STRUCTURAL ENGINEERS

ASCE 41-13 Tier 1 and 2 Seismic Evaluation Report

Prepared for: Ross Drulis Cusenbery Architecture, Inc. **July 19, 2016**

Kensington Public Safety Building
215 Arlington Avenue
Kensington, California 94707

IDA Project Number 1600

1 Introduction

IDA Structural Engineers (IDA) has performed a seismic evaluation of the Kensington Public Safety Building, located at 215 Arlington Avenue, California, using an ASCE-41-13, Tier 2 seismic evaluation procedure. ASCE 41-13, titled "*Seismic Evaluation and Retrofit of Existing Buildings*," published by the American Society of Civil Engineers (ASCE) in 2013, is the industry standard procedure for the seismic evaluation and retrofit of existing buildings.

The primary intent of the Tier 1 screening and Tier 2 deficiency based procedure is to evaluate and where warranted, reduce seismic risk efficiently where possible and appropriate by using simplified procedures targeted to a specific building type.

The information below forms the foundation for the evaluation. This information is either derived from owner requirements, such as risk category and desired structural performance level, or is site specific, such as seismic hazard level.

Building	Kensington Public Safety Building
Address	215 Arlington Ave, Kensington, CA
Latitude and Longitude	37.906234, -122.278724
Risk Category	IV, buildings and other structures designated as essential facilities
Basic Performance Objective for Existing Buildings (BPOE)	1-B Immediate Occupancy Structural Performance (S-1) Position Retention Nonstructural Performance (N-B)
Seismic Hazard Level	BSE-1E 20% in 50 years, 225 year return period
Level of Seismicity	High
Soil Type	NEHRP C
Site Class	C
Building Type	Wood framed building, sheathed with wood structural shear panels.

1.1 Performance Objective

The performance objective consists of one or more pairings of a selected Seismic Hazard Level with a target Structural Performance Level and Nonstructural Performance Level.

The Basic Performance Objective for Existing Buildings (BPOE) is a specific, seismic Performance Objective (from several available choices) and is dependent on the Risk Category of the building and the desired seismic performance expected by the owner. The BPOE for existing buildings is a slightly lower category which may result in a lower level of safety and a higher probability of collapse than what may be provided by building codes for new buildings. Buildings meeting the BPOE are expected to incur very little damage from relatively frequent, small to moderate earthquakes but are expected to incur greater levels of damage and economic loss from severe earthquakes. The level of damage and potential economic loss for buildings rehabilitated to the BPOE likely will be greater than expected for the Basic Performance Objective for New Buildings (BPON).

Accepting a seismic performance objective (BPOE) which could be less than "new code" (BPON) allows that relatively new existing buildings are not evaluated as deficient when updated and more conservative codes are adopted over time.

The increase in seismic risk is tempered by the recognition that existing buildings often have a shorter remaining useful lifespan than new buildings. That is, if the traditional code based demand for new buildings presumes a 50 year life, then an existing building with a 30 year remaining lifespan has a lower probability of experiencing a code level (or major) earthquake over its remaining lifespan. The standard also recognizes that the cost of achieving smaller probability of damage caused by the higher level of performance is often disproportionate to the incremental cost.

The Performance Level is 1-B in the BPOE, which provides most of the protection obtained under the Operational Building Performance Level without the added cost of providing standby utilities and performing rigorous seismic qualification of building equipment performance.

1.1.1 Structural Performance Level for BPOE

The structural performance level for BPOE is S-1, which provides for Immediate Occupancy performance of the building following an earthquake meeting the criteria discussed under the seismic hazards section.

A structure conforming to the Immediate Occupancy seismic performance level should be expected to have a very limited damage state following the anticipated seismic event. The basic lateral and vertical force resisting systems of the building should retain almost all of their pre-earthquake strength and stiffness. The risk of life-threatening injury (life safety) as a result of structural damage is very low. Although minor structural repairs might be anticipated, repairs would generally not be required before re-occupancy.

1.1.2 Nonstructural Performance Level for BPOE

The nonstructural performance level is N-B, Position Retention (for BPOE).

Continued use of the building post-earthquake is not only limited by its structural condition but might be limited by damage to or disruption to nonstructural elements of the building, furnishings or equipment or the availability of external utility services. Nonstructural performance level N-B, "Position Retention," is the post-earthquake damage state in which nonstructural components could be damaged, and may not function, but are anchored in place so that they do not fall, topple, or break connections. By avoiding potential component falling or toppling, or breaking of utility connections (such as, water, gasses, or electricity) life safety is provided to building occupants. Building access and life safety systems include doors, hallways, stairways, elevators, emergency lighting, fire alarms and fire suppression systems, are generally expected remain available and operable provided that power and utility services are available at the building. Occupants should be able to occupy the building safely. Potentially, some use may be impaired, and some clean up may be needed. The N-B, Position Retention nonstructural Performance Level essentially mirrors the requirements of new building design for cases where the structure is designed for life safety and not immediate occupancy.

1.1.3 Seismic Hazard Level for BPOE

The Basic Safety Earthquake for BPOE is 1E, which requires ground motions with a 20% probability of exceedance in 50 years (or a 225 year recurrence interval). For reference ASCE 7-10 uses a design procedure based on 2/3 values of the MCEr earthquake at any site for new design (generally based on 2% probability of exceedance in 50 year period earthquake, with a 2500 year recurrence interval, however, in high seismic near fault regions the probabilistic earthquake is modified to a deterministic calculation by USGS which reduces the ground motions from absolute probabilities). The ASCE 7 procedures along with the seismic ground motions strive to achieve a 10% probability of collapse for MCEr for properly designed buildings.

The commentary in ASCE 41-13 notes that for Risk Category III and IV buildings, the BPOE (basic performance objective for existing buildings) using the BSE-1E earthquake (20% in 50 years, 975 year recurrence) has not traditionally been used and instead, Risk Category III and IV buildings have been evaluated to levels consistent with new building design, using 2/3 of MCEr per ASCE 7-10 procedures. This would produce seismic demands greater than what the BSE-1E earthquake demands would be. Given these facts, it is most likely not financially feasible to pursue a new building equivalent seismic hazard level for this building.

2 Site Description

The Kensington Public Safety Building is located along Arlington Avenue constructed amongst single family residential buildings. The building is constructed on a slope into the uphill side of the hill. The first floor is built into the slope with a retaining wall at the rear of the building. The second floor exits to a parking lot behind the building. There is an additional concrete retaining wall at the rear of the parking lot which supports residential lots above. A sloped driveway along the south side of the building connects the Arlington Avenue to the parking lot in the rear. The building is south of Oberlin Avenue and East of Amherst Avenue.

3 Building Description

The building, constructed in the early 1960's is a two story wood framed structure supported on continuous concrete foundations. The seismic load resisting system appears to be light framed walls sheathed with plywood structural sheathing. The ground floor is constructed into the hillside with a retaining wall at the rear of the building which is approximately the height of the first floor. The top of concrete foundation on the sides slopes from the top of the wall to the bottom of the first floor. The first floor appears to be constructed as a concrete slab-on-grade. The total building area is approximately 5700 square feet. The overall building dimensions are approximately 40 feet by 80 feet with a maximum height of about 45 feet. See Figures 1 to 5 for photos of the

In 1998 a renovation was performed on the building which included a partial seismic retrofit. In this renovation, plywood shear walls were strengthened in the middle of the building at a wall between the apparatus bay and the offices. The front of the building was strengthened with steel moment frames at the entry of the apparatus bay. Drilled piers were also added at the exterior of the building in an attempt to resist sliding of the building downhill.

In 2004 another renovation was performed. In this renovation, some minor framing changes were made at the second floor over the apparatus bay. The shear wall between the apparatus bay and the offices was strengthened again. The beam/column connections at the apparatus bay moment frame were strengthened during this renovation.

4 Geotechnical Information

For this evaluation, two previous geotechnical evaluations were provided. A 1990 geotechnical evaluation by Seidelman Associates, Inc. was performed to evaluate potential fault traces on site. A 1997 geotechnical evaluation by Geomatrix Consultants evaluated potential earthquake-related earthquake hazards such as surface fault rupture and landslide/ slope stabilities. However, these reports do not provide current seismic ground motion data values. Therefore the seismic ground motions used in this evaluation were derived from United States Geological Survey and California Geological Survey maps and fault information. See Appendix C for information used.

The geotechnical reports do not indicate that liquefaction is a consideration at this site.

5 Site Observation Notes:

A site visit to observe the existing building was performed on July 13, 2016. The building generally appeared to be in good shape. There were no visible observed signs of rot or decay. There were areas of the slab in the garage concrete slab exhibiting signs of slab settlement in the form of cracks. Settling of exterior paving at the rear parking lot and minor cracking at the exterior footings along the driveway side of the building appear to be indicators of settlement on site. It is unclear whether the movement occurred before or after the retrofit measures performed as part of the 1998 renovation.

6 Available Documents

The following drawings were available for review for this evaluation:

- Original architectural, dated March 27, 1969 by Jeffries, Lyons, and Hill Architects.
- Renovation drawings dated September 10, 1998, by Marcy Li Wong Architects.
- Renovation drawings dated September 10, 1998, by The Crosby Group.
- Renovation drawings dated June 21, 2004, by Baseline Engineering.
- Renovation drawings dated June 29, 2004, by Italo A. Calpestri III & Associates, AIA.

7 Tier 1 Deficiencies

The checklists and calculations for Tier 1 evaluation are located in Appendix B.

7.1 Vertical irregularities

At the front of the building long Line E, assumed shear walls between 4 and 7 and the second floor do not align vertically with the moment frame at the apparatus bay.

7.2 Slope Failure

The 1997 Geotechnical Evaluation by Geomatrix determined that there was risk of slope failure due to a seismic event. The renovation drawings by Crosby Group from September 1998 appear to have partially addressed this risk by the installation of concrete piers in the driveway outside of the apparatus bay between grid lines 1 to 4. It does not appear any mitigation measures were installed between lines 4 to 7 to resist the movement of the building downslope. Signs such as minor foundation cracking and slab cracks indicate that some foundation movement has occurred. However, it is unclear if this movement is due to normal foundation settlement or indications of slope failure.

7.3 Shear Stress Check

There is insufficient information on the drawings to determine the extent of plywood shear wall nailing in areas of the building not documented in the 1998 and 2004 renovations. For this analysis we have assumed the presence of nominally nailed plywood around the exterior of the building. This analysis combined the assumed strength of these walls with the addition of the new shear walls documented in the renovation drawings. The shear stresses in the walls exceed the allowable in the Tier 1 checks in several locations.

7.4 Diaphragm Continuity

The diaphragm at the second floor has a split level and does not meet the Tier 1 check for diaphragm continuity.

7.5 Steel Moment Frames with Flexible Diaphragms: Steel Column Connections

This Tier 1 check evaluates the ability of the column anchor connection to resist the foundation.

8 Tier 2 Analysis

The checklists and calculations for Tier 2 evaluation are located in Appendix B.

8.1 Vertical irregularities

A Tier 2 evaluation finds that the collector connections are adequate. The shear wall capacities and moment frame capacities at this line are evaluated further under shear stress checks and moment frame checks. The shear stress check found walls which were not compliant. The moment frame checks for beam and column flexural stresses were compliant. 2^{nd} floor shear walls between grid lines 4 and 7 do not appear to have posts or holdowns to transfer overturning forces to the first floor.

