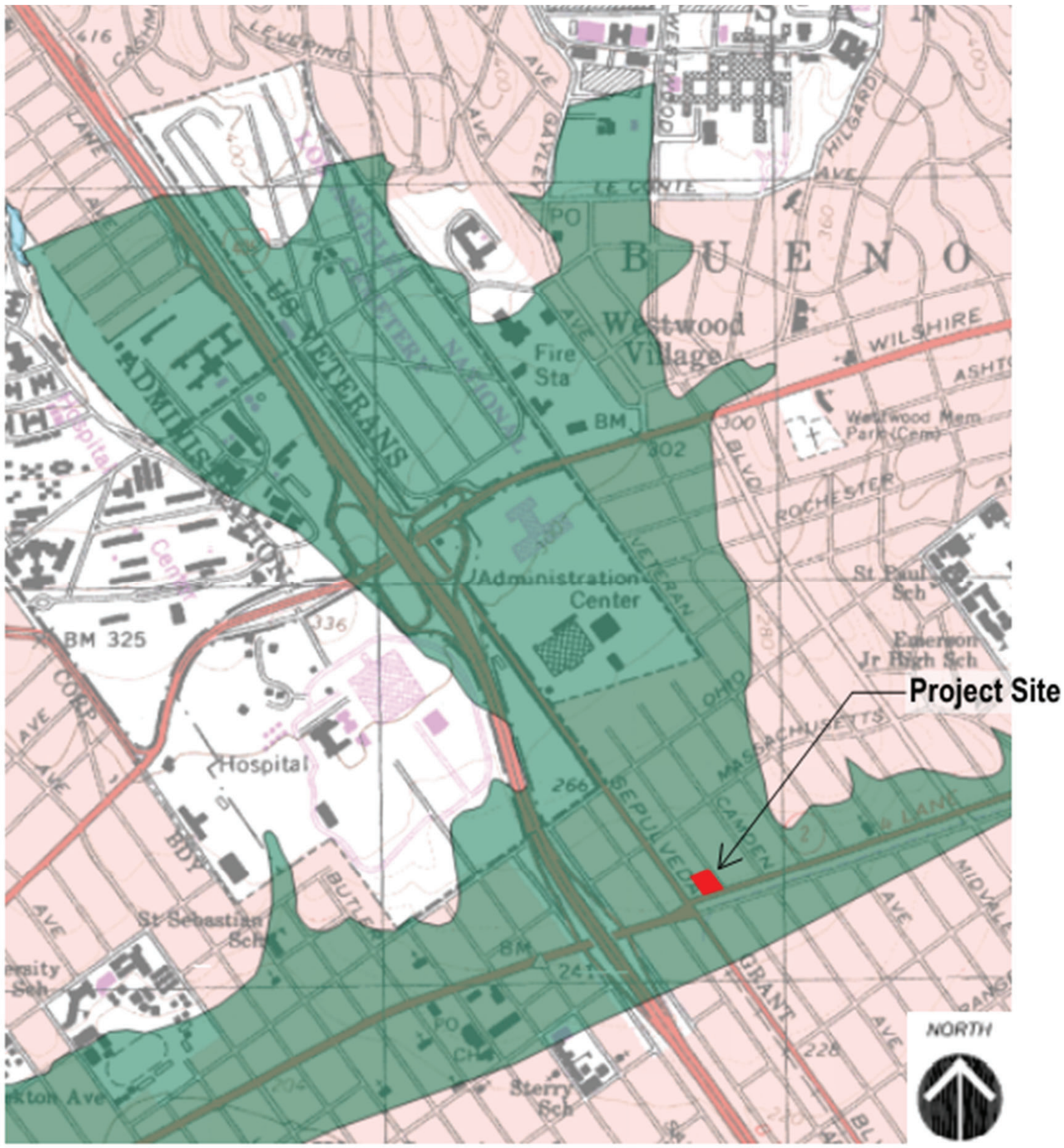
The State of California has issued a set of regulatory maps detailing regions of potential liquefaction, landslide and ground fault rupture. Landslide and ground fault rupture have not been identified as potential hazards at the site; however, in 2015 the California Geological Survey published a map identifying the Beverly Hills Preliminary Fault Rupture Study Area, the southern boundary of which just includes the subject building, as shown in Figure 4.2. Although the map has not been finalized, the City of Los Angeles currently requires that proposed development projects within this zone perform a fault rupture study.

