Appendix I

Los Angeles Conservancy Comment Review

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May 25, 2023

via email: Ed.Casey@alston.com

Mr. Ed Casey
Alston & Bird LLP
333 S. Hope Street, 16th Floor
Los Angeles, California 90071

Regarding: Barry Building (11973 San Vicente Boulevard, Los Angeles, CA 90049)
Los Angeles Conservancy Comments Review

Dear Mr. Casey,

Per your request, we have reviewed the comments generated by the Los Angeles Conservancy and Corin Kahn in form of letter dated April 18, 2023, regarding the Barry Building located at 11973 San Vicente Boulevard, Los Angeles, CA 90049. Our review was limited to Comments No. A3-4 and A3-5 stated below.

Conservancy Comment No. A3-4

II. Demolition by neglect is being used as a tactic to circumvent and piecemeal historic preservation regulations and CEQA.

This comment suggests that the seismic instability of the Barry Building is due to neglect in maintenance and repair of the building. In response, it is our opinion that the identified seismic deficiencies in the building are not result of the owner's negligence in proper maintenance of the building. Instead, the deficiencies are due to the design of the building when it was built in the early 1950s. Buildings designed and constructed at that time had low seismic demands and requirements. Today the demands are much higher. So, in addition to strengthening the existing shear walls in the building, new (not replacement) shear walls and steel moment frames would need to be added, specifically 20 new (and additional) two-story shear walls and three new (and additional) steel moment frames would need to be added to the building to meet today's seismic standards. The absence of such shear walls and moment frames is not due to lack of maintenance and repair.

Conservancy Comment No. A3-5

III. Refusal to comply with City's mandatory soft-story seismic retrofit ordinance(s) is no excuse for approval to demolish.

In addition, to saying that the owner of the Barry Building has not performed a seismic retrofit in accordance with the City's soft story ordinance, this comment also makes these statements—

"City ordinance 183893 (approved November 15, 2015) and 184081 (approved February 1, 2016) that outline the City's mandatory soft-story seismic retrofit requirements allow for flexibility and specifically call out "qualified historic buildings" and state they "shall comply with requirements of the California Historical Building Code established under Part 8, Title 24 of the California Code of Regulations." This provides additional flexibility should owners pursue this option.

Within the Draft EIR and Alternatives section, statements are made that the soft-story seismic retrofit requirements only apples to the south wing on the building, and does not affect the east, north or west wings of the building. While additional structural deficiencies may be needing to be addressed there, there is no limitation to completing this scope. This demonstrates the required work is isolated and therefore can be effectively addressed to meet the City's order to comply without calling for the demolition of the Barry Building."

Englekirk Structural Engineers performed a seismic assessment of the Barry Building using the requirements outlined in ASCE 41-13, in June of 2022. Our findings and proposed retrofit scheme were summarized in the report dated June 6, 2022 (reference Exhibit A). In addition to the seismic retrofit work identified for the south wing of the building, the report also determined that the north, east, and west wings range are 230% - 650% overstressed. The report identified specific seismic retrofit work for those wings, including new and strengthened wood shear walls, new foundations to support the seismic loads resisted by the new shear walls, and adding and strengthening the first floor, second floor, and roof diaphragms.

As stated in the report, based on our evaluation per the ASCE/SEI 41-13 Tier 1 checklist, the seismic force resisting system of the subject property is generally highly overstressed. The analysis indicates very high demand over capacity ratios for all parts of the existing building. These high ratios indicate that the building is likely to suffer significant damage when subject to a moderate to strong earthquake in the Los Angeles basin. Some portions of the building have no significant seismic resisting elements that can resist the seismic forces from the roof and second floor and can result in a possible collapse when subject to a moderate to strong earthquake. These structural deficiencies represent life safety hazards to occupants in and around the building. Reference Section 5 and 6 of the report for complete list of deficiencies.

A substantial portion of the seismic retrofit work identified in the reports would still be needed if the seismic requirements in the California Historical Building Code were applied. Under that Code, a historical building shall be retrofitted to meet 75% of the current building code forces. However, due to the very high level of overstress in the building, 230% to 650% in the structural members, a substantial portion of the work would still be required. Strengthening of existing shear walls and floor/roof plywood diaphragm, additional shear walls and moment frames would still have to be added.

Finally, as noted by another commentor (Corin Kahn), a simple series of temporary wooden frames is not a valid retrofit option because it would not meet current requirements under either the Uniform Building Code or the Historical Building Code.

Englekirk
STRUCTURAL ENGINEERS

Respectfully submitted,

V. Volnyy

Vladimir Volnyy, PE, SE
Principal

VV:gh

Attachments: Exhibit A – ASCE 41-13 Seismic Assessment

EXHIBIT A
ASCE 41-13
Seismic
Assessment
(June 6, 2022)

11973 San Vicente Boulevard

ASCE 41-13 Seismic Assessment

Los Angeles, California

REGISTERED PROFESSIONAL ENGINEER
RUSSELL TANOUYE
NO. 2850
STRUCTURAL
STATE OF CALIFORNIA

06/06/2022

Englekirk
STRUCTURAL ENGINEERS

June 6, 2022

Job No. 12-L038B

11973 San Vicente Boulevard

ASCE 41-13 Seismic Assessment

Los Angeles, California

Submitted to:

Alston & Bird LLP
333 South Hope Street
16th Floor
Los Angeles, CA 90071
(213) 576-2526
Attn: Mr. Greg Berlin

June 6, 2022

Job No. 12-L038B

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1.0 INTRODUCTION

This report summarizes findings of the Seismic assessment per ASCE 41-13 (Tier 1) for the existing building located at 11973 San Vicente Boulevard. A seismic retrofit scheme was also developed for the report, based on ASCE 41-13.