8.2 Shear Stress Check

Tier 2 evaluation of the shear walls finds that the shear stress checks are not compliant. The shear stresses in some walls exceed the assumed capacity of the walls. The wall lines which require strengthening are identified in the mitigation plan.

8.3 Diaphragm Continuity

The diaphragm at the second floor has a split level and therefore does not meet the Tier 1 check. Based on evaluation of the diaphragm load path it appears that the diaphragm is insufficient is insufficient to transfer seismic loads across the discontinuity.

8.4 Steel Moment Frames with Flexible Diaphragms: Steel Column Connections

The existing column connection was evaluated for the seismic demand of the moment frames and is non-compliant for the Tier 2 check.

9 Mitigation

See Appendix A for schematic mitigation plan which identifies the locations of the mitigation measures. Below is a description of the different mitigation items.

9.1 Vertical Irregularity

Provide posts and holdowns at the first floor to transfer overturning forces from the 2nd floor shear walls to the first floor.

9.2 Slope Failure

Obvious signs of slope failure and movement downhill of the building were not observed during the site visit. A monitoring program is recommended to track potential movement of the building over time. Because drilled piers were installed between grid lines 1 and 4, particular attention should be paid to the section between grid lines 4 and 7. If a monitoring program identifies that building is moving downslope, it is recommended to add drilled piers parallel to line E between lines 4 and 7 to mitigate further movement of the building.

9.3 Shear Stress in Wood Shear Walls

Add plywood shear walls and holdowns or increase nailing at existing shear walls and replace holdowns as required.

9.4 Diaphragm Continuity

Increase nailing at floor diaphragm and at split level transition to transfer loads across the diaphragm split level.

9.5 Steel Moment Frames with Flexible Diaphragms: Steel Column Connections

Install additional anchor bolts to strengthen connection of moment frame columns to foundations.

10 Conclusions

The building appears to be in good overall condition. Based on the ASCE 41 evaluation, there are a number of items which should be addressed. It should be noted that these findings are based on limited information on existing drawings and assumptions on existing conditions such as shear wall nailing. Information from investigation of existing conditions through local demolition may result in determining that the elements are compliant.

However, given the vintage of construction, it is likely that these elements require the mitigation recommendations noted in this report to meet the Immediate Occupancy goals for an essential service facility such as the Kensington Public Safety Building.

Please do not hesitate to call with any questions regarding this analysis.

IDA Structural Engineers, Inc.

Jason M. Lee, SE
Associate

Figure 1: Aerial View

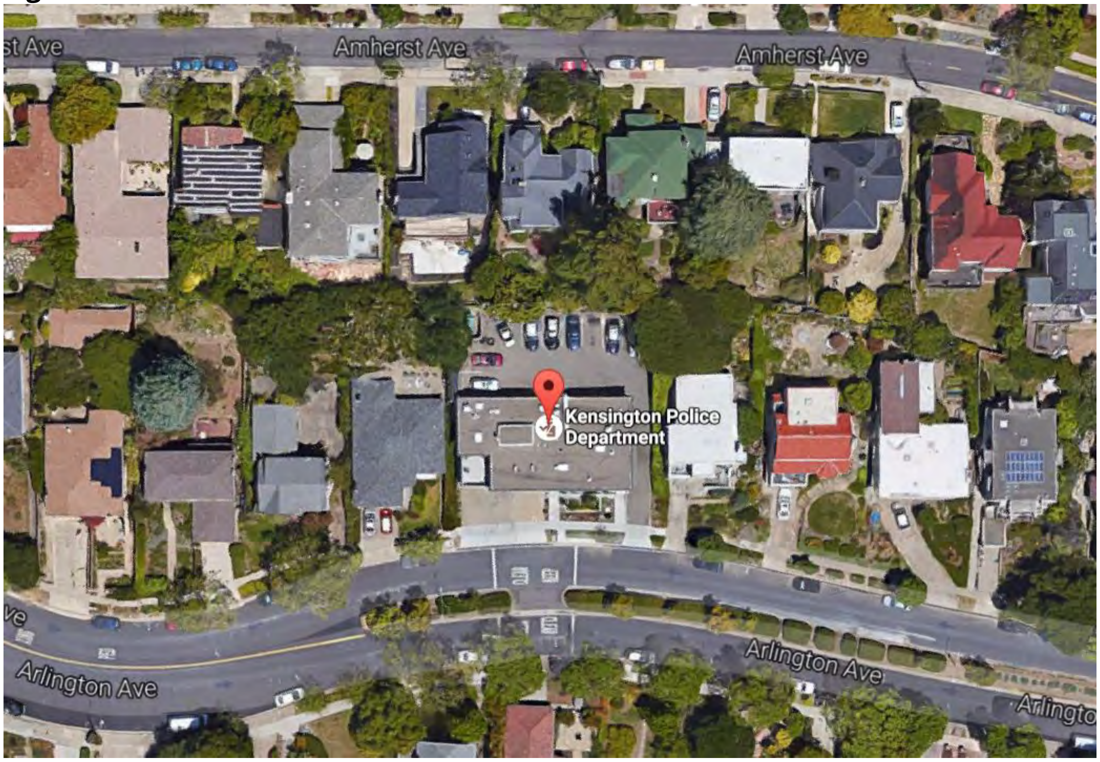


Figure 2: View from the North

Figure 3: View from the South

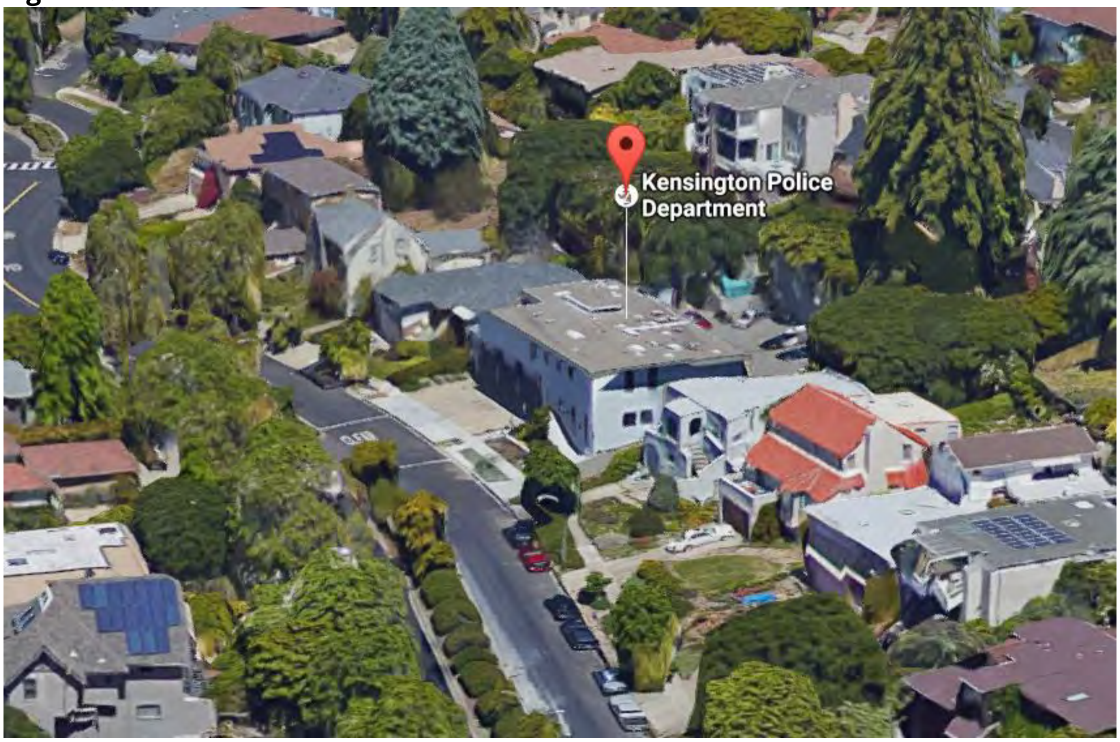
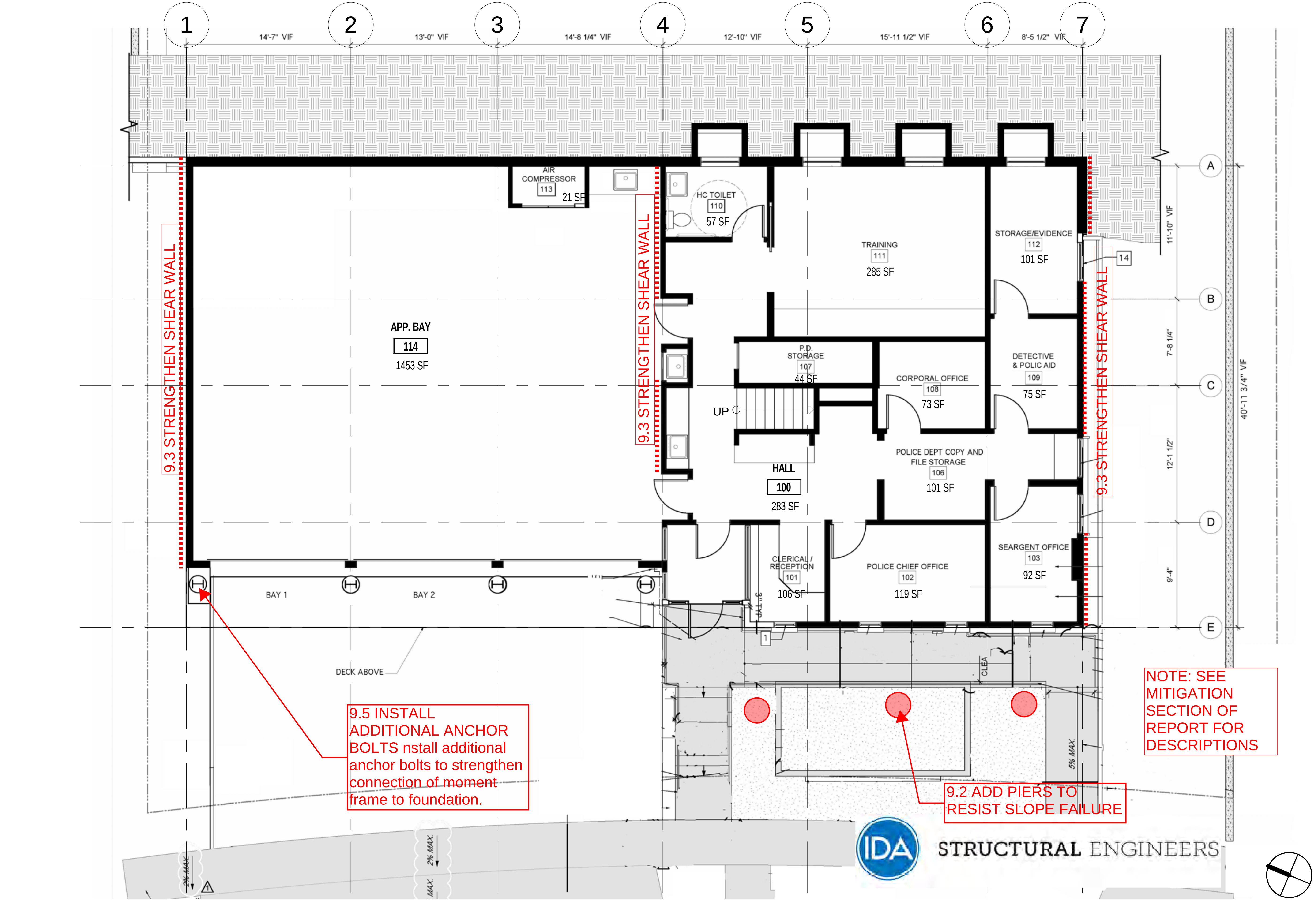


Figure 4: View from the West

Figure 5: View from the East

APPENDIX A

MITIGATION PLANS



1 FIRST FLOOR PLAN
1/8" = 1'-0"

Preliminary design. Not for bidding or construction purposes.