The building falls in a liquefaction study zone identified by the *California Geological Survey – Earthquake Zones of Required Investigation (Beverly Hills Quadrangle).* A portion of this map is shown in Figure 4.3. The significance of both potential hazards is discussed in Section 5.

Map Explanation

City of Los Angeles Preliminary Fault Rupture Study Areas: zones where active faults may exist and present a potential for surface ground rupture to occur during a local earthquake.

Figure 4.2 Beverly Hills Preliminary Fault Rupture Study Zone



Map Explanation

Liquefaction Zones: areas where historical occurrence of liquefaction, or local geological, geotechnical and ground water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

Figure 4.3 - California Geological Survey – Earthquake Zones of Required Investigation (Beverly Hills Quadrangle)

5.0 Seismic Evaluation

5.1 ASCE 41-13 Tier 1

Given the date of the building code assumed to govern the design of the seismic force resisting system (SFRS) (i.e., the 1979 Uniform Building Code), the building does not satisfy the requirements of a benchmark building. Generally speaking, a benchmark building is deemed to satisfy the specified seismic performance levels and no additional review is required. In the case of this building, a Tier 1 analysis is required.

Tier 1 checklists were completed for the SFRS for the applicable building types in ASCE 41-13 Table 3-1. Below the first elevated level, the seismic force resisting system (SFRS) consists of *RM2: Reinforced Masonry Bearing Walls with Rigid Diaphragms.* Above the first elevated level, the building uses a horizontal combination of two SFRS in the north-south direction: *S1A: Steel Moment Frames with Flexible Diaphragms* and *RM1: Reinforced Masonry Bearing Walls with Flexible Diaphragms*. In the east-west direction, the SFRS consists of only Type *S1A*. Evaluation of this building using Tier 1 and Tier 2 procedures is permitted by ASCE 41-13 Section 3.3.1.2.2.1, even when a horizontal combination is present, because the building complies with the requirements specified in this section.

The following Tier 1 checklists were completed:

- 16.1.2LS: Life Safety Basic Configuration Checklist.
- 16.4LS: Life Safety Structural Checklist for Building Type S1A: Steel Moment Frames with Flexible Diaphragms.
- 16.15LS: Life Safety Structural Checklist for Building Types RM1: Reinforced Masonry Bearing Walls with Flexible Diaphragms and RM2: Reinforced Masonry Bearing Walls with Stiff Diaphragms.

All the checklist items were found to comply or not be applicable to this building with the exceptions identified below. A non-compliant item does not mean that a structural deficiency necessarily exists, but flags the item for additional review using a Tier 2 evaluation.

5.1.1 Non-Compliant Items for Life Safety Structural Checklist for Building Type S1A: Steel Moment Frames with Flexible Diaphragms

- Moment Resisting Connections: connections predate code requirements that followed the 1994 Northridge Earthquake.

5.1.2 Non-Compliant Items for Life Safety Structural Checklist for Building Types RM1: Reinforced Masonry Bearing Walls with Flexible Diaphragms and RM2: Reinforced Masonry Bearing Walls with Stiff Diaphragms

- Wall Anchorage: connections from exterior CMU walls to the diaphragms are not exposed. ASCE 41-13 Tier 1 suggests the use of a *Quick Check Procedure* to verify the connection strength from the wall to the diaphragm for out-of-plane forces.
- Wood Ledgers and Transfer to Shear walls: detailing for the connection of the wood diaphragm to the CMU walls are not exposed.