This building is also considered a Historical Building and thus can be considered to be subject to the 2016 California Historical Building Code.

2.0 INFORMATION REVIEWED

Existing building plans were provided to our office. The existing building plans were prepared by Milton Caughey Architect for the “Office and Store Building Mr. David Barry” building. There is no construction date shown on these plans. These plans include Sheets 1 through 8, and include the foundation plan and typical framing sections. Based on the site visit performed on March 27, 2012, the existing building condition generally matched the existing building plans. Some discrepancies were observed. These discrepancies include new windows, new doorways, and modified interior demising walls. These discrepancies appear to have been created due to various tenant improvement revisions during the life of the building. This report was performed as an observation of the visible portions of the building and based on the available drawings. No destructive testing was performed.

3.0 BUILDING STRUCTURAL DESCRIPTION

The existing building is a two-story wood framed structure. The floor plan is 100’ x 107’ with an open 43’ x 56’ courtyard. The courtyard essentially separates the building into four wings. The north and south wings at the second floor and roof are raised by about 1’-6” from the east and west wings. This essentially creates four separate structural building elements with no common floor or roof diaphragm.

The first floor consists of a 4” concrete slab on grade. The second floor system consists of a 2” diagonal sheathed wood floor supported by sawn lumber joists. The roof system consists of 1” diagonal sheathing supported by sawn lumber joists. Both the floor and roof levels have a ceiling. Typical bearing walls are 2x4 studs. The story height is about 12’ at the first floor and 11’-6” at the second floor.

The lateral bracing for this building consists of the horizontal floor and roof diaphragms and the perimeter vertical shear walls. The second floor and roof consist of diagonal sheathing. The nailing pattern for the sheathing is unknown. This diagonal sheathed floor and roof diaphragm span to the exterior perimeter walls. These exterior walls serve as the vertical shear walls that brace this building. The interior demising walls do not form a complete lateral bracing system as they are discontinuous between floors, and several of these walls have been removed and the wall locations are irregularly distributed.

The foundation system consists of continuous and spread footings that bear on the foundation soil. The plans note that the design bearing pressure is 2,000 psf. The bearing walls are founded on an 8" continuous stem wall which is then supported on a 16" wide x 8" deep continuous footing.

The south wing that faces San Vicente Boulevard utilizes a pass-through at the ground floor that accesses the interior courtyard. As a result, there are no bearing walls that extend to the foundation. Instead, the second floor is supported on a series of steel columns. There are some exterior walls on the eastern side, but they are discontinuous between floors.

4.0 SEISMICITY

4.1 Ground Motion Estimates for Seismic Review (ASCE 41-13)

A geotechnical report was not provided for review. Site geotechnical conditions were assumed to be consistent with Site Class D. The spectral accelerations were obtained from probabilistic hazard mapping software developed by the United States Geological Survey (USGS).

Spectral accelerations were obtained from the USGS for the Basic Safety Earthquake-1E (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 evaluate the level of seismicity of the site as required for the Tier 1 Checklist. The ordinates are illustrated in Figure 4.1.

Base on the 0.2 second and 1.0 second spectral accelerations, in accordance with ASCE 41 Table 2-4, the level of seismicity at this site is defined as High. This classification determines the ASCE 41-13 structural checklists required for use in evaluating the building.

4.2 Seismic or Geotechnical Hazards

The state of California has issued a set of regulatory maps detailing regions of potential liquefaction, landside and ground fault rupture. This site is in the Beverly Hills Quadrangle, as shown in Figure 4.2. Areas shown in white have not been identified as locations of potential liquefaction, landside or ground fault rupture. The map indicates that the site, shown in Figure 4.2, has not been identified as a potential location for any of these seismic or geotechnical hazards.

5.0 SEISMIC EVALUATION SUMMMARY

5.1 ASCE 41-13 Tier 1

The building site is classified as “high seismicity” and in accordance with Tier 1 evaluation requirements, the following checklists were reviewed, and applicable “quick checks” were performed:

16.1 Basic Checklist
16.1.2LS Life Safety Basic Configuration Checklist
16.3LS Life Safety Structural Checklist for Building Type W2: Wood Frames, Commercial and Industrial

A copy of the checklists is found in Appendix A. A summary is provided in Table 5.1 below for items that were found “Non-Compliant” or “Unknown”.

Table 5.1: Summary of Checklist Findings

16.1 Basic Checklist		
Item	**Non-Compliant/Unknown**	**Description**
Load Path	Non-Compliant	Discontinuous horizontal diaphragms occur at second floor and roof. Vertical elements of seismic-force-resisting system (such as wood shear walls or frames) were not found at all sides of the perimeter. Interior demising walls do not form a complete seismic-force-resisting system as they are discontinuous between floors.
16.1.2LS Life Safety Basic Configuration Checklist		
Item	**Non-Compliant/Unknown**	**Description**
Load Path	Non-Compliant	See 16.1 for Description
Weak Story	Non-Complaint	Vertical discontinuities of seismic-force-resisting system were not found at all sides of the perimeter. Interior demising walls do not form a complete lateral bracing system as they are discontinuous between floors.