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KENSINGTON FIRE STATION

KENSINGTON, CA

Sheet Title

EXISTING FLOOR PLAN - LEVEL 1

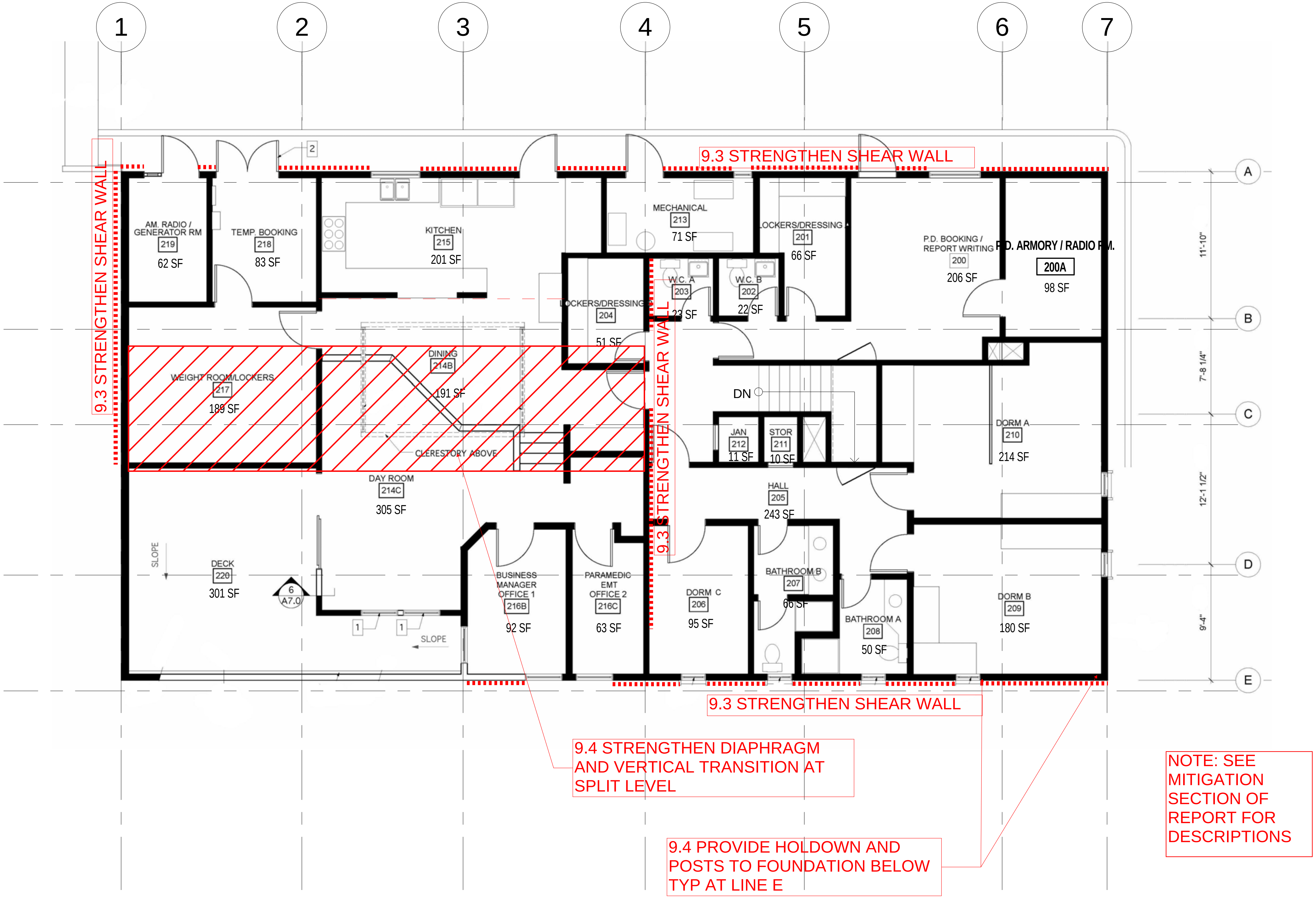
STRUCTURAL MITIGATION

Scale: 1/8" = 1'-0"

Date: 2016/07/07

S1

Drawing No.



1 EXISTING SECOND FLOOR PLAN
1/8" = 1'-0"

IDA STRUCTURAL ENGINEERS

Preliminary design. Not for bidding or construction purposes.

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KENSINGTON, CA

Sheet Title
EXISTING FLOOR PLAN - LEVEL 2
STRUCTURAL MITIGATION

Scale: 1/8" = 1'-0"
Date: 2016/07/07
S2
Drawing No.

APPENDIX B

CHECKLISTS AND CALCULATIONS

APPENDIX C
SUMMARY DATA SHEET

BUILDING DATA

Building Name: Kensington Public Safety Building
Date:
Building Address: Kensington Public Safety Building
Latitude: 37.906233
Longitude: -122.278758
By: JML
Year Built: 1969
Year(s) Remodeled: 1998, 2004
Original Design Code:
Area (sf): 5800
Length (ft): 79' 2"
Width (ft): 40' 8"
No. of Stories: 2
Story Height: +/- 11' 3"
Total Height: 22' 6"

USE ☐ Industrial ☐ Office ☐ Warehouse ☐ Hospital ☐ Residential ☐ Educational ☒ Other: Essential services facility

CONSTRUCTION DATA

Gravity Load Structural System: Light framed wood bearing walls
Exterior Transverse Walls:
Openings?
Exterior Longitudinal Walls:
Openings?
Roof Materials/Framing: Built up Roofing over 1/2" PW spanning between 2x8 joists @ 16"oc
Intermediate Floors/Framing: 3/4" PW over either 2x10 or 2x14 floor joists @ 16" oc
Ground Floor: Reinforced concrete slab, 7" thick in apparatus bay, 4" thick in remaining areas
Columns: Wood and steel columns
Foundation: Continuous reinforced concrete footing, six concrete drilled pier
General Condition of Structure: Well maintained
Levels Below Grade? Ground floor is partially embedded in slope
Special Features and Comments: Building is built into a slope. Parking at rear is elevation of upper floor.

LATERAL-FORCE-RESISTING SYSTEM

	Longitudinal	Transverse
System:	Dual system, Wood shear walls and moment frame	
Vertical Elements:	Wood shear walls and moment frame	
Diaphragms:	Plywood/Flexible	
Connections:		

EVALUATION DATA

BSE-1N Spectral Response Accelerations: S_{Ds} = 1.655 S_{D1} = 1.031
Soil Factors: Class = D F_a = 1.0 F_v = 1.5
BSE-1E Spectral Response Accelerations: S_{XS} = 1.231 S_{X1} = 0.69
Level of Seismicity: High
Performance Level: Immediate Occupancy
Building Period: T = 0.207 s
Spectral Acceleration: S_a = 1.231
Modification Factor: $C_mC_1C_2$ = 1.1
Building Weight: W = 211 k
Pseudo Lateral Force: V= $C_mC_1C_2S_aW$= 286 kip

BUILDING CLASSIFICATION:

REQUIRED TIER 1 CHECKLISTS	**Yes**	**No**
Basic Configuration Checklist	☒	☐
Building Type W2 **Structural Checklist**	☒	☐
Nonstructural Component Checklist	☐	☒

FURTHER EVALUATION REQUIREMENT: Life safety and Immediate Occupancy, Tier 2 as required for Tier 1 non-complicance. Checklist S1 IO also evaluated for moment frame installed in 1998.

Project: Kensington Public Safety Building Location: Kensington, CA

Completed by: JML Date: ______

16.1.2LS LIFE SAFETY BASIC CONFIGURATION CHECKLIST

Low Seismicity

Building System

General

C NC N/A U 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 N/A U ADJACENT BUILDINGS: 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 N/A U 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 N/A U 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. A2.2.2. Tier 2: Sec. 5.4.2.1)

C NC N/A U 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 N/A U 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 N/A U GEOMETRY: There are no changes in the net 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 N/A U 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 N/A U 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 N/A U LIQUEFACTION: Liquefaction-susceptible, saturated, loose 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: 5.4.3.1) Underlain by dense,relatively hard shale per project geotechnical investigation

C NC N/A U SLOPE FAILURE: The building site is 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: 5.4.3.1)

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

High Seismicity: Complete the Following Items in Addition to the Items for Low and Moderate Seismicity.

Foundation Configuration

C NC N/A U 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 N/A U TIES 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)

Project: Kensington Public Safety Building　Location: Kensington, CA

Completed by: JML　Date: ______

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 N/A U 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)

C NC N/A U 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 N/A U 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 N/A U GYPSUM WALLBOARD OR PLASTER SHEAR WALLS: Interior plaster or gypsum wallboard is not used as shear walls on buildings more than one story high with the exception of the uppermost level of a multi-story building. (Commentary: Sec. A.3.2.7.3. Tier 2: Sec. 5.5.3.6.1)

C NC N/A U 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 N/A U 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 N/A U HILLSIDE SITE: For structures that are taller on at least one side by more than one-half story because of 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 N/A U 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 N/A U 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 N/A U 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 N/A U 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 N/A U 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 N/A U 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 N/A U 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 N/A U 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 N/A U 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 N/A U 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 N/A U DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 ft 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 N/A U 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 N/A U WOOD SILL BOLTS: Sill bolts are spaced at 6 ft 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)

Project: Kensington Public Safety Building   Location: Kensington, CA

Completed by: JML   Date: ______

16.3IO IMMEDIATE OCCUPANCY STRUCTURAL CHECKLIST FOR BUILDING TYPE W2: WOOD FRAMES, COMMERCIAL AND INDUSTRIAL

Very Low Seismicity

Seismic-Force-Resisting System

C NC N/A U 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)

C NC N/A U 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 N/A U 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 N/A U GYPSUM WALLBOARD OR PLASTER SHEAR WALLS: Interior plaster or gypsum wallboard is not used as shear walls on buildings more than one story high with the exception of the uppermost level of a multi-story building. (Commentary: Sec. A.3.2.7.3. Tier 2: Sec. 5.5.3.6.1)

C NC N/A U 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 N/A U 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 N/A U HILLSIDE SITE: For structures that are taller on at least one side by more than one-half story because of a sloping site, all shear walls on the downhill slope have an aspect ratio less than 1-to-2. (Commentary: Sec. A.3.2.7.6. Tier 2: Sec. 5.5.3.6.3)

C NC N/A U 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 N/A U 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)

C NC N/A U HOLD-DOWN ANCHORS: All shear walls have hold-down anchors, constructed per acceptable construction practices, attached to the end studs. (Commentary: Sec. A.3.2.7.9. Tier 2: Sec. 5.5.3.6.6)

Connections

C NC N/A U 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 N/A U 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 N/A U 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)

Foundation System

C NC N/A U DEEP FOUNDATIONS: Piles and piers are capable of transferring the lateral forces between the structure and the soil. (Commentary: Sec. A.6.2.3.)

