STRUCTURAL ENGINEERS

Conceptual Retrofit Design based on ASCE 41-17 Tier 1 and 2 Seismic Evaluation

Kensington Public Safety Building
217 Arlington Avenue
Kensington, CA 94707

Prepared for: Ross Drulis Cusenbery Architecture, Inc. **September 5, 2019**

IDA Project Number 16066

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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-41-17, Tier 2 seismic evaluation procedure. ASCE 41-17, titled "*Seismic Evaluation and Retrofit of Existing Buildings,*" published by the American Society of Civil Engineers (ASCE) in 2017, 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)	<u>BSE-1E</u> Immediate Occupancy Structural Performance (S-1) Position Retention Nonstructural Performance (1-B) <u>BSE-2E</u> Life Safety Structural Performance (S-3) Hazards Reduced Nonstructural Performance (3-D)
Seismic Hazard Level	BSE-1E 20% in 50 years, 225 year return period BSE-2E 5% in 50 years, 975 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.

2 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.

For this Risk Category IV building, it is required to evaluate the building for the BSE-1E for Immediate Occupancy Structural Performance and Life Safety Structural Performance for the BSE-2E. The Nonstructural Performance Level is 1-B for the BSE-1E and 3-D for the BSE-2E 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.

2.1.1 Structural Performance Level for BPOE

The structural performance level for BPOE is S-1 (Immediate Occupancy) performance for the BSE-1E, which provides for of the building following a 20% in 50 year earthquake and and S-3 (Life Safety) for the BSE-2E performance following a 10% in 50 year 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.

2.1.2 Nonstructural Performance Level for BPOE

An evaluation of non-structural elements and systems were not included as part of the scope of this evaluation.

2.1.3 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 Immediate Occupancy and 2E, which requires ground motions with a 5% exceedance in 50 years (or a 975 year recurrence interval). A higher recurrence interval represents a larger earthquake with a statistically rarer occurrence. 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 41-17 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, 225 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.

3 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.

4 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.

5 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.

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

An updated geotechnical study was performed by Rockridge Geotechnical dated January 31, 2018. The primary purpose of this study was to identify fault traces with a seismic refraction study to determine the feasibility of constructing a new building beyond the existing footprint of the building. Given the limitations of the seismic refraction study, the Geotechnical Engineer has recommended a 50 foot setback from the suspected fault feature at the eastern property line. This limitation severely limits the ability to extend the building eastward beyond the existing footprint.

6 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.

7 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.

8 Code Assessment for Future Renovation

In order to determine extent of potential renovations, research of the code was performed to find guidance on construction of Essential Services Facilities on or near earthquake fault traces. Based on this research we believe that it is acceptable to retrofit and remodel the building up to 50% of its value.

This assessment starts with the California Administrative Code which provides guidance on essential services facilities. Section 4-206 of the California Administrative Code states:

> *4-206 Approval of new essential services buildings*
>
> *Plans and specifications shall be submitted to the appropriate enforcement agency for every new owned or leased essential services building before the plans are adopted by*

the governing board, authority, owner, corporation or other agency proposing to construct any essential services building.

Before any agency may convert an existing building into an essential services building, that agency shall submit plans and specifications for the alteration of the building to the appropriate enforcement agency for approval. The plans shall provide for the alterations necessary for compliance with the requirements of these rules and regulations.

Authority: Health and Safety Code Section 16022.

Reference: Health and Safety Code Section 16011.

From this section, we refer to the Health and Safety Code:

2016 California Code
Health and Safety Code - HSC
DIVISION 12.5 - BUILDINGS USED BY THE PUBLIC
CHAPTER 2 - Essential Services Buildings
ARTICLE 3 - General Requirements and Administration
Section 16014.

Universal Citation: CA Health & Safety Code § 16014 (2016)

*16014. (a) Except as otherwise provided in subdivision (b), drawings and specifications submitted pursuant to this chapter for construction, reconstruction, remodeling, additions, or alterations which affect structural elements of structures in existence on January 1, 1986, shall be based upon an assessment of the geological conditions of the site and the potential for earthquake damage, relying upon geologic and engineering investigations and studies by personnel who are competent to report on geologic conditions and their potential for causing earthquake damage. One-story Type V and Type II N construction of 4,000 square feet or less shall be exempt from this section, unless the project is within a special studies zone established **pursuant to Section 2622 of the Public Resources Code**.*

(b) The requirements of subdivision (a) may be waived by the enforcement agency if it determines that these requirements for the proposed essential services building project are unnecessary and would not be beneficial to the safety of the public.