Soft Story	Unknown	Stiffness of the seismic-force-resisting system cannot be confirmed, as the seismic-force-resisting system (wood shear walls) are not found at all sides of perimeter, and wood shear walls are found discontinuous between floors.
Vertical Irregularities	Non-Complaint	Vertical elements of seismic-force-resisting system (Wood shear walls) were found discontinuous between floors.
Torsion	Unknown	The story center of rigidity cannot be confirmed.
Overturning		
16.3LS Life Safety Checklist for Building Type W2: Wood Frames, Commercial and Industrial		
Item	**Non-Compliant/Unknow**	**Description**
Redundancy	Non-Complaint	Vertical discontinuities of seismic-force-resisting system were not found at all sides of the perimeter.
Shear Stress Check	Non-Complaint	The shear stress check provides an assessment of the overall level of demand on the structure. Existing shear walls are found to be overstressed.
Stucco (Exterior Plaster) Shear Wall	Unknown	Plywood sheathing on existing exterior wall shear walls cannot be confirmed. Existing shear walls could be a stucco shear wall
Gypsum Wallboard or Plaster Shear Walls	Non-Complaint	Existing interior demising walls are found to be Gypsum board.
Narrow Wood Shear Walls	Non-Compliant	Existing shear walls were found with an aspect ratio less than 2-to-1.

6.0 VOLUNTARY SEISMIC EVALUATION

Based on the potential deficiencies outlined in Section 5.1, additional analyses were performed to review the elements of the seismic-force-resisting system. Shear stress of shear walls and diaphragms were reviewed. The Basic Safety Earthquake-1E (BSE-1E) hazard level per ASCE/SEI 41-13 was used to determine building element ‘demand over capacity ratios’ (DCRs). These ratios compare the seismic demand versus the estimated capacity to provide a comparative estimate as to what level these building elements are overstressed. The lateral capacity of existing building elements is based on ASCE 41-13 Table 12-1, “The Default Expected Strength Values for Wood and Light Frame Shear Walls,” and Table 12-2, “The Default Expected Strength Values for Wood Diaphragms.”

The existing building geometry structurally separates the building into four separate wings. Discontinuities at the second floor and roof occur at each wing interface, thereby creating discontinuous horizontal diaphragms between each wing. Because they are separate wings, each wing cannot rely on the adjacent wings to resist seismic loads. Therefore, each wing was evaluated individually.

6.1 North Wing

In the north-south direction, roughly 120 feet of existing walls are located, such that they act as lateral resisting elements. In the east-west direction, roughly 42 feet of existing walls are located, such that they act as lateral resisting elements. The DCR for the walls in the north-south direction is 230% overstressed. The DCR for the walls in the east-west direction is 650% overstressed.

6.2 East Wing

In the north-south direction, there is no existing wall located as a lateral resisting element. The exterior wall along grid H and the interior courtyard wall along grid G do not contain structural elements that can be identified as a lateral resisting element. In the east-west direction, roughly 90 feet of existing walls are located as lateral resisting element. The DCR for walls in the north-south direction cannot be determined since no lateral resisting element can be identified. Significant lateral displacement may be expected in the north-south direction of the east wing during a seismic event. The DCR for walls in the east-west direction is 190% overstressed.

6.3 South Wing

There is no existing wall or lateral resisting element to resist seismic loads from the second floor and roof in either the north-south or east-west directions. As a result, significant lateral displacement may be expected during a seismic event. The steel posts that support this wing will be subjected to this potential lateral displacement. Since the steel posts do not possess any lateral resistance, a possible collapse of this wing can result during a seismic event.

6.4 West Wing

In the north-south direction, roughly 50 feet of existing walls are located, such that they act as a lateral resisting element. In the east-west direction, roughly 40 feet of existing walls are located, such that they act as a lateral resisting element. There is no wall located at the south end of the wing. Significant lateral displacement may be expected in the east-west direction during a seismic event. The DCR for the walls in the north-south direction is 360% overstressed. The DCR for the walls in the east-west direction is 400% overstressed.

6.5 Typical Existing Roof and Floor Diaphragm

The DCR for the typical diaphragm at the roof and second floor is highly overstressed. Diaphragm shear stress cannot be determined at areas where vertical seismic-force resisting elements are not found.

7.0 Voluntary Seismic Retrofit Scheme

To conform to the seismic force resisting requirements for a new structure, we propose a seismic retrofit scheme that includes strengthening the existing walls, adding new 2-story shear walls, and new steel moment frames. (See Figure 7.1 for conceptual shear wall and steel moment frame locations)

7.1 Strengthening Existing Shear Wall

The existing shear walls need to be continuous between floors. The strengthening requirements include adding new plywood sheathing and nailing, new hold-down anchors at each end of the wall, new floor to wall connection, and new footing/enhancing for the existing footing.

New Shear Wall: New wood shear walls need to be continuous between floors. The new wood shear wall construction includes new 2x stud wall framing, new plywood sheathing and nailing, new hold-down anchors at each end of the wall, and new footing.

New Floor and Roof Diaphragm Sheathing: New ¾" plywood sheathing over the entirety of the existing floor and roof sheathing.

Steel Moment Resisting Frame: Two-story steel moment resisting frames are to be introduced at the south wing where no continuous shear wall may be feasible. The steel moment resisting frames consist of new wide flange steel columns, wide flange steel beams, and new concrete footings.

Consideration for Reducing Impact of Retrofit on Historical Fabric: The above seismic retrofit can be done to minimize the impact on the building historic fabric. The addition of new plywood shear walls can be performed on the inside force of the exterior walls to avoid removing or damage the exterior skin. The new walls can be located to avoid closing any existing historic windows. The new steel moment resisting frames that are located at the front wing can be placed interior to the building footprint. The second floor and roof diaphragm will require enhanced nailing to allow the adjustment of the frame relocations.