C NC N/A U SLOPING SITES: The difference in foundation embedment depth from one side of the building to another shall not exceed one story high. (Commentary: Sec. A.6.2.4)

Project: Kensington Public Safety Building Location: Kensington, CA

Completed by: TR Date:

16.4IO IMMEDIATE OCCUPANCY STRUCTURAL CHECKLIST FOR BUILDING TYPES S1: STEEL MOMENT FRAMES WITH STIFF DIAPHRAGMS AND S1A: STEEL MOMENT FRAMES WITH FLEXIBLE DIAPHRAGMS

Very Low Seismicity

Seismic-Force-Resisting System

C NC N/A U DRIFT CHECK: The drift ratio of the steel moment frames, calculated using the Quick Check procedure of Section 4.5.3.1, is less than 0.015. (Commentary: Sec. A.3.1.3.1. Tier 2: Sec. 5.5.2.1.2)

C NC N/A U COLUMN AXIAL STRESS CHECK: The axial stress caused by gravity loads in columns subjected to overturning forces is less than $0.10F_y$. Alternatively, the axial stress caused by overturning forces alone, calculated using the Quick Check procedure of Section 4.5.3.6, is less than $0.30F_y$. (Commentary: Sec. A.3.1.3.2. Tier 2: Sec. 5.5.2.1.3)

C NC N/A U FLEXURAL STRESS CHECK: The average flexural stress in the moment frame columns and beams, calculated using the Quick Check procedure of Section 4.5.3.9, is less than F_y. Columns need not be checked if the strong column—weak beam checklist item is compliant. (Commentary: Sec. A.3.1.3.3. Tier 2: Sec. 5.5.2.1.2)

Connections

C NC N/A U STEEL COLUMNS: The columns in seismic-force-resisting frames are anchored to the building foundation. (Commentary: Sec. A.5.3.1. Tier 2: Sec. 5.7.3.1)

Low Seismicity: Complete the Following Items in Addition to the Items for Very Low Seismicity.

Seismic-Force-Resisting System

C NC N/A U REDUNDANCY: The number of lines of moment frames in each principal direction is greater than or equal to 2. The number of bays of moment frames in each line is greater than or equal to 3. (Commentary: Sec. A.3.1.1.1. Tier 2: Sec. 5.5.1.1) Three bays

C NC N/A U INTERFERING WALLS: All concrete and masonry infill walls placed in moment frames are isolated from structural elements. (Commentary: Sec. A.3.1.2.1. Tier 2: Sec. 5.5.2.1.1)

Connections

C NC N/A U TRANSFER TO STEEL FRAMES: Diaphragms are connected for transfer of seismic forces to the steel frames, and the connections are able to develop the lesser of the strength of the frames or the diaphragms. (Commentary: Sec. A.5.2.2. Tier 2: Sec. 5.7.2)

C NC N/A U STEEL COLUMNS: The columns in seismic-force-resisting frames are anchored to the building foundation, and the anchorage is able to develop the least of the tensile capacity of the column, the tensile capacity of the lowest level column splice (if any), or the uplift capacity of the foundation. (Commentary: Sec. A.5.3.1. Tier 2: Sec. 5.7.3.1)

Moderate Seismicity: Complete the Following Items in Addition to the Items for Very Low and Low Seismicity.

Seismic-Force-Resisting System

C NC N/A U MOMENT-RESISTING CONNECTIONS: All moment connections are able to develop the expected strength of the adjoining members based on the specified minimum yield stress of the steel. (Commentary: Sec. A.3.1.3.4. Tier 2: Sec. 5.5.2.2.1). Note: more restrictive requirements for High Seismicity.

C NC N/A U PANEL ZONES: All panel zones have the shear capacity to resist the shear demand required to develop 0.8 times the sum of the flexural strengths of the girders framing in at the face of the column. (Commentary: Sec. A.3.1.3.5. Tier 2: Sec. 5.5.2.2.2)

C NC N/A U COLUMN SPLICES: All column splice details located in moment frames include connection of both flanges and the web, and the splice develops the strength of the column. (Commentary: Sec. A.3.1.3.6. Tier 2: Sec. 5.5.2.2.3)

C NC N/A U STRONG COLUMN—WEAK BEAM: The percentage of strong column–weak beam joints in each story of each line of moment-resisting frames is greater than 50%. (Commentary: Sec. A.3.1.3.7. Tier 2: Sec. 5.5.2.1.5)

C NC N/A U COMPACT MEMBERS: All frame elements meet section requirements set forth by AISC 341, Table D1.1, for highly ductile members. (Commentary: Sec. A.3.1.3.8. Tier 2: Sec. 5.5.2.2.4)

C NC N/A U BEAM PENETRATIONS: All openings in frame-beam webs are less than one quarter of the beam depth and are located in the center half of the beams. (Commentary: Sec. A.3.1.3.9. Tier 2: Sec. 5.5.2.2.5)

C NC N/A U GIRDER FLANGE CONTINUITY PLATES: There are girder flange continuity plates at all moment frame joints. (Commentary: Sec. A.3.1.3.10. Tier 2: Sec. 5.5.2.2.6)

C NC N/A U OUT-OF-PLANE BRACING: Beam–column joints are braced out-of-plane. (Commentary: Sec. A.3.1.3.11. Tier 2: Sec. 5.5.2.2.7)

C NC N/A U BOTTOM FLANGE BRACING: The bottom flanges of beams are braced out-of-plane. (Commentary: Sec. A.3.1.3.12. Tier 2: Sec. 5.5.2.2.8)

Diaphragms (Stiff or Flexible)

C NC N/A U PLAN IRREGULARITIES: There is tensile capacity to develop the strength of the diaphragm at reentrant corners or other locations of plan irregularities. (Commentary: Sec. A.4.1.7. Tier 2: Sec. 5.6.1.4)

C NC N/A U 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 N/A U OPENINGS AT FRAMES: Diaphragm openings immediately adjacent to the moment frames extend less than 15% of the total frame length. (Commentary: Sec. A.4.1.5. Tier 2: Sec. 5.6.1.3)

Flexible Diaphragms

C NC N/A U CROSS TIES: There are continuous cross ties between diaphragm chords. (Commentary: Sec. A.4.1.2. Tier 2: Sec. 5.6.1.2)

C NC N/A U STRAIGHT SHEATHING: All straight sheathed diaphragms have aspect ratios less than 1-to-1 in the direction being considered. (Commentary: Sec. A.4.2.1. Tier 2: Sec. 5.6.2)

C NC N/A U SPANS: All wood diaphragms with spans greater than 12 ft consist of wood structural panels or diagonal sheathing. (Commentary: Sec. A.4.2.2. Tier 2: Sec. 5.6.2)

C NC N/A U DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 30 ft and aspect ratios less than or equal to 3-to-1. (Commentary: Sec. A.4.2.3. Tier 2: Sec. 5.6.2)

C NC N/A U NONCONCRETE FILLED DIAPHRAGMS: Untopped metal deck diaphragms or metal deck diaphragms with fill other than concrete consist of horizontal spans of less than 40 ft and have aspect ratios less than 4-to-1. (Commentary: Sec. A.4.3.1. Tier 2: Sec. 5.6.3)

C NC N/A U 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)

High Seismicity: Complete theFollowing Items in Addition to the Items for Very Low, Low, and Moderate Seismicity.

Seismic-Force-Resisting System

C NC N/A U MOMENT-RESISTING CONNECTIONS: All moment connections are able to develop the strength of the adjoining members or panel zones based on 110% of the expected yield stress of the steel per AISC 341, Section A3.2. (Commentary: Sec. A.3.1.3.4. Tier 2: Sec. 5.5.2.2.1)

Foundation System

C NC N/A U DEEP FOUNDATIONS: Piles and piers are capable of transferring the seismic forces between the structure and the soil. (Commentary: Sec. A.6.2.3.)

C NC N/A U SLOPING SITES: The difference in foundation embedment depth from one side of the building to another does not exceed one story high. (Commentary: Sec. A.6.2.4)

STRUCTURAL ENGINEERS

Subject:	Lateral Analysis and Design	Job Number:		Date:	11/18/16
Job:		Engr:		Page:	

BUILDING BASE SHEAR AND LATERAL DESIGN:

2013 California Building Code (CBC) Equivalent Lateral Force Procedure Base Shear & Vertical Force Distribution

Based on ASCE 7-10 as amended by the 2013 CBC. All references are made to ASCE 7-10, unless otherwise noted.

Seismic Base Shear

Input Data:

Site Class =	D		Geotechnical Report
Nature of Occupancy =	Essential		Per Architect
Occupancy Category =	IV		Table 1-1
Seismic Design Category based on S_{D1}=	F		CBC, 1613.3.5
Seismic Design Category based on S_{DS}=	D		CBC, 1613.3.5
Governing Seismic Design Category =	**F**		CBC, 1613.3.5
Short Period, S_s =	2.48	g	Geotechnical Report
Site Coefficient, F_A =	1.00		Table 11.4-1
Maximum Considered Earthquake, S_{MS} =	2.48	g	Eqn 11.4-1
Damped Short Period Acceleration, S_{DS} =	1.66	g	Eqn 11.4-3
One Second Period, S_1 =	1.03	g	Geotechnical Report
Site Coefficient, F_V =	1.50		Table 11.4.2
Maximum Considered Earthquake, S_{M1} =	1.55	g	Eqn 11.4-2
Damped One Second Period Acceleration, S_{D1} =	1.03	g	Eqn 11.4-4
Importance factor, I =	1.50		Table 1.5-2

Design Maps Summary Report

User–Specified Input

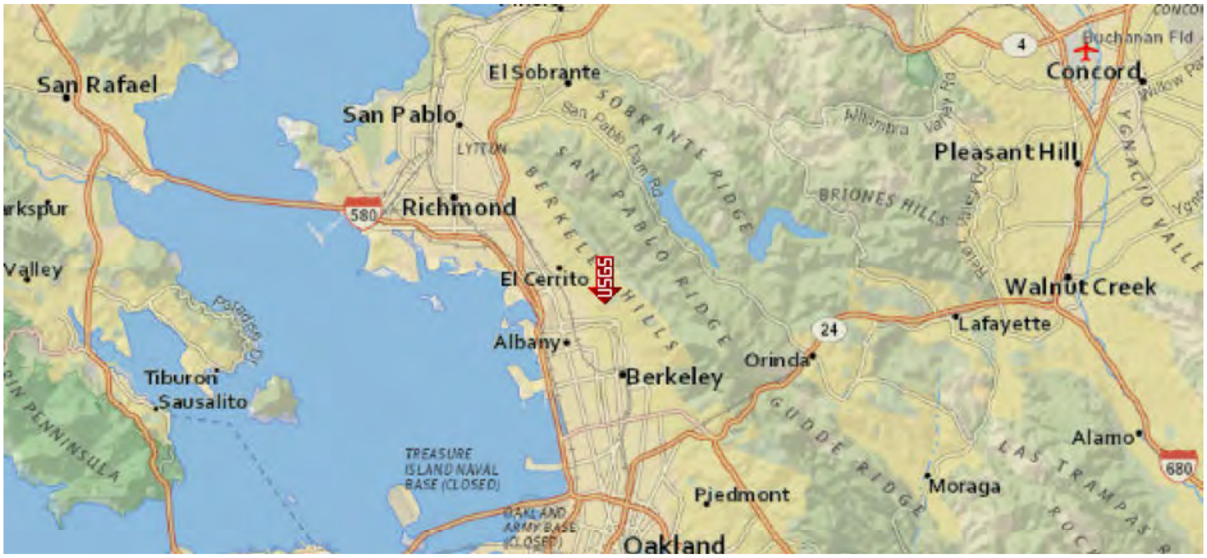
Report Title Kensington Firestation
Tue November 15, 2016 17:41:13 UTC