5.2 ASCE 41-13 Tier 2 Seismic Evaluation

Based upon on the potential deficiencies outlined in Section 5.1, an analysis was performed to assess their significance on the seismic performance of the structure. The evaluation was performed using the ASCE 41-13 Tier 2 Deficiency-Based Evaluation.

5.2.1 Component Strength versus Acceptance Criteria

ASCE 41-13 classifies actions as deformation-controlled or force-controlled. A deformation-controlled action is defined as an action that has an allowable deformation greater than the deformation associated with the yield strength of a member, and a force-controlled action is defined as an action that has a deformation that is not allowed to exceed the deformation associated with the yield strength of the member. Members with limited ductility are considered force-controlled.

Where actions are considered deformation-controlled, ASCE 41-13 allows the use of a component capacity modification factor, "*m,*" to account for the expected ductility associated with these actions at the selected Performance Level. The demand is then compared with the component's expected strength, calculated using conventional structural engineering methods and standards (e.g., AISC 360 for steel elements), multiplied by a strength reduction factor, ϕ, equal to unity.

For deformation-controlled actions, the equivalent lateral load, Q_{UD}, is equal to:

$$Q_{UD} = Q_G \pm Q_E$$

For deformation-controlled actions, the acceptance criteria are defined as:

$mkQ_{CE} \geq Q_{UD}$

where:

Q_{CE} = Expected strength of the component

m = Component modification factor

k = Knowledge factor

Where actions are considered force-controlled, the demands are compared with the lower bound strength, calculated using conventional structural engineering methods and standards, multiplied by a strength reduction factor, ϕ, equal to unity.

For force-controlled actions, the equivalent lateral load, Q_{UF} , is equal to:

$Q_{UF} = Q_G \pm Q_E/C_1C_2J$

where:

Q_G = Gravity load effect

Q_E = Seismic load effect

C_1 *and* C_2 = Modification factors specified for the structural component being evaluated

J = Force delivery reduction factor

For force-controlled actions, the acceptance criteria are defined as:

$kQ_{CL} > Q_{UF}$

where:

Q_{CL} = Lower bound strength of the component

Although structural drawings are not available for this building, conventional lower-bound material strengths were assumed for the structural steel and masonry. Thus, it is reasonable to use a knowledge factor of 1.0 because the building was constructed under the inspection requirements of the Los Angeles Department of Building and Safety. While this does not guarantee that all aspects of the building code were satisfied, as the post-Northridge Earthquake steel frame investigations revealed, our experience working in the Southern California construction market during the time the building was constructed suggests that under-strength material was not a common problem.

5.2.2 Analytical Results

Based upon the results of examinations and analyses conducted using Tier 2 procedures, non-compliant conditions identified in the Tier 1 Screening were found to comply with the requirements of ASCE 41-13. A discussion is presented for each type of SFRS.

5.2.2.1 S1A: Steel Moment Frames with Flexible Diaphragms

Moment-Resisting Connections: Structural models were developed using the structural analysis software package SAP2000 to study the steel moment frame connections that predate the 1994 Northridge Earthquake, a potential deficiency of the building. Site observations permitted classification of the building's steel beam-column connections as *welded unreinforced flange (WUF)* per ASCE 41-13 Table 9-5, described as *"full-penetration welds between beams and columns flanges, bolted or welded web, designed before code changes that followed the Northridge Earthquake."*

Per ASCE 41-13 Section 9.4.2.4, fully restrained (FR) beam-column connections, such as the ones present in the building, are considered deformation-controlled, and the connection acceptance criteria are dependent on the detailing of continuity plates, the strength of the panel zone, the beam span-to-depth ratio, and the slenderness of the beam and web flanges. A typical beam-column connection for the building is shown in Figure 5.1. A typical elevation of a perimeter moment frame based on field measurements and interpolation assumptions is shown in Figure 5.2.

m-factors for the indicated connections were obtained from ASCE 41-13 Table 9-6 and modified based upon detailing observed on the site using the criteria in ASCE 41-13 Section 9.4.2.4. Connection components were evaluated following the acceptance criteria shown in Section 5.2.1 of this report, and the Tier 2 analysis verified acceptable performance of the steel moment frame beam-column connections under the BSE-R hazard, with demand-over-capacity ratios less than unity for critical limit states.