Seismic Retrofit Cost: The cost to retrofit the building can vary, depending on the specific repair details, sequencing, and potential unforeseen conditions. We estimate the retrofit cost will be about $2.0M to $2.5M. This cost does not include any costs such as possible code required upgrades such as the American Disability Act (ADA), plumbing, mechanical, lighting, etc. Also, the addition of new shear walls may render portions of the building less rentable because of the shear wall obstruction at storefront windows, office windows, etc.

8.0 CONCLUSIONS

Based on our evaluation per the ASCE/SEI 41-13 Tier 1 checklist, the seismic force resisting system of the subject property is generally highly overstressed.

The analysis indicates high demand over capacity ratios for all parts of the existing building. These high ratios indicate that the building is likely to suffer significant damage when subject to a moderate to strong earthquake in the Los Angeles basin. Some portions of the building have no significant seismic resisting elements that can resist the seismic forces from the roof and second floor and can result in a possible collapse when subject to a moderate to strong earthquake. These structural deficiencies represent life safety hazards to occupants in and around the building. The above mentioned seismic retrofits would correct the structural deficiencies identified in this report.

The California Historical Building Code allows an analysis and retrofit to meet 75% of the current building code forces. A direct comparison of this force level to ASCE 41-13 was not performed. However, based on the level of overstress, it is our opinion that the same conclusion and retrofit recommendations will apply.