Building Code Reference Document ASCE 7-10 Standard
(which utilizes USGS hazard data available in 2008)

Site Coordinates 37.90616°N, 122.2789°W

Site Soil Classification Site Class D – "Stiff Soil"

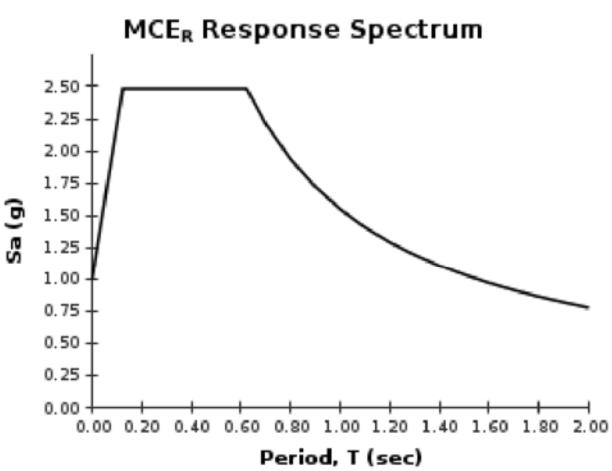
Risk Category IV (e.g. essential facilities)

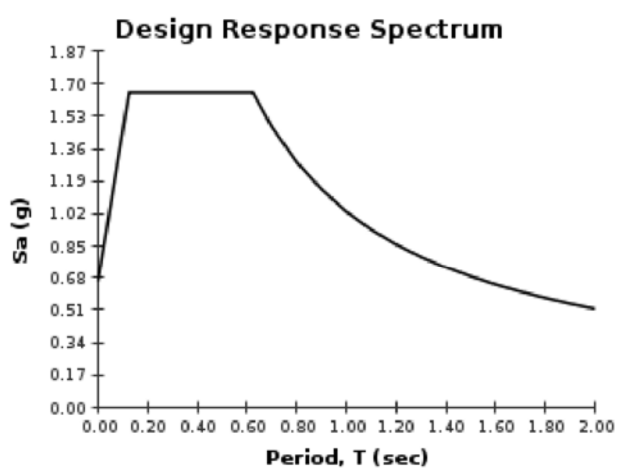


USGS–Provided Output

S_S =	2.483 g	S_{MS} =	2.483 g	S_{DS} =	1.655 g
S_1 =	1.031 g	S_{M1} =	1.547 g	S_{D1} =	1.031 g

For information on how the SS and S1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the "2009 NEHRP" building code reference document.



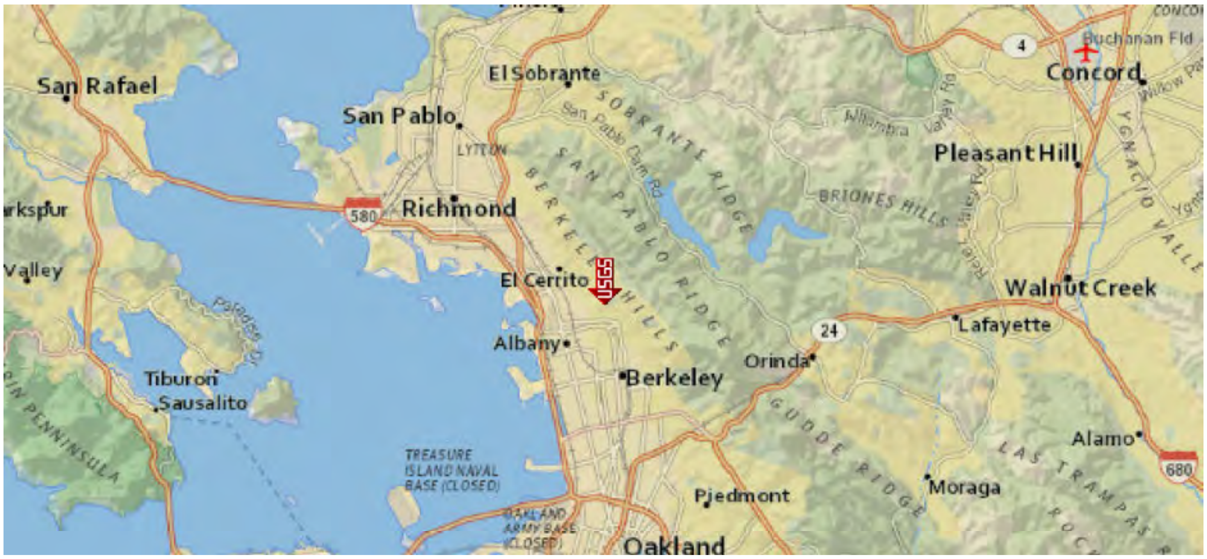


For PGA_M, T_L, C_{RS}, and C_{R1} values, please view the detailed report.

USGS Design Maps Summary Report

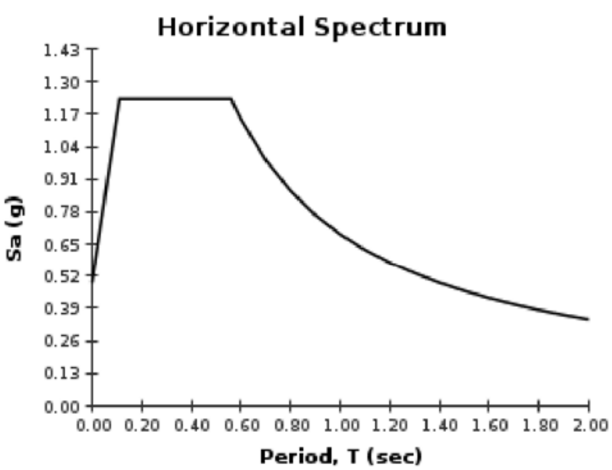
User–Specified Input

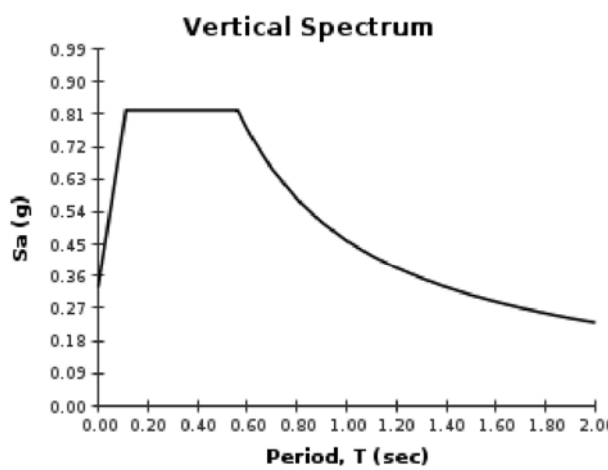
Report Title	Kensington Firestation Mon November 14, 2016 19:32:11 UTC
Building Code Reference Document	ASCE 41-13 Retrofit Standard, BSE-1E (which utilizes USGS hazard data available in 2008)
Site Coordinates	37.90616°N, 122.2789°W
Site Soil Classification	Site Class D – "Stiff Soil"



USGS–Provided Output

$S_{S,20/50}$	1.213 g	$S_{XS,BSE-1E}$	1.231 g
$S_{1,20/50}$	0.443 g	$S_{X1,BSE-1E}$	0.690 g





Design Maps Summary Report

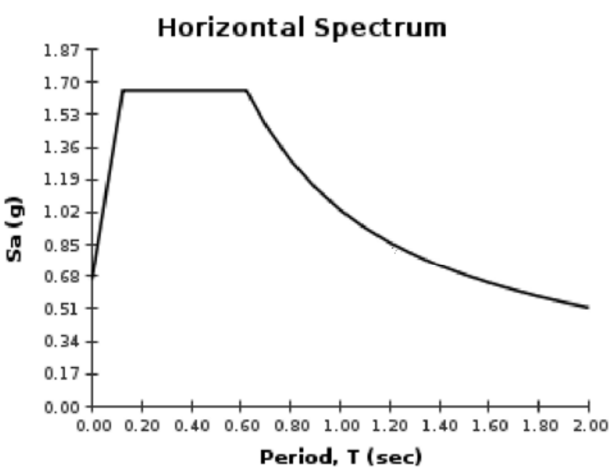
User–Specified Input

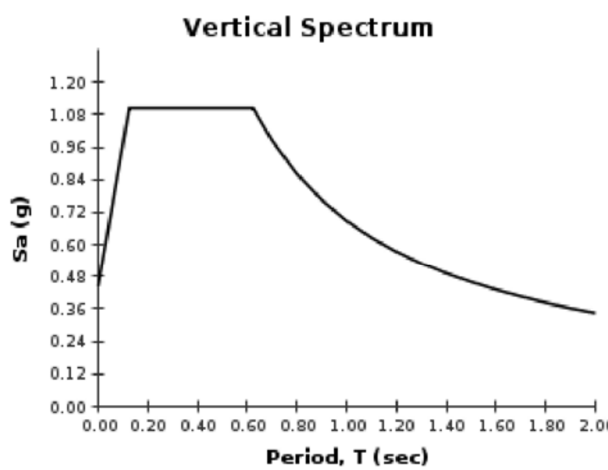
Building Code Reference Document	ASCE 41-13 Retrofit Standard, BSE-1N (which utilizes USGS hazard data available in 2008)
Site Coordinates	37.90656°N, 122.27925°W
Site Soil Classification	Site Class D – "Stiff Soil"

USGS–Provided Output

$S_{XS,BSE-1N}$ 1.656 g

$S_{X1,BSE-1N}$ 1.032 g





ASCE 41 Shear Stress in Shear Walls

Determine V, the pseudo lateral force from Equation 4-1. V is a function of

- C
- S_a, the response spectral acceleration at the fundamentla period of the building in the diretion under consideration. S_a shall be calculated in accordance with Section 4.5.2.3
- W, the total dead load

Building type — W1 Wood Light Frames

the modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response, taken from Table 4-8 — $C := 1.1$ — Number of stories=2

Determine S_a

1 second period spectral acceleration of the BSE-1E — $S_{X1} := 0.69$

Short period spectral acceleration of the BSE-1E Design — $S_{XS} := 1.231$

Factor per table 4-9 — $M_s := 2.0$ — Immediate Occupancy Level of Performance

Determine T

Coefficient to determine building period, from Section 4.5.2.4 — $C_t := 0.020$

Height in feet above the base to the roof level — $h_n := 22.5\ ft$

$\beta := 0.75$

Fundamental period of vibration of the building, calculated in accordance with Section 4.5.2.4 — $T := C_t \cdot \left(\frac{h_n}{1\ ft}\right)^{\beta} = 0.207$