This section leads to the Alquist-Priolo Earthquake Zone Act which provides guidance on construction located within an earthquake fault zone. The purpose of this act is to restrict development across fault traces. An excerpt reads:

2621.5. (a) It is the purpose of this chapter to provide for the adoption and administration of zoning laws, ordinances, rules, and regulations by cities and counties in implementation of the general plan that is in effect in any city or county. The Legislature declares that this chapter is intended to provide policies and criteria to assist cities, counties, and state agencies in the exercise of their responsibility to ***prohibit the location of developments and structures for human occupancy across the trace of active faults****. Further, it is the intent of this chapter to provide the citizens of the state with increased safety and to minimize the loss of life during and immediately following earthquakes by facilitating seismic retrofitting to strengthen buildings, including historical buildings, against ground shaking.*

An exception in section 2621.7 act allows for alterations or additions which do not exceed 50 percent of the value of the structure. Because of this exception we believe that a retrofit or alteration up to 50% value of the structure may be performed without further restrictions of the Alquist-Priolo Act. This exception can also be interpreted to allow for an addition up to 50% of the value of the structure, however the updated Geotechnical Study performed by Rockridge Geotechnical recommends against new construction beyond the existing footprint of the building. The Alquist-Priolo Act also allows for an exception for seismic retrofits of certain types of buildings but the current building does not meet the requirements for any of these building types.

9 Tier 1 Deficiencies

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

9.1 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.

9.2 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.

9.3 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.

9.4 Diaphragm Continuity

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

9.5 Steel Moment Frames with Flexible Diaphragms: Drift

The Tier 1 check evaluates the drift ratio of the steel moment frame using the Quick Check procedure. The steel moment frame at the apparatus bay installed as part of the 1998 Renovation does not satisfy the Tier 1 quick check for drift.

9.6 Steel Moment Frames with Flexible Diaphragms: Flexural Stress

The Tier 1 check evaluates the average flexural stress in the moment frame columns and beams. The steel moment frame beams and columns do not satisfy the Tier 1 quick check for flexural stress.

9.7 Steel Moment Frames with Flexible Diaphragms: Transfer to Steel Frames Connections

This Tier 1 check evaluates the capacity of the diaphragm connections to transfer loads to the moment frame.

9.8 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.

10 Tier 2 Analysis

10.1 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.

10.2 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.

10.3 Diaphragm Continuity

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

10.4 Moment Frame Drift and Beam Strength

The moment frames were not compliant for the expected strength of moment frame beams and drift limits.

11 Mitigation

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

11.1 Vertical Irregularity

Strengthen diaphragm with plywood diaphragm nailing.

11.2 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.

11.3 Shear Stress in Wood Shear Walls

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

11.4 Holdown and Posts to Foundation

Add holdown and posts to end of shear to upper level shear wall to address discontinuity of shear wall tension and compression forces.

11.5 Install Additional Anchor Bolts

Install additional anchor bolts to strengthen connection of moment frame to foundation.

11.6 Strengthen Moment Frame Beams

Strengthen moment frame beams by adding steel to build up beam section. This will also reduce drift. Column strengthening may also be required to address drift exceedance. Another option is to install a new pre-fabricated moment frame similar to a Simpson Strong-Frame within the existing moment frames. The advantage of a Strong-Frame is the replaceable fuse which can be replaced if ductile yielding occurs.

12 Conclusions

The purpose of this study is to identify the deficiencies of the existing building and determine the feasibility of renovation or addition to the existing building.

It is our opinion that it is acceptable to retrofit and remodel the building up to 50% of its value. It is not allowed to construct a new public safety building or new structure across a fault, nor is it allowed to increase its occupancy. Per the geotechnical recommendations, new construction is not allowed within a 50 foot setback of the suspected fault feature along the eastern property line. This severely limits potential new construction on the eastern portion of the building.

Thank you for the opportunity to be of service. Please call with any questions.

IDA Structural Engineers, Inc.