RT:gh

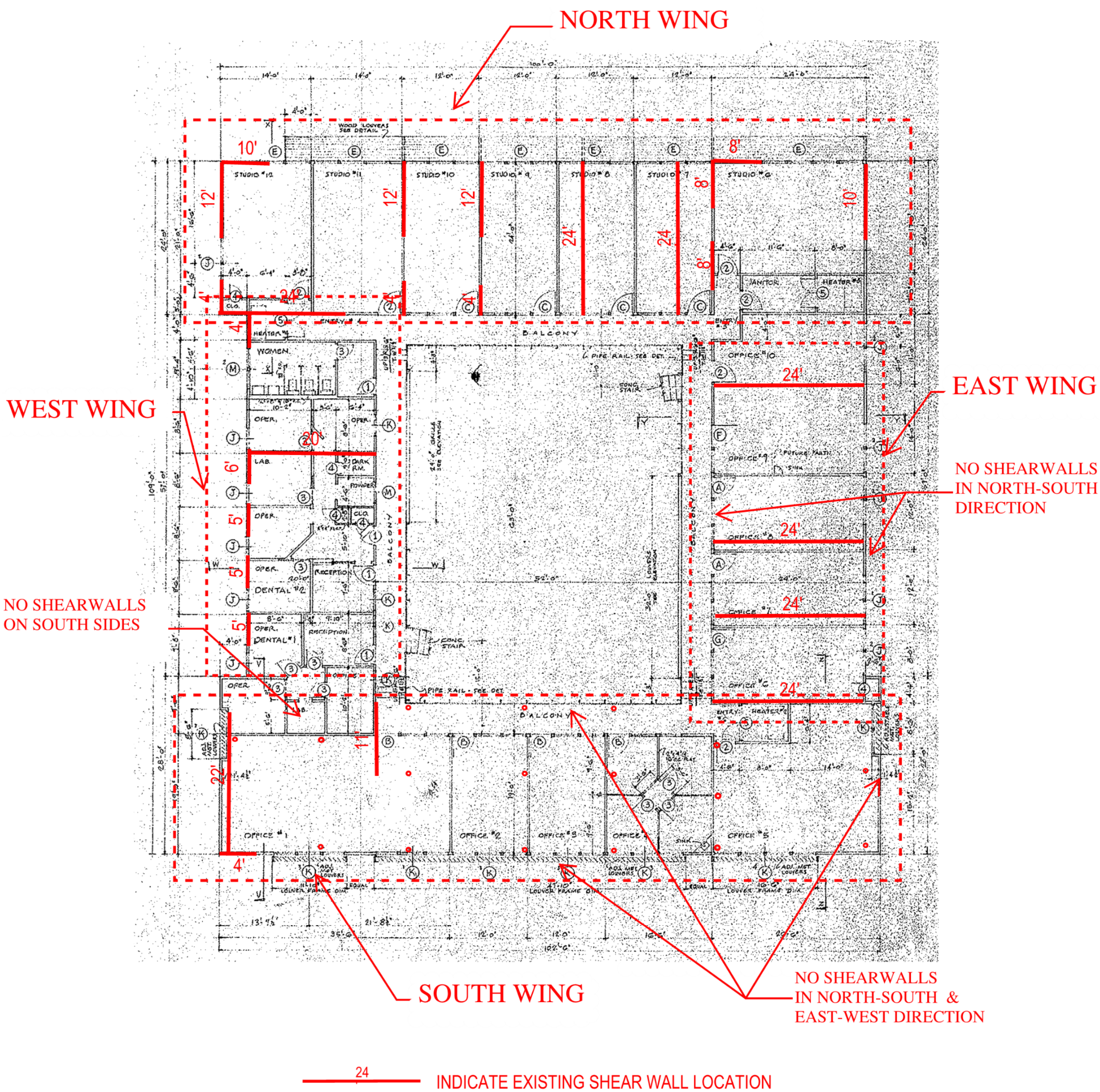


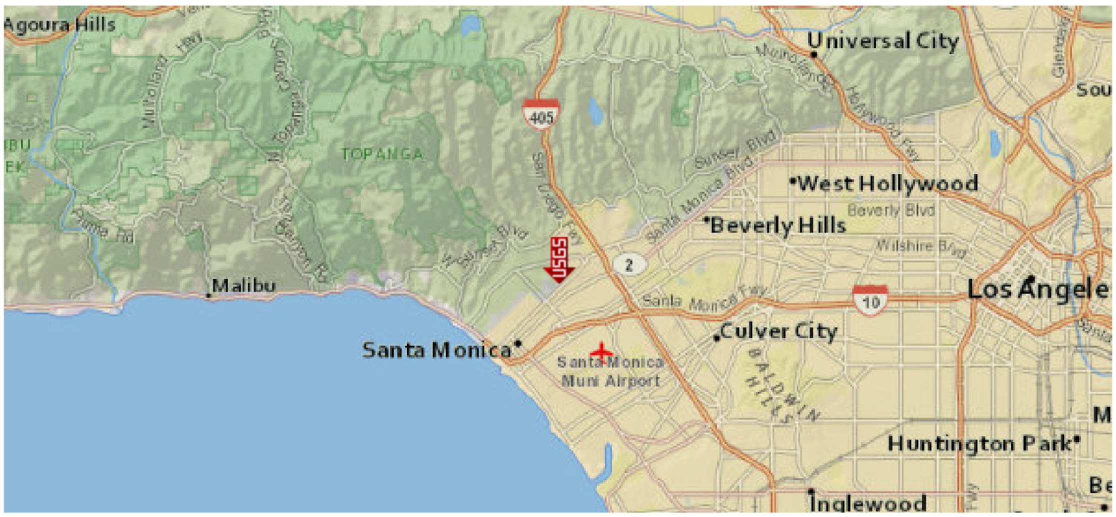
Figure 3.1: Existing Shear Wall Locations

May 26, 2021
Job No. 12-L038B

USGS Design Maps Summary Report

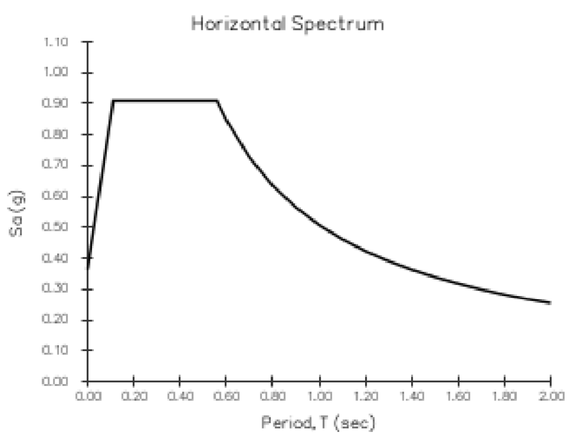
User–Specified Input

Report Title	11973 San Vicente Blvd Wed May 31, 2017 18:40:24 UTC
Building Code Reference Document	ASCE 41-13 Retrofit Standard, BSE-1E (which utilizes USGS hazard data available in 2008)
Site Coordinates	34.05251°N, 118.47185°W
Site Soil Classification	Site Class D – "Stiff Soil"



USGS–Provided Output

$S_{S,20/50}$	0.760 g	$S_{XS,BSE-1E}$	0.909 g
$S_{1,20/50}$	0.274 g	$S_{X1,BSE-1E}$	0.508 g