Minimum base dimension $base := 40.66\ ft$

$$S_a := min\left(\frac{S_{X1}}{T}, S_{XS}\right) = 1.231$$

$$0.6 \cdot S_a = 0.74 \qquad \frac{base}{h_n} = 1.81$$

Minimum base dimension

$$Overturning := \mathbf{if}\left(\frac{base}{h_n} > 0.6 \cdot S_a, \text{“Compliant”}, \text{“Non compliant”}\right)$$

$$Overturning = \text{“Compliant”}$$

Arrays are second floor and roof

$$Floors := 2.0$$

Floor heights from base

$$h := \begin{bmatrix} 11.75 \\ 22.5 \end{bmatrix} ft$$

Length of the wall in North South Direction $L_{NS\_1stFlr} := 105\ ft$

$$L_{NS\_2ndFlr} := 51.4\ ft$$

Length of the wall in East West Direction $L_{EW\_1stFlr} := 84.63\ ft$

$$L_{EW\_2ndFlr} := 75.3\ ft$$

For wood-framed walls, the length shall be used rather than wall per 4.5.3.3

Area of walls in north south direction in

$$A_{wNS} := \begin{bmatrix} L_{NS\_1stFlr} \cdot \frac{1}{ft} \\ L_{NS\_2ndFlr} \cdot \frac{1}{ft} \end{bmatrix} ft^2$$

$$A_{wNS} = \begin{bmatrix} 105 \\ 51.4 \end{bmatrix} ft^2$$

Area of walls in east west direction

$$A_{wEW} := \begin{bmatrix} L_{EW\_1stFlr} \cdot \frac{1}{ft} \\ L_{EW\_2ndFlr} \cdot \frac{1}{ft} \end{bmatrix} ft^2$$

$$A_{wEW} = \begin{bmatrix} 84.63 \\ 75.3 \end{bmatrix} ft^2$$

$$FloorArea_{2ndFlr} := 79\ ft \cdot 41\ ft = 3239\ ft^2$$

$$FloorArea_{Roof} := 79\ ft \cdot 41\ ft = 3239\ ft^2$$

$$FloorWeight_{2ndFlr} := 24\ psf$$

$$FloorWeight_{Roof} := 19\ psf$$

$$WallWeights := 10\ psf$$

$$WallTrib_{2ndFlr} := \frac{11.75}{2}\ ft + \frac{10.75}{2}\ ft = 11.25\ ft$$

$$WallTrib_{Roof} := \frac{11.75}{2}\ ft = 5.88\ ft$$

Sesimic Weight of Walls :

$$w_{wall\_2ndfFlr} := 61.5\ kip$$

$$w_{wall\_Roof} := 9.5\ kip$$

See effective seismic weight calculations for walls and moment frame

$SeismicWeight_{2ndFlr} := FloorArea_{2ndFlr} \cdot FloorWeight_{2ndFlr} + w_{wall\_2ndfFlr}$

$SeismicWeight_{2ndFlr} = 139.24\ \mathbf{kip}$

$SeismicWeight_{Roof} := FloorArea_{Roof} \cdot FloorWeight_{Roof} + w_{wall\_Roof}$

$SeismicWeight_{Roof} = 71.04\ \mathbf{kip}$

Portion of total seismic weight on each floor, the first element in the array is for first floor and so on

$$w := \begin{bmatrix} 140 \\ 71 \end{bmatrix} \mathbf{kip}$$

Total seismic weight of structure

$$W := \sum_{i=1}^{\text{length}(w)} w_i = 211\ \mathbf{kip}$$

Psuedo seismic force per 4.5.2.1 Eq. 4-1

$$V := C \cdot S_a \cdot W = 286\ \mathbf{kip}$$

Factor per 4.5.2.2

$$k := \mathbf{if}\left(T > 2.5, 2, \mathbf{if}\left(T \le 0.5, 1, 0.5 \cdot T + 0.75\right)\right)$$

$$k = 1$$

$$x := 1 \,..\, Floors$$
$$j := 1 \,..\, Floors$$

Vertical distribution of psuedo seismic force per 4.5.2.2 Eq 4-3a

$$F_x := \frac{w_x \cdot h_x^{\ k}}{\sum_{i=1}^{Floors} w_i \cdot h_i^{\ k}} \cdot V = \begin{bmatrix} 145 \\ 141 \end{bmatrix} \mathbf{kip}$$

Story shear at story level j

$$V_j := \sum_{x=j}^{Floors} F_x = \begin{bmatrix} 286 \\ 141 \end{bmatrix} \, \mathbf{kip}$$

Shear stress in shear walls
in north sourth direction

$$v_{NS_j} := \frac{1}{M_s} \cdot \frac{V_j}{A_{wNS_j} \cdot \frac{1}{\mathbf{ft}}} = \begin{bmatrix} 1360.55 \\ 1369.31 \end{bmatrix} \, \mathbf{plf}$$

Shear stress in shear walls
in east west direction

$$v_{EW_j} := \frac{1}{M_s} \cdot \frac{V_j}{A_{wEW_j} \cdot \frac{1}{\mathbf{ft}}} = \begin{bmatrix} 1688.02 \\ 934.69 \end{bmatrix} \, \mathbf{plf}$$

ASCE 41 Quick check limit:

$ShearstressAllowable := 1000 \, \mathbf{plf}$

The shear stress in the shear walls,calculated using the Quick Check procedure of Section 4.5.3.3 is less than than allowable value of 1000plf

DATE: PAGE:

BY: JOB No.

PROJECT:

Effective Seismic weight calculations of walls and moment frame

LENGTH CONC. WALL ALONG GRID A IN 1ST FLOOR.

$L_{CONCWALL} = 45.75' + 5.75' + 5.75' + 5.6' + 3.75' = 66.6'$

WGT. OF CONC WALL TRIB TO 2ND FLOOR.

$W_{CONCWALL} = 150pcf \times \left(\frac{6}{12}\right)' \times 66.6' \times \frac{11.75'}{2}$

$= 30^K$

WGT. OF OMF COLUMNS. TRIB TO. 2ND FLOOR.

$W_{COL-OMF} = (4 NOS)\left(\frac{11.75}{2}'\right)\times(58\#)$

W12X58 $= 1.4^K$.

WGT. OF OMF BEAMS.

$W_{BM-OMF} = ((13.5') + (13.25') + (13.25'))\times(40\#)$

$= 1.6^K$

LENGTH OF WALL ALONG. GRID 1 IN FIRST FLOOR.

$L_{GRID1-1STFLR} = 36.75'$

LENGTH OF WALL ALONG. GRID. E 1ST FLOOR.

$L_{GRIDE-1STFLR} = 2.75' + 1.75' + 5.75' + 5.75' + 5.75' + 2.6'$

$= 25'$

LENGTH OF WALL ALONG. GRID 7 1ST FLOOR

$L_{GRID7-1STFLR} = 41'$ (NO OPENINGS DEDUCTED)

DATE: PAGE:

BY: JOB No.

PROJECT:

LENGTH OF WALL ALONG GRID A 2ND FLOOR

$$L_{GRIDA-2ND\,FLR} = 2' + 1.5' + 7.5' + 8' + 5.5' + 5.75' + 8.5' + 2.75' + 10.25' = 52'$$

LENGTH OF WALL ALONG GRID E 2ND FLOOR.

$$L_{GRIDE-2ND\,FLOOR} = 15.75' + 3.5' + 2' + 4.75' + 1.25' + 5.5' + 5' + 5.75' + 5.75' + 10.25' = 60'$$

LENGTH OF WALL ALONG GRID 1 2ND FLOOR.

$$L_{GRID1-2ND\,FLOOR} = 24'$$

LENGTH OF WALL ALONG GRID 7 2ND FLOOR

$$L_{GRID7-2ND\,FLOOR} = 41'$$

SEISMIC WGT. OF WALLS TRIB TO 2ND FLOOR

$$= 30^K + 1.4^K + 1.6^K + (36.75' + 24')\left(\frac{11.75'}{2} + \frac{10.75'}{2}\right) \times (10\,psf)$$
$$+ (52')\left(\frac{10.75'}{2}\right) \times (10\,psf)$$
$$+ (25' + 60')\left(\frac{11.75'}{2} + \frac{10.75'}{2}\right) \times (10\,psf)$$
$$+ (41' + 41')\left(\frac{11.75'}{2} + \frac{10.75'}{2}\right) \times (10\,psf)$$
$$= 61.5^K.$$

Tier 2 analysis of shear wall

Total seismic weight of the building $W = 211$ ***kip***

C_1 Modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response

C_2 Modification factor to represent the effect of pinched hysteresis shape, cyclic stiffness degradation, and strength deterioation on maximum displacement response.

Cm Effective mass factor to account for higher modal mass participation effects

$C_1C_2 := 1.1$ Table 7-3

$C_m := 1.0$ Table 7-4

$S_a = 1.23$

The Pseudo lateral force in a given horizontal direction of a building is determined using Eq. (7-21) :

$$V := C_1C_2 \cdot C_m \cdot S_a \cdot W$$

$$V = 285.72 \ \textbf{kip}$$

Vertical distribution of psuedo seismic force per 7.4.1.3.2 Eq (7-24)

$$F_x := \frac{w_x \cdot h_x^{\ k}}{\sum_{i=1}^{Floors} w_i \cdot h_i^{\ k}} \cdot V = \begin{bmatrix} 145 \\ 141 \end{bmatrix} \textbf{kip}$$

Story shear at story level j

$$V_j := \sum_{x=j}^{Floors} F_x = \begin{bmatrix} 286 \\ 141 \end{bmatrix} \textbf{kip}$$

Diaphragm inertial force

$$F_{p_x} := \frac{\sum_{i=1}^{Floors} F_i}{\sum_{i=1}^{Floors} w_i} \cdot w_x = \begin{bmatrix} 190 \\ 96 \end{bmatrix} \, kip$$

EXPECTED STRENGTH OF WOOD STRUCTURAL PANEL SHEATHING DIAPHRAGM PER C12.5-3.6.2:

DIAPHRAGM INERTIAL FORCE-ROOF = F_{px} = 96 kips.

FOR NORTH-SOUTH DIRECTION EQ.

$$Q_{ud\,DIAPHRAMS} = \frac{F_{px}/2}{D} = \frac{96/2}{79'} = 0.60\ klf$$

(E) ROOF SHEATHING: 1/2" PLYWOOD
CDX GRADE.
8d @ 4" O.C BOUNDARY NAILING.
8d @ 10" O.C FIELD NAILING.
LRFD SHEAR CAPACITY = 2x320 plf (ϕ = 1.0)
= 640 plf

$\Rightarrow$ $Q_{CE\,DIAPHRAGMS}$ = 640 plf = 0.64 klf.

$$\frac{Q_{ud\,DIAPHRAGMS}}{Q_{CE\,DIAPHRAGMS}} = \frac{0.60\ klf}{0.64\ klf} = 0.94\ \underline{\underline{OK}}$$

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STORY FORCE @ SECOND FLOOR FOR N-S. DIRECTION EQ. = 190^k.

$$Q_{ud} = \frac{\frac{190^k}{2}}{79'} = \frac{95^k}{79'} = 1.20 \text{ klf}.$$

(E) FLOOR SHEATHING: 3/4" PW.

10d @ 4" O.C BOUNDARY NAILING

10d @ 4" O.C. EDGE NAILING.