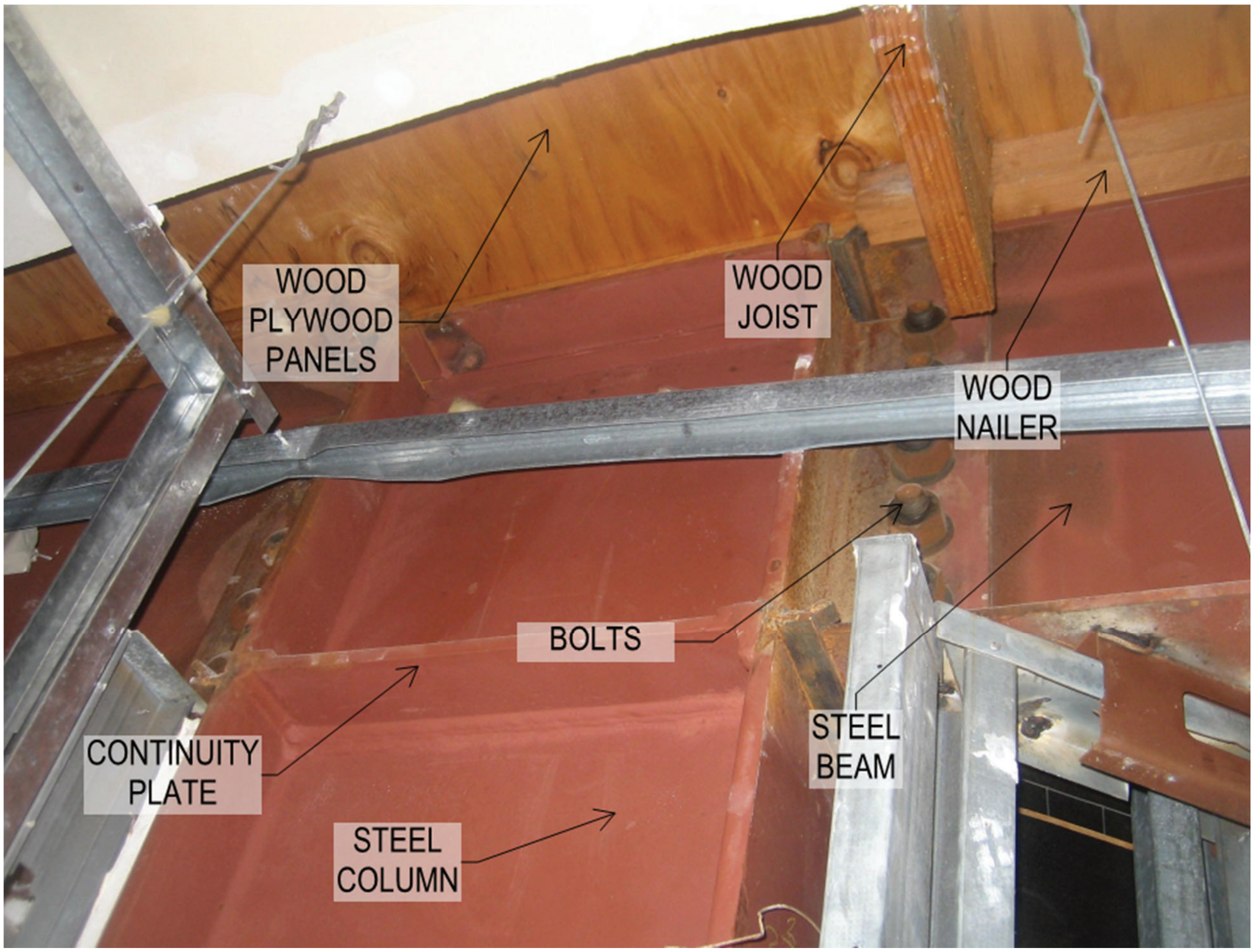


Figure 5.1 - Steel Moment Frame Beam-Column Connection

Drift, column axial stress, and beam flexural stress checks performed as part of the Tier 1 Screening were found compliant when evaluated against the specified earthquake.