Jason M. Lee, SE
Associate

Figure 1: Aerial View

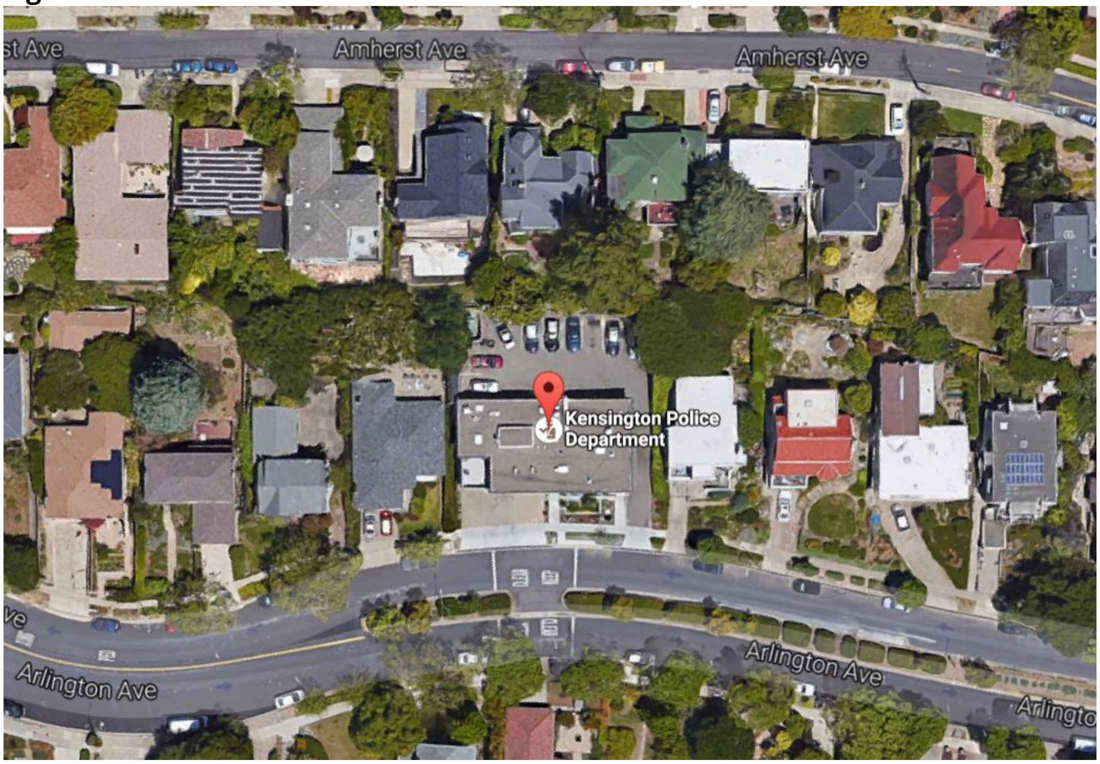


Figure 2: View from the North