10d @ 10" O.C FIELD NAILING.

LRFD SHEAR CAPACITY = 2 X 425 plf
($\phi = 1.0$) = 850 plf.
= 0.85 klf.

$$\frac{Q_{ud}}{Q_{CE}} = \frac{1.20^{klf}}{0.85 \text{ klf}} = 1.41 \text{ klf}. \quad \underline{\underline{NG}}$$

Calculation of available Shear Wall Length

Length of shear wall in N-S
Direction in Ground Floor:

$$L_{Ground\_GridA} := 45.75\ ft + 5.66\ ft + 5.75\ ft + 5.5\ ft = 62.66\ ft$$

Length of shear wall in E-W
Direction in Ground Floor :

$$L_{Ground\_Grid1} := 36.75\ ft$$

$$L_{Ground\_Grid4} := 12\ ft + 8\ ft = 20\ ft$$

$$L_{Ground\_Grid7} := 6.83\ ft + 14\ ft + 8.5\ ft = 29.33\ ft$$

Length of shear wall in N-S
Direction in Second Floor :

$$L_{Second\_GridA} := 7.5\ ft + 8\ ft + 5.5\ ft + 5.75\ ft + 8.6\ ft + 10.25\ ft = 45.6\ ft$$

$$L_{Second\_GridE} := 4.9\ ft + 5.5\ ft + 5\ ft + 5.66\ ft + 5.66\ ft + 10.25\ ft = 36.97\ ft$$

Length of shear wall in E-W
Direction in Second Floor :

$$L_{Second\_Grid1} := 24\ ft$$

$$L_{Second\_Grid4} := 17\ ft + 5.66\ ft = 22.66\ ft$$

$$L_{Second\_Grid7} := 24.25\ ft + 4.42\ ft + 8.42\ ft = 37.09\ ft$$

Calculation of Shear Loads to the Shear Walls

Roof

North South Direction

$V_x := 141\ kip$ Input Story Shear

Shear to Walls along grid A :

$R_{A\_Roof} := \frac{V_x}{2} = 70.5\ kip$

$v_{A\_Roof} := \frac{R_{A\_Roof}}{L_{Second\_GridA}} = 1546.05\ plf$ $L_{Second\_GridA} = 45.6\ ft$

Shear to Walls along grid E:

$R_{E\_Roof} := \frac{V_x}{2} = 70.5\ kip$

$v_{E\_Roof} := \frac{R_{E\_Roof}}{L_{Second\_GridE}} = 1906.95\ plf$ $L_{Second\_GridE} = 36.97\ ft$

East West Direction

Shear to Walls along grid 1 :

$R_{1\_Roof} := \frac{21\ ft}{79\ ft} \cdot V_x = 37.48\ kip$

$v_{1\_Roof} := \frac{R_{1\_Roof}}{L_{Second\_Grid1}} = 1561.71\ plf$

Shear to Walls along grid 4 :

$R_{4\_Roof} := \frac{(45\ ft + 37\ ft) \cdot 0.5}{79\ ft} \cdot V_x = 73.18\ kip$

$v_{4\_Roof} := \frac{R_{4\_Roof}}{L_{Second\_Grid4}} = 3229.36\ plf$ $L_{Second\_Grid4} = 22.66\ ft$

Shear to Walls along grid 7 : $R_{7\_Roof} := \frac{37\ ft \cdot 0.5}{79\ ft} \cdot V_x = 33.02\ kip$

$v_{7\_Roof} := \frac{R_{7\_Roof}}{L_{Second\_Grid7}} = 890.24\ plf$ $L_{Second\_Grid7} = 37.09\ ft$

Second Floor

North South Direction

$V_x := 286\ kip$ Input Story Shear

Shear to Walls along grid A : $R_{A\_2ndFlr} := \frac{V_x}{2} = 143\ kip$

$v_{A\_2ndFloor} := \frac{R_{A\_2ndFlr}}{L_{Ground\_GridA}} = 2282.16\ plf$ $L_{Ground\_GridA} = 62.66\ ft$

Shear to Walls along grid E: $R_{E\_2ndFlr} := \frac{V_x}{2} = 143\ kip$ Moment Frame

East West Direction

Shear to Walls along grid 1 : $R_{1\_2ndFlr} := \frac{21\ ft}{79\ ft} \cdot V_x = 76.03\ kip$

$v_{1\_2ndFlr} := \frac{R_{1\_2ndFlr}}{L_{Ground\_Grid1}} = 2068.72\ plf$

Shear to Walls along grid 4 : $R_{4\_2ndFlr} := \frac{(45\ ft + 37\ ft) \cdot 0.5}{79\ ft} \cdot V_x = 148.43\ kip$

$$v_{4\_2ndFlr} := \frac{R_{4\_2ndFlr}}{L_{Ground\_Grid4}} = 7421.52\ \mathbf{plf} \qquad L_{Ground\_Grid4} = 20\ \mathbf{ft}$$

Shear to Walls along grid 7 :

$$R_{7\_2ndFlr} := \frac{37\ \mathbf{ft} \cdot 0.5}{79\ \mathbf{ft}} \cdot V_x = 66.97\ \mathbf{kip}$$

$$v_{7\_2ndFlr} := \frac{R_{7\_2ndFlr}}{L_{Ground\_Grid7}} = 2283.49\ \mathbf{plf} \qquad L_{Ground\_Grid7} = 29.33\ \mathbf{ft}$$

Existing Allowable Shear Wall Capacity

Acceptance Criteria for Deformation Controlled Actions for LSP,Section 7.5.2.2

$m := 1.7$ Component modification factor to account for expected ductility. For linear procedures,m-factors for use with deformation-controlled actions shall be taken from Table 12-3.

$k := 0.9$ Knowledge factor defined per section 6.2.4/Table 6-1

Q_{CE} Expected strength of wood structural panel sheathing per Section 12.4.4.6.2. Expected strengths of wood structural panel shear walls shall be permitted to based on 1.5 times yield strengths. Yield strength shall be determined using LRFD procedure contained in AWC SDPWS, except the resistance factor, ϕ, shall be taken as 1.0

Roof

North South Direction

Shear wall capacity along Grid A

$Q_{CE\_AWall} := 1.5 \cdot 255\ \textbf{plf} \cdot 2 \cdot 1 = 765\ \textbf{plf}$

(10d nails @ 6" oc edge nailing)

$m \cdot k \cdot Q_{CE\_AWall} = 1170.45\ \textbf{plf}$

$m \cdot k \cdot Q_{CE\_AWall} \cdot L_{Second\_GridA} = 53.37\ \textbf{kip}$

$Q_{ad} := R_{A\_Roof} = 70.5\ \textbf{kip}$

$$\frac{Q_{ad}}{m \cdot k \cdot Q_{CE\_AWall} \cdot L_{Second\_GridA}} = 1.32$$ NG

Shear wall capacity along Grid E

$Q_{CE\_AWall} := 1.5 \cdot 255\ \mathbf{plf} \cdot 2 \cdot 1 = 765\ \mathbf{plf}$

$m \cdot k \cdot Q_{CE\_AWall} = 1170.45\ \mathbf{plf}$

$m \cdot k \cdot Q_{CE\_AWall} \cdot L_{Second\_GridE} = 43.27\ \mathbf{kip}$

$Q_{ad} := R_{E\_Roof} = 70.5\ \mathbf{kip}$

$$\frac{Q_{ad}}{m \cdot k \cdot Q_{CE\_AWall} \cdot L_{Second\_GridE}} = 1.63$$ NG

East West Direction

Shear wall capacity along Grid 1

$Q_{CE\_AWall} := 1.5 \cdot 255\ \mathbf{plf} \cdot 2 \cdot 1 = 765\ \mathbf{plf}$

$m \cdot k \cdot Q_{CE\_AWall} = 1170.45\ \mathbf{plf}$

$m \cdot k \cdot Q_{CE\_AWall} \cdot L_{Second\_Grid1} = 28.09\ \mathbf{kip}$

$Q_{ad} := R_{1\_Roof} = 37.48\ \mathbf{kip}$

$$\frac{Q_{ad}}{m \cdot k \cdot Q_{CE\_AWall} \cdot L_{Second\_Grid1}} = 1.33$$ NG

Shear wall capacity along Grid 4:

$Q_{CE\_AWall} := 1.5 \cdot 255\ \mathbf{plf} \cdot 2 \cdot 1 = 765\ \mathbf{plf}$ Added in 1998

$Q_{CE\_CWall} := 1.5 \cdot 310\ \mathbf{plf} \cdot 2 \cdot 1 = 930\ \mathbf{plf}$ Added in 2004

$m \cdot k \cdot \left(Q_{CE\_AWall} + Q_{CE\_CWall}\right) \cdot 17\ \mathbf{ft} + m \cdot k \cdot Q_{CE\_AWall} \cdot 5.66\ \mathbf{ft} = 50.71\ \mathbf{kip}$

$Q_{ad} := R_{4\_Roof} = 73.18\ \mathbf{kip}$

$$\frac{Q_{ad}}{m \cdot k \cdot \left(Q_{CE\_AWall} + Q_{CE\_CWall}\right) \cdot 17\ \mathbf{ft} + m \cdot k \cdot Q_{CE\_AWall} \cdot 5.66\ \mathbf{ft}} = 1.44$$ NG

Shear wall capacity along Grid 7:

$Q_{CE\_AWall} := 1.5 \cdot 255\ \mathbf{plf} \cdot 2 \cdot 1 = 765\ \mathbf{plf}$

$m \cdot k \cdot Q_{CE\_AWall} \cdot L_{Second\_Grid7} = 43.41\ \mathbf{kip}$

$Q_{ad} := R_{7\_Roof} = 33.02\ \mathbf{kip}$

$\dfrac{Q_{ad}}{m \cdot k \cdot Q_{CE\_AWall} \cdot L_{Second\_Grid7}} = 0.76$ OK

Second Floor

North South Direction

Shear wall capacity along Grid A

$Q_{ad} := R_{A\_2ndFlr} = 143\ \mathbf{kip}$

Shear to concrete shear wall

Shear wall capacity along Grid E

$Q_{ad} := R_{E\_2ndFlr} = 143\ \mathbf{kip}$

Loads to Moment Frame

East West Direction

Shear wall capacity along Grid 1

$Q_{CE\_AWall} := 1.5 \cdot 255\ \mathbf{plf} \cdot 2 \cdot 1 = 765\ \mathbf{plf}$

$m \cdot k \cdot Q_{CE\_AWall} = 1170.45\ \mathbf{plf}$

$m \cdot k \cdot Q_{CE\_AWall} \cdot L_{Ground\_Grid1} = 43.01\ \mathbf{kip}$

$Q_{ad} := R_{1\_2ndFlr} = 76.03\ \mathbf{kip}$

$$\frac{Q_{ad}}{m \cdot k \cdot Q_{CE\_AWall} \cdot L_{Ground\_Grid1}} = 1.77$$ NG

Shear wall capacity along Grid 4:

$Q_{CE\_DWall} := 1.5 \cdot 600\ \mathbf{plf} \cdot 2 \cdot 1 = 1800\ \mathbf{plf}$