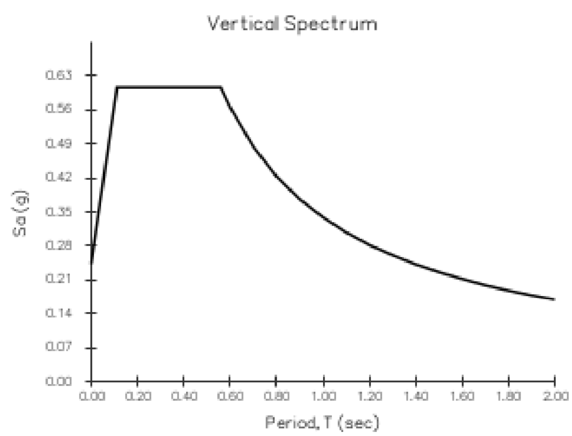


Figure 4.1: Spectral Ordinates per ASCE 41-13

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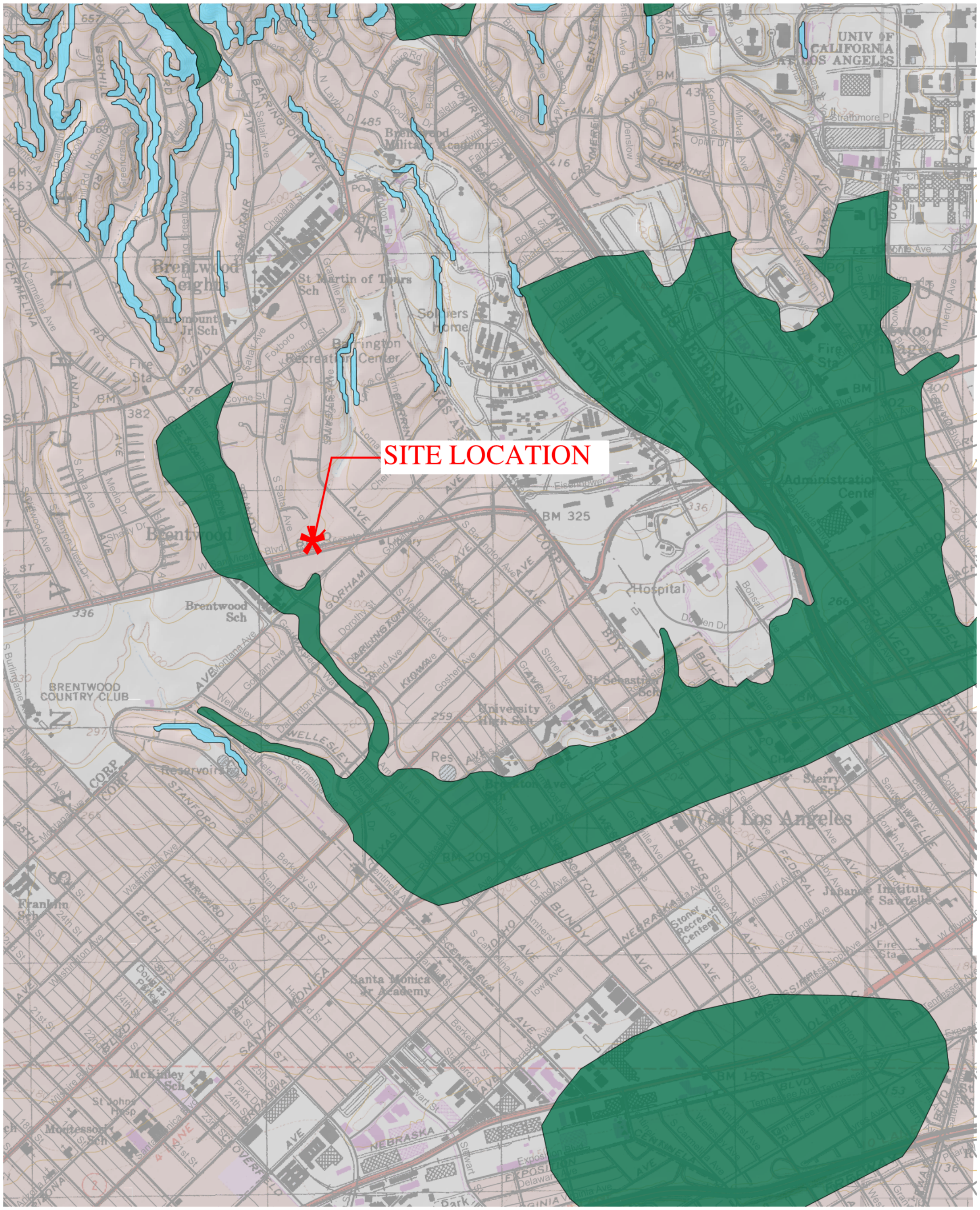

Figure 4.2: State of California Regulatory Map for Seismic Hazards
(Beverly Hills Quadrangle)

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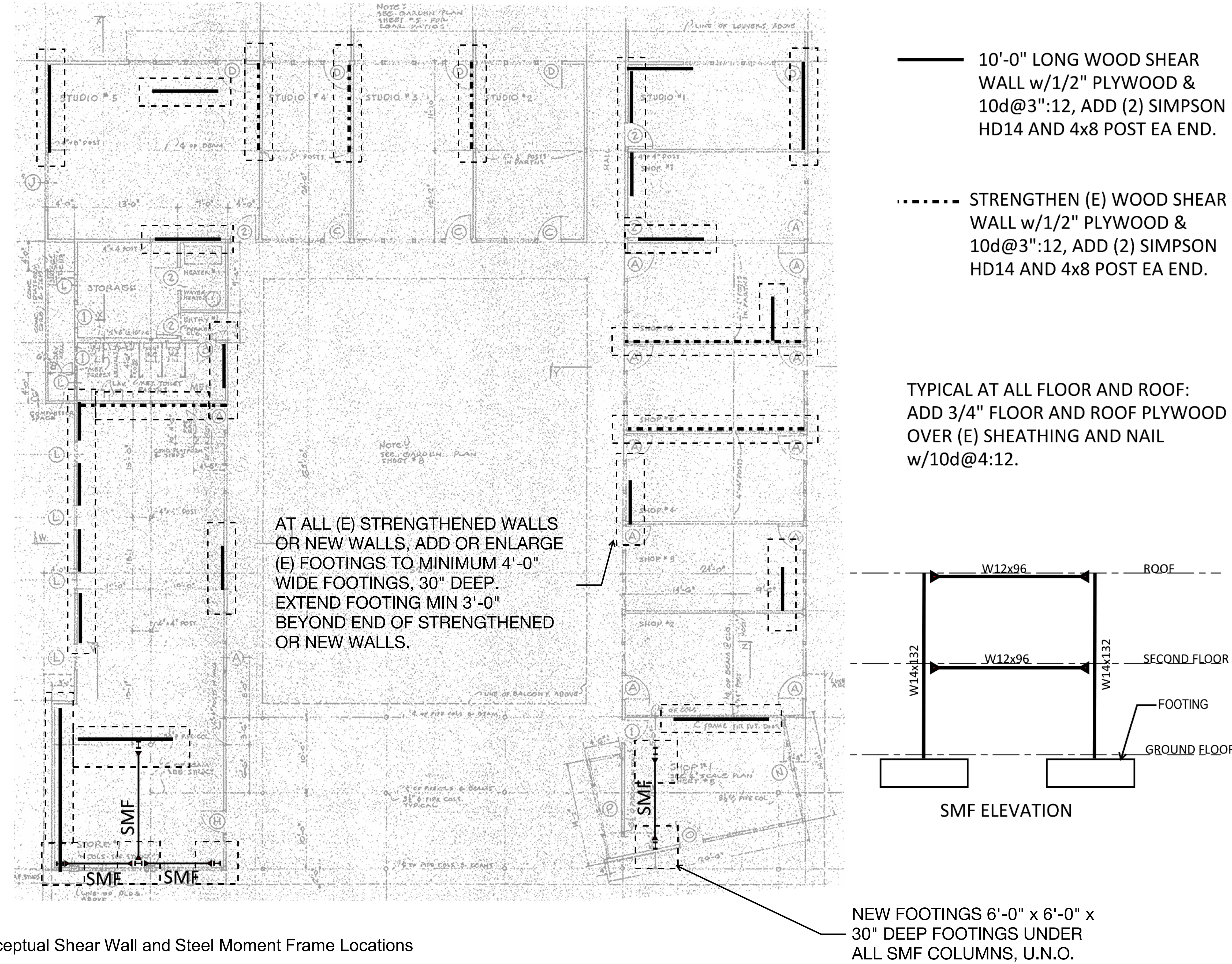