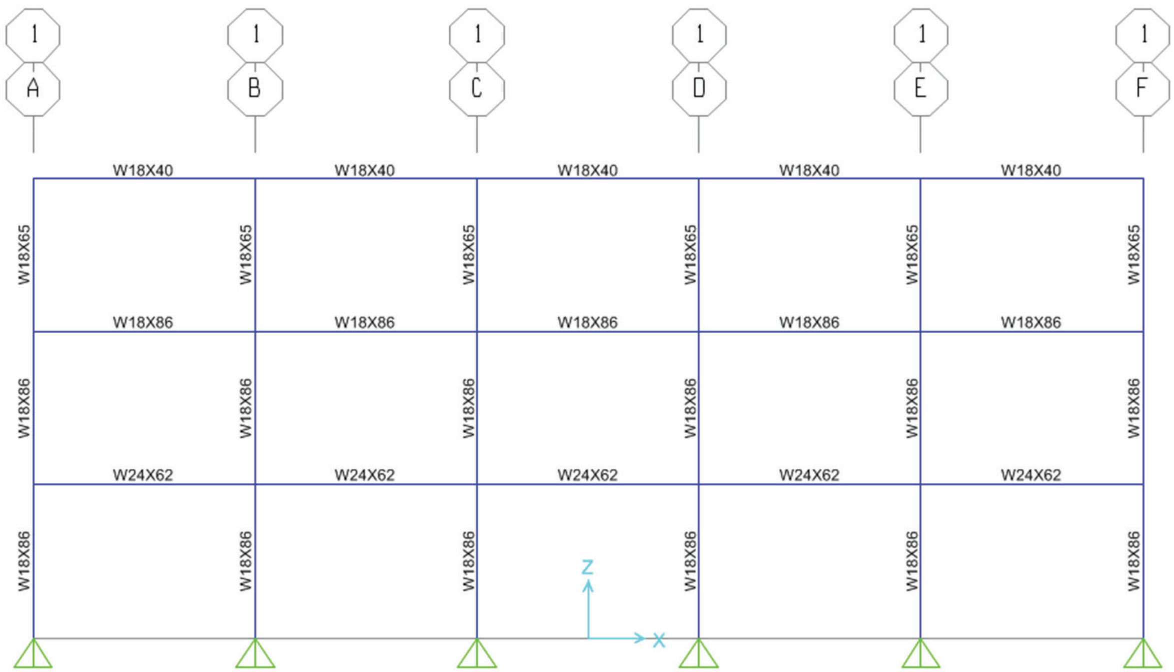


Figure 5.2 - Typical *SAP2000*Perimeter Steel Moment Frame Model

5.2.2.2 RM1: Reinforced Masonry Bearing Walls with Flexible Diaphragms and RM2: Reinforced Masonry Bearing Walls with Stiff Diaphragms

Wall Anchorage: ASCE 41-13 *Tier 1 Screening* establishes a minimum connection force for which CMU wall anchorage to the diaphragm must have sufficient strength to resist. Section 2310 of the Uniform Building Code - 1979 Edition (UBC 1979), establishes that concrete or masonry walls must be anchored to all floors and roof that provide lateral support for the wall and that such anchorage must be capable of resisting specified horizontal forces. A comparison with the minimum connection force established in ASCE 41-13 Tier 1 Screening and the horizontal diaphragm forces established in Section 2312 (j) 2 D of the UBC 1979 (both at service levels) shows that the design force for the wall anchorage is greater than the minimum force required by ASCE 41-13. Based on this comparison, the out-of-plane wall connections are judged to be in conformance with ASCE 41.