$Q_{CE\_BWall} := 1.5 \cdot 460\ \mathbf{plf} \cdot 2 \cdot 1 = 1380\ \mathbf{plf}$

$$m \cdot k \cdot \left(Q_{CE\_BWall} + Q_{CE\_DWall}\right) \cdot 8\ \mathbf{ft} + m \cdot k \cdot \left(Q_{CE\_DWall} \cdot 12\ \mathbf{ft} + Q_{CE\_BWall} \cdot 18\ \mathbf{ft}\right) = 109.98\ \mathbf{kip}$$

$Q_{ad} := R_{4\_2ndFlr} = 148.43\ \mathbf{kip}$

$$\frac{Q_{ad}}{m \cdot k \cdot \left(Q_{CE\_BWall} + Q_{CE\_DWall}\right) \cdot 8\ \mathbf{ft} + m \cdot k \cdot \left(Q_{CE\_DWall} \cdot 12\ \mathbf{ft} + Q_{CE\_BWall} \cdot 18\ \mathbf{ft}\right)} = 1.35$$ NG

Shear wall capacity along Grid 7:

$Q_{CE\_AWall} := 1.5 \cdot 255\ \mathbf{plf} \cdot 2 \cdot 1 = 765\ \mathbf{plf}$

$m \cdot k \cdot Q_{CE\_AWall} \cdot L_{Ground\_Grid7} = 34.33\ \mathbf{kip}$

$Q_{ad} := R_{7\_2ndFlr} = 66.97\ \mathbf{kip}$

$$\frac{Q_{ad}}{m \cdot k \cdot Q_{CE\_AWall} \cdot L_{Ground\_Grid7}} = 1.95$$ NG

Conclusion : The existing shear walls are not adeqaute to resist the seismic forces

Immediate Occupancy Structural Checklist for Building Types S1A: Steel Moment Frames with Flexible Diaphragms

Section 4.5.3.1 Story Drift for Moment Frames ,
Quick Check Procedure

$h := 13.5\ ft$	Story Height (in)
$I_b := 307\ in^4$	Moment of Inertia of beam (in^4)
$I_c := 475\ in^4$	Moment of Inertia of beam (in^4)
$L := 161\ in$	Beam Length from center-to-center of adjacent columns (in)
$E := 29000\ ksi$	Modulus of elasticity (kip/in^2)
$V_c := \frac{286}{2}\ kip$ $V_c = 143\ kip$	Shear in the column (kip). The column shear forces are calculated using the story forces in accordance with Section 4.5.2.2
$k_b := \frac{I_b}{L}$	for the representative beam
$k_c := \frac{I_c}{h}$	for the representative column

Drift Ratio: $D_r := \frac{(k_b + k_c)}{k_b \cdot k_c} \cdot \frac{h}{12 \cdot E} \cdot V_c = 0.0576$

$\textbf{if}\left(D_r < 0.015, \text{“OK”}, \text{“NG”}\right) = \text{“NG”}$

Tier 2 evaluation of Drift :

Demands : $Q_{ud\_col} := 396\ kip \cdot ft$ Based on RISA 3d analysis of frame

$Q_{ud\_beam} := 396\ kip \cdot ft$

Expected Strength of Beams :

$M_{CE} := 114\ in^3 \cdot 36\ ksi = 4104\ kip \cdot in$ W12x40 beam with 5/8"x4.5" wide stiff plate

$Q_{CE\_beam} := M_{CE} = 4104\ kip \cdot in$

$m := 2.0$ Table 9-4, Beams-Flexure, IO

$Q_{CE\_beam} \cdot m \cdot k = 7387.2\ kip \cdot in$

$\frac{Q_{ud\_beam}}{Q_{CE\_beam} \cdot m \cdot k} = 0.64$ Comply

Expected Strength of Columns:

Note: Assuming negligible axial load on the columns

$M_{CE} := 167\ in^3 \cdot 36\ ksi = 6012\ kip \cdot in$ W12x58 columns with 3/4"x5.5" flange stiff plate

$Q_{CE\_col} := M_{CE} = 6012\ kip \cdot in$

$Q_{ud\_col} = 4752\ kip \cdot in$

$m := 2.0$ Table 9-4, Columns-Flexure, IO

$\frac{Q_{ud\_col}}{m \cdot Q_{CE\_col}} = 0.4$ Comply

Conclusion:
The anlaysis of frame was performed in accordance with Section 5.2.4.
Adequacy of the beams and columns was checked per Tier 2: Section 5.5.2.1.2 .
The strength of the beams and columns is adequate. The moment frame comply the drift check.

COLUMN AXIAL STRESS CHECK USING QUICK CHECK PROCEDURE

$f_y := 36\ \textbf{ksi}$ A36 steel

$0.30\ f_y = 10.8\ \textbf{ksi}$

Column Axial stress Caused by
Overturning calculated using quick
check procedure of Section 4.5.3.6

$n_f := 3$ Total number of frames in the direction of loading

$V := 143\ \textbf{kip}$ Pseudo Seismic force

$h_n := 13.5\ \textbf{ft}$ Height above the base to the roof level

$L := 39.832\ \textbf{ft}$ Total length of the frame

$M_s := 1.3$ System Modification Factor Immediate Occupancy Performace Level

$A_{col} := 17\ \textbf{in}^2$ Area of the end column of the frame

$$p_{ot} := \frac{1}{M_s} \cdot \left(\frac{2}{3}\right) \cdot \left(\frac{V \cdot h_n}{L \cdot n_f}\right) \cdot \left(\frac{1}{A_{col}}\right) = 0.49\ \textbf{ksi}$$

$p_{ot} < 0.30\ f_y$ *ok*

FLEXURAL STRESS CHECK USING QUICK CHECK PROCEDURE OF SECTION 4.5.3.9:

$n_c := 4$ Total number of frame columns at the level, j, under consideration

$n_f := 3$ Total number of frames in the direction of loading at the level ,j, under consideration

$V_j := 143\ \textbf{kip}$ Story shear computed in accordance with Section 4.5.2.2

$h := 13.5\ \textbf{ft}$ Story Height

$Z_c := 4 \cdot 167\ \textbf{in}^3 = 668\ \textbf{in}^3$ The sum of the plastic section moduli of all the frame columns at the level under consideration

$Z_b := 6 \cdot 114\ \textbf{in}^3 = 684\ \textbf{in}^3$ The sum of the plastic section moduli of all the frame beams with moment resisting connections at the level under consideration

$M_s := 3.0$ Immediate Occupancy System Modification Factor

$$f_{j\_col} := V_j \cdot \frac{1}{M_s} \cdot \frac{n_c}{n_c - n_f} \cdot \frac{h}{2} \cdot \frac{1}{Z_c} = 23.12\ \textbf{ksi}$$ < Fy=36ksi OK

$$f_{j\_beam} := V_j \cdot \frac{1}{M_s} \cdot \frac{n_c}{n_c - n_f} \cdot \frac{h}{2} \cdot \frac{1}{Z_b} = 22.58\ \textbf{ksi}$$ < Fy=36ksi OK

PANEL ZONES (MODERATE SEISMICITY) Ref: 9.4.2.3 Strength of FR Moment Frames

$d_c := 12.2\ in$ Column depth W12x58

$d_b := 12\ in$ Depth of W12x40 beam

$t_{fb} := 0.515\ in$ Thickness of W12x40 flange beam

$E := 29000\ ksi$ Modulus of elasticity

$F_{ye} := 36\ ksi$ Expected Yield strength of the material, A36 steel

$t_p := \frac{1}{2}\ in \cdot 2 + 0.36\ in$ Total thickness of panel zone including doubler, 1/2" thk dblr plate both sides

The expected plastic shear capacity of the panel zone :

$V_{CE} := 0.55\ F_{ye} \cdot d_c \cdot t_p$ Equation 9-5

$V_{CE} = 328.52\ kip$

$m := 1.5$ Column panel zone shear, Immediate Occupancy, Table 9-4,deformation controlled

$m \cdot k \cdot V_{CE} = 443.5\ kip$

The plastic moment capacity of beam:

$Z := 57\ in^3$ Plastic section modulus of W12x40 beam

$M_{CE} := Z \cdot F_{ye} = 2052\ kip \cdot in$

$\Sigma M_{CE} := 2 \cdot M_{CE} = 4104\ kip \cdot in$

Shear Demand :

$$d_z := d_b - t_{fb} = 11.49\ \mathbf{in}$$ Depth of panel zone

$$V_{ud} := \frac{\Sigma M_{CE}}{d_z} = 357.34\ \mathbf{kip}$$

Demand Capacity Ratio
Panel Zone Strength :

$$\frac{V_{ud}}{m \cdot k \cdot V_{CE}} = 0.81$$ OK

MOMENT RESISTING CONNECTION CHECK (MODERATE SEISMICITY AND HIGH SEISMICITY):

The expected shear strength of beam :

$t_w := 0.295\ \boldsymbol{in}$ $d := 12\ \boldsymbol{in}$ $t_f := 0.515\ \boldsymbol{in}$ $d - 2 \cdot t_f = 10.97\ \boldsymbol{in}$

$A_w := t_w \cdot \left(d - 2 \cdot t_f\right) = 3.24\ \boldsymbol{in}^2$

$V := 0.6 \cdot F_{ye} \cdot A_w = 69.9\ \boldsymbol{kip}$ Equation 9-7

$V_{ud} := V = 69.9\ \boldsymbol{kip}$

Strength of beam web-to-column connection weld :

$V_{CE} := 1.39\ \dfrac{\boldsymbol{kip}}{\boldsymbol{in}} \cdot 5 \cdot 2 \cdot \left(d - 2\ t_f\right) = 152.48\ \boldsymbol{kip}$

(5/16" fillet weld provided at both side of shear plate to column connection)

$m := 1.0$

$m \cdot k \cdot V_{CE} = 137.23\ \boldsymbol{kip}$

Demand Capacity Ratio

$\dfrac{V_{ud}}{m \cdot k \cdot V_{CE}} = 0.51$ OK for moderate seismicity

$1.1 \cdot \dfrac{V_{ud}}{m \cdot k \cdot V_{CE}} = 0.56$ OK for high seismicity

STRONG COLUMN WEAK BEAM (MODERATE SEISMICITY)

STRONG COLUMN-WEAK BEAM (MODERATE SEISMICITY)

$m := 2.5$ Tier2: Section 5.5.2.1.5

$Z_c = 668\ in^3$ Plastic section modulus of Columns

$Z_b = 684\ in^3$ Plastic section modulus of Beams

$2 \cdot Z_b = 1368\ in^3$

$p_{ot} = 0.49\ ksi$ Axial stress in the column due to overturning using quick check procedure

$f_a := p_{ot} = 0.49\ ksi$

$$\frac{Z_c \cdot \left(F_{ye} - f_a\right)}{2 \cdot Z_b \cdot F_{ye}} = 0.482$$

$$m \cdot \frac{Z_c \cdot \left(F_{ye} - f_a\right)}{2 \cdot Z_b \cdot F_{ye}} = 1.2$$ >1.0 Comply

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COMPACT MEMBER CHECK

W 12X58

$\frac{bf}{2tf} = 7.82$. OK

$0.30 \times \sqrt{\frac{29000 ksi}{36 ksi}} = 8.51$

$\frac{h}{tw} = 27$. OK

$2.45 \sqrt{\frac{29000 ksi}{36 ksi}} = 69.56$

W 12X40

$\frac{bf}{2tf} = 7.77$ OK

$\frac{h}{tw} = 33.6$. OK