Figure 7.1: Conceptual Shear Wall and Steel Moment Frame Locations

APPENDIX A
Tier 1 Checklists

Chapter 16.0 Tier 1 Checklist

STRUCTURAL COMPONENTS	
C NC U NA	LOAD PATH. The structure shall contain a complete, well-defined load path, including structural elements and connections that serves to transfer the inertial forces associated with the mass of all elements of the building to the foundation. (Commentary: Sec. A.2.1.1. Tier 2: Sec. 5.4.1.1)
C NC U NA	WALL ANCHORAGE. Exterior concrete or masonry walls that are dependent on the diaphragm for lateral support are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed into the diaphragm. Connections shall have adequate strength to resist the connection force calculated in the Quick Check procedure of Section 4.5.3.7. (Commentary: Sec. A.5.1.1. Tier 2: Sec. 5.7.1)

16.1.2LS Life Safety Basic Configuration Checklist

Low Seismicity

Building System

GENERAL	
C NC U NA	LOAD PATH. The structure shall contain a complete, well defined load path, including structural elements and connections that serves to transfer the inertial forces associated with the mass of all elements of the building to the foundation. (Commentary: Sec. A.2.1.1. Tier 2: Sec. 5.4.1.1)
C NC U NA	ADJACENT BUILDING. The clear distance between the building being evaluated and any adjacent building is greater than 4% of the height of the shorter building. This statement shall not apply for the following building types: W1, W1a, and W2. (Commentary: Sec. A.2.1.2. Tier 2: Sec. 5.4.1.2)
C NC U NA	MEZZANINES. Interior mezzanine levels are braced independently from the main structure or are anchored to the seismic-force-resisting elements of the main structure. (Commentary: Sec. A.2.1.3. Tier 2: Sec. 5.4.1.3)
BUILDING CONFIGURATION	
C NC U NA	WEAK STORY. The sum of the shear strengths of the seismic-force resisting system in any story in each direction is not less than 80% of the strength in the adjacent story above. (Commentary: Sec. A.2.2.2. Tier 2: Sec. 5.4.2.1)
C NC U NA	SOFT STORY. The stiffness of the seismic-force-resisting system in any story is not less than 70% of the seismic-force-resisting system stiffness in an adjacent story above or less than 80% of the average seismic-force-resisting system stiffness of the three stories above. (Commentary: Sec. A.2.2.3. Tier 2: Sec. 5.4.2.2)
C NC U NA	VERTICAL IRREGULARITIES. All vertical elements in the seismic-force-resisting system are continuous to the foundation. (Commentary: Sec. A.2.2.4. Tier 2: Sec. 5.4.2.3)
C NC U NA	GEOMETRY. There are no changes in the horizontal dimension of the seismic-force-resisting system of more than 30% in a story relative to adjacent stories, excluding one-story penthouses and mezzanines. (Commentary: Sec. A.2.2.5. Tier 2: Sec. 5.4.2.4)
C NC U NA	MASS. There is no change in effective mass more than 50% from one story to the next. Light roofs, penthouses, and mezzanines need not be considered. (Commentary: Sec. A.2.2.6. Tier 2: Sec. 5.4.2.5)
C NC U NA	TORSION. The estimated distance between the story center of mass and the story center of rigidity is less than 20% of the building width in either plan dimension. (Commentary: Sec. A.2.2.7. Tier 2: Sec. 5.4.2.6)

Moderate Seismicity (Complete the following items in addition to the items for Low Seismicity)

GEOLOGIC SITE HAZARDS	
C NC U NA	LIQUEFACTION. Liquefaction-susceptible, saturated, loose granular soils granular soils that could jeopardize the building's seismic performance shall not exist in the foundation soils at depths within 50 ft. under the building. (Commentary: Sec. A.6.1.1. Tier 2: Sec. 5.4.3.1)
C NC U NA	SLOPE FAILURE. The building site sufficiently remote from potential earthquake-induced slope failures or rockfalls to be unaffected by such failures or is capable of accommodating any predicted movements without failure. (Commentary: Sec. A.6.1.2. Tier 2: Sec. 5.4.3.1)

C NC U NA	SURFACE FAULT RUPTURE. Surface fault rupture and surface displacement at the building site are not anticipated. (Commentary: Sec. A.6.1.3. Tier 2: Sec. 5.4.3.1)

High Seismicity (Complete the following items in addition to the items for Low and Moderate Seismicity)

FOUNDATION CONFIGURATION	
C NC U NA	OVERTURNING. The ratio of the least horizontal dimension of the seismic-force-resisting system at the foundation level to the building height (base/height) is greater than $0.6S_a$. (Commentary: Sec. A.6.2.1. Tier 2: Sec. 5.4.3.3)
C NC U NA	THIS BETWEEN FOUNDATION ELEMENTS. The foundation has ties adequate to resist seismic forces where footings, piles, and piers are not restrained by beams, slabs, or soils classified as Site Class A, B, or C. (Commentary: Sec. A.6.2.2. Tier 2: Sec. 5.4.3.4)