Wood Ledgers and Transfer to Shear Walls: Although detailing of the CMU wall-diaphragm connection is not exposed, Section 2312 (j) 3 A of the UBC 1979 requires that where wood diaphragms are used to laterally support concrete or masonry walls, anchorage may not be accomplished using toenails or nails subjected to withdrawal, nor should wood framing be used in cross-grain bending or cross-grain tension. This requirement of UBC 1979 parallels the one in ASCE 41; hence this item is judged as compliant.

The building was constructed under the plan check and inspection requirements of the Los Angeles Department of Building and Safety. While this does not guarantee that all aspects of the building code were satisfied, our experience working in the Southern California construction market during the time the building was constructed suggests that wall-to-diaphragm connections and wall ledger detailing complying with requirements similar to those in ASCE 41-13 were commonly provided in buildings of this type.

The shear stress check for CMU walls, performed as part of the *Tier 1 Screening*, is compliant under the BSE-1E event.

5.3 Other Issues

The building is in the general vicinity of high levels of shaking as well as other steel frame and concrete buildings that sustained significant structural damage during the 1994 Northridge Earthquake (e.g., Olympic Boulevard and Sepulveda, Olympic Boulevard and Barrington Avenue). The USGS classified the Modified Mercalli Intensity at the site on the border between VII and VIII, as shown in Figure 5.3. The site was within the minimum 0.2g ground acceleration region within which a 1995 City of Los Angeles ordinance required earthquake inspection of steel frame buildings following the Northridge Earthquake. A search of public records showed no permit issued for repair of the steel frame, although many building permits were issued for tenant improvements in the years following the Northridge Earthquake. Based on the lack of observable repair work on the building exterior and the lack of a building permit for steel frame damage repair, it appears that no damage to the steel frame was found following the Northridge Earthquake.

The building is in a preliminary fault rupture study zone, albeit at the very southern edge. No geotechnical report for the project has been located, although it is unlikely that in the early 1980s a fault rupture study would have been undertaken. Public records do not report evidence of faulting discovered during the excavation of the subterranean parking. A search of public records for a high-rise building with a basement, constructed one block away from the subject building in the mid-1980s (11111 Santa Monica Blvd.), also does not reveal evidence of faulting discovered during the excavation of the basement. Based on the location of the subject building relative to the southern boundary of the preliminary fault rupture study zone and the lack of evidence of reported faulting discovered during excavation activities at this and an adjacent site, it appears unlikely that the building will be subject to significant effects of ground rupture.

Although the building is in a liquefaction study zone, it is one block to the southern edge of the study zone. A review of other foundation systems in the general vicinity did not suggest the presence of liquefiable soils based on foundation type, and no liquefaction was reported during the 1994 Northridge Earthquake. In addition, while liquefaction may increase the amount of structural damage, it is not believed to represent a life-safety hazard for this building.

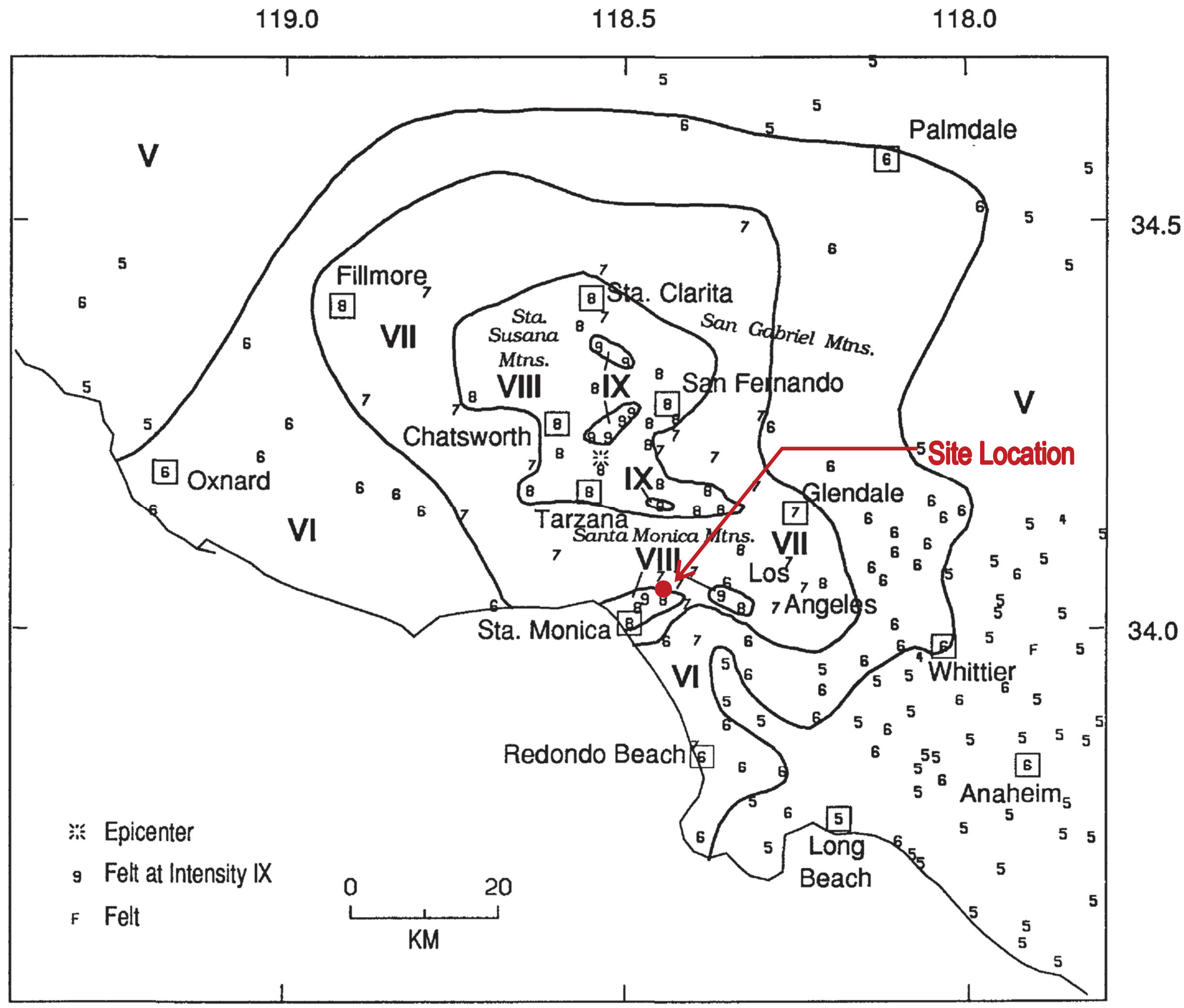


Figure 5.3 – Distribution of Modified Mercalli intensities in the epicentral region. Roman numerals give average intensities within isoseismals. Arabic numerals represent intensities at specific locations. Squares denote towns labeled in the figure.

Source: Dewey, J., Glen Reagor, B., Dengler, L., Moley, K. "Intensity Distribution and Isoseismal Maps for the Northridge, California, Earthquake of January 17, 1994." U.S. Department of the Interior, U.S. Geological Survey, 1995, p. 3.

6.0 CONCLUSIONS

Based on an evaluation of the building using ASCE 41-13 Tier 1 Checklists, the results from a Tier 2 analysis, site observations, the lack of building permit records for damage repair following the 1994 Northridge Earthquake and the requirements of 2016 CBC Part 10, we recommend a Seismic Performance Rating of **Level IV** as defined by the UC Seismic Safety Policy Table 3.1.

TAS:cc:klc