16.3LS Life Safety Structural Checklist for Building Type W2: Wood Frames, Commercial and Industrial

Low and Moderate Seismicity

LATERAL-SEISMIC-FORCE-RESISTING SYSTEM	
C NC U NA	REDUNDANCY. The number of lines of shear walls in each principal direction is greater than or equal to 2. (Commentary: Sec. A.3.2.1.1. Tier 2: Sec. 5.5.1.1, and)
C NC U NA	SHEAR STRESS CHECK: The shear stress in the shear walls, calculated using the Quick Check procedure of Section 4.5.3.3, is less than the following values (Commentary: Sec. A.3.2.7.1. Tier 2: Sec. 5.5.3.1.1): Structural panel sheathing 1,000 lb/ft Diagonal sheathing 700 lb/ft Straight sheathing 100 lb/ft All other conditions 100 lb/ft
C NC U NA	STUCCO (EXTERIOR PLASTER) SHEAR WALLS. Multi-story buildings do not rely on exterior stucco walls as the primary seismic-force-resisting system. (Commentary: Sec. A.3.2.7.2. Tier 2: Sec. 5.5.3.6.1)
C NC U NA	GYPSUM WALLBOARD OR PLASTER SHEAR WALLS. Interior plaster or gypsum wallboard is not used as shear walls on buildings over one story in height with the exception of the uppermost level of a multistory building. (Commentary: Sec. A.3.2.7.3. Tier 2: Sec. 5.5.3.6.1)
C NC U NA	NARROW WOOD SHEAR WALLS. Narrow wood shear walls with an aspect ratio greater than 2-to-1 are not used to resist seismic forces. (Commentary: Sec. A.3.2.7.4. Tier 2: Sec. 5.5.3.6.1)
C NC U NA	WALLS CONNECTED THROUGH FLOORS. Shear walls have an interconnection between stories to transfer overturning and shear forces through the floor. (Commentary: Sec. A.3.2.7.5. Tier 2: Sec.5.5.3.6.2)
C NC U NA	HILLSIDE SITE. For structures that are taller on at least one side by more than one-half story due to a sloping site, all shear walls on the downhill slope have an aspect ratio less than 1-to-1. (Commentary: Sec. A.3.2.7.6. Tier 2: Sec. 5.5.3.6.3)
C NC U NA	CRIPPLE WALLS. Cripple walls below first-floor-level shear walls are braced to the foundation with wood structural panels. (Commentary: Sec. A.3.2.7.7. Tier 2: Sec. 5.5.3.6.4)
C NC U NA	OPENINGS: Walls with openings greater than 80% of the length are braced with wood structural panel shear walls with aspect ratios of not more than 1.5-to-1 or are supported by adjacent construction through positive ties capable of transferring the seismic forces. (Commentary: Sec. A.3.2.7.8. Tier 2: Sec. 5.5.3.6.5)
CONNECTIONS	
C NC U NA	WOOD POSTS. There is a positive connection of wood posts to the foundation. (Commentary: Sec. A.5.3.3. Tier 2: Sec. 5.7.3.3)
C NC U NA	WOOD SILLS. All wood sills are bolted to the foundation. (Commentary: Sec. A.5.3.4. Tier 2: Sec. 5.7.3.3)
C NC U NA	GIRDER/COLUMN CONNECTION. There is a positive connection using plates, connection hardware, or straps between the girder and the column support. (Commentary: Sec. A.5.4.1. Tier 2: Sec. 5.7.4.1)

High Seismicity (Complete the following items in addition to the items for Low and Moderate Seismicity)

DIAPHRAGMS	
C NC U NA	DIAPHRAGM CONTINUITY. The diaphragms are not composed of split-level floors and do not have expansion joints. (Commentary: Sec. A.4.1.1. Tier 2: Sec. 5.6.1.1)
C NC U NA	ROOF CHORD CONTINUITY. All chord elements are continuous, regardless of changes in roof elevation. (Commentary: Sec. A.4.1.3. Tier 2: Sec. 5.6.1.1)
C NC U NA	DIAPHRAGM REINFORCEMENT AT OPENINGS. There is reinforcing around all diaphragm openings larger than 50% of the building width in either major plan dimension. (Commentary: Sec. A.4.1.8. Tier 2: Sec. 5.6.1.5)
C NC U NA	STRAIGHT SHEATHING. All straight sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered. (Commentary: Sec. A.4.2.1. Tier 2: Sec. 5.6.2)
C NC U NA	SPANS. All wood diaphragms with spans greater than 24 ft. consist of wood structural panels or diagonal sheathing. Wood commercial and industrial buildings may have rod-braced systems. (Commentary: Sec. A.4.2.2. Tier 2: Sec. 5.6.2)
C NC U NA	DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS. All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 feet and aspect ratios less than or equal to 4-to-1. (Commentary: Sec. A.4.2.3. Tier 2: Sec. 5.6.2)
C NC U NA	OTHER DIAPHRAGMS. The diaphragm does not consist of a system other than wood, metal deck, concrete, or horizontal bracing. (Commentary: Sec. A.4.7.1. Tier 2: Sec. 5.6.5)
CONNECTIONS	
C NC U NA	WOOD SILL BOLTS. Sill bolts are spaced at 6 feet or less, with proper edge and end distance provided for wood and concrete. (Commentary: A.5.3.7. Tier 2: Sec. 5.7.3.3)