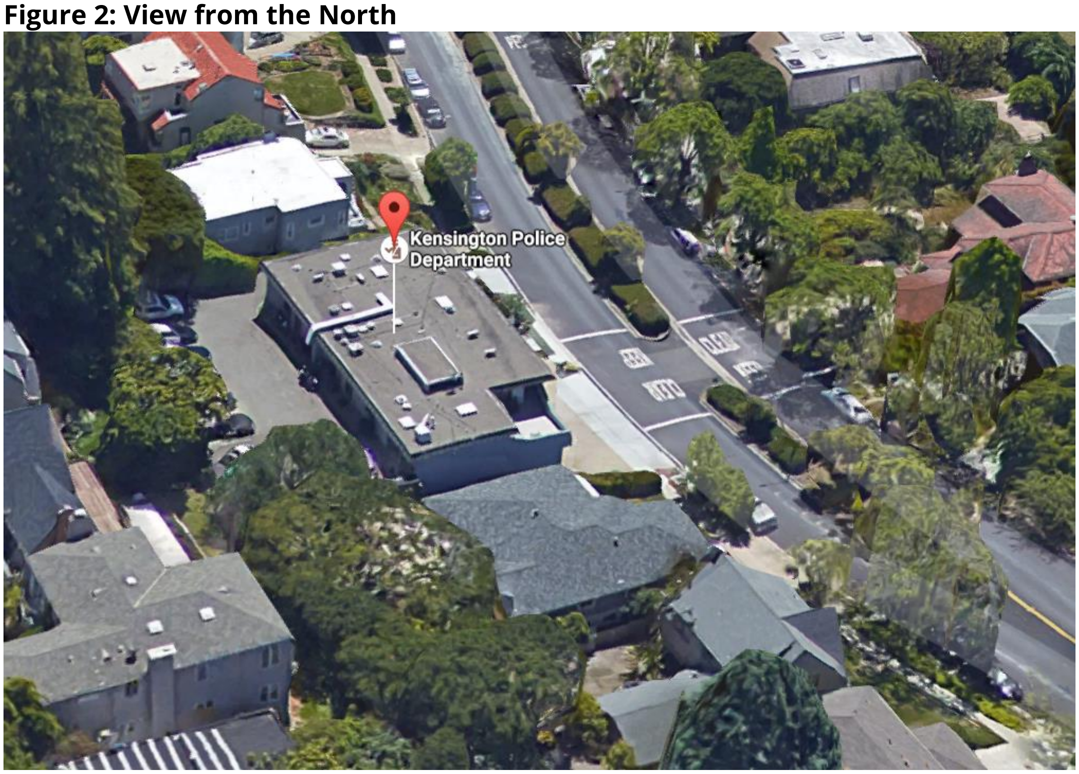


Figure 3: View from the South

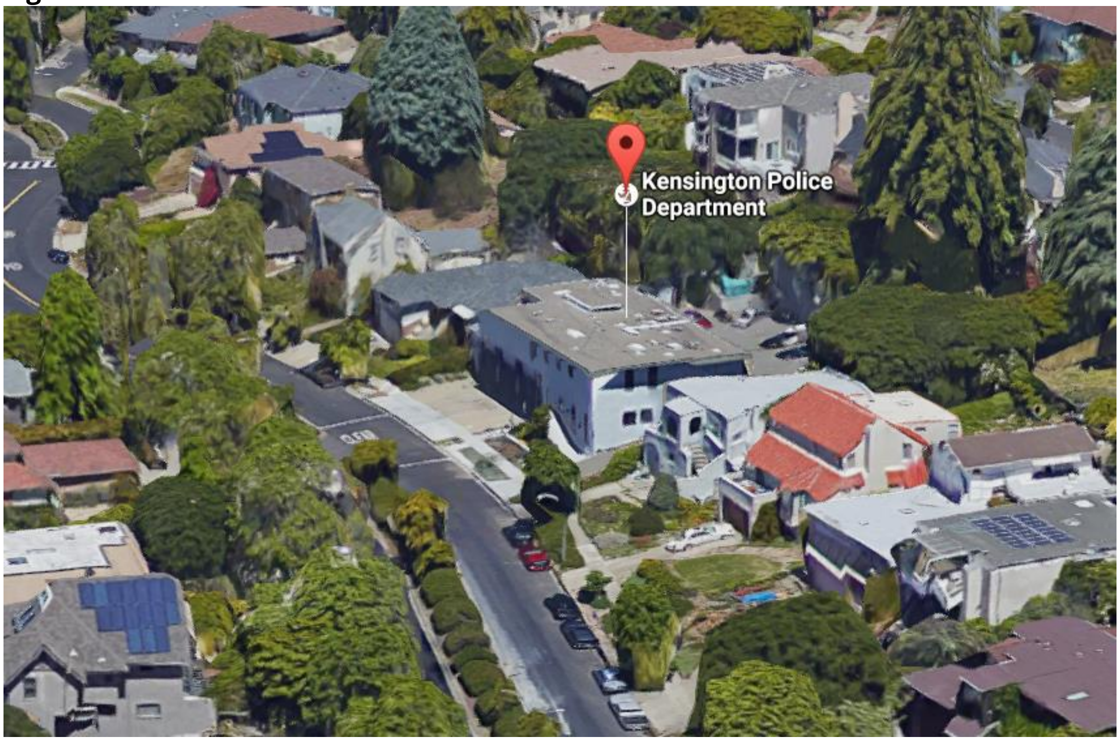


Figure 4: View from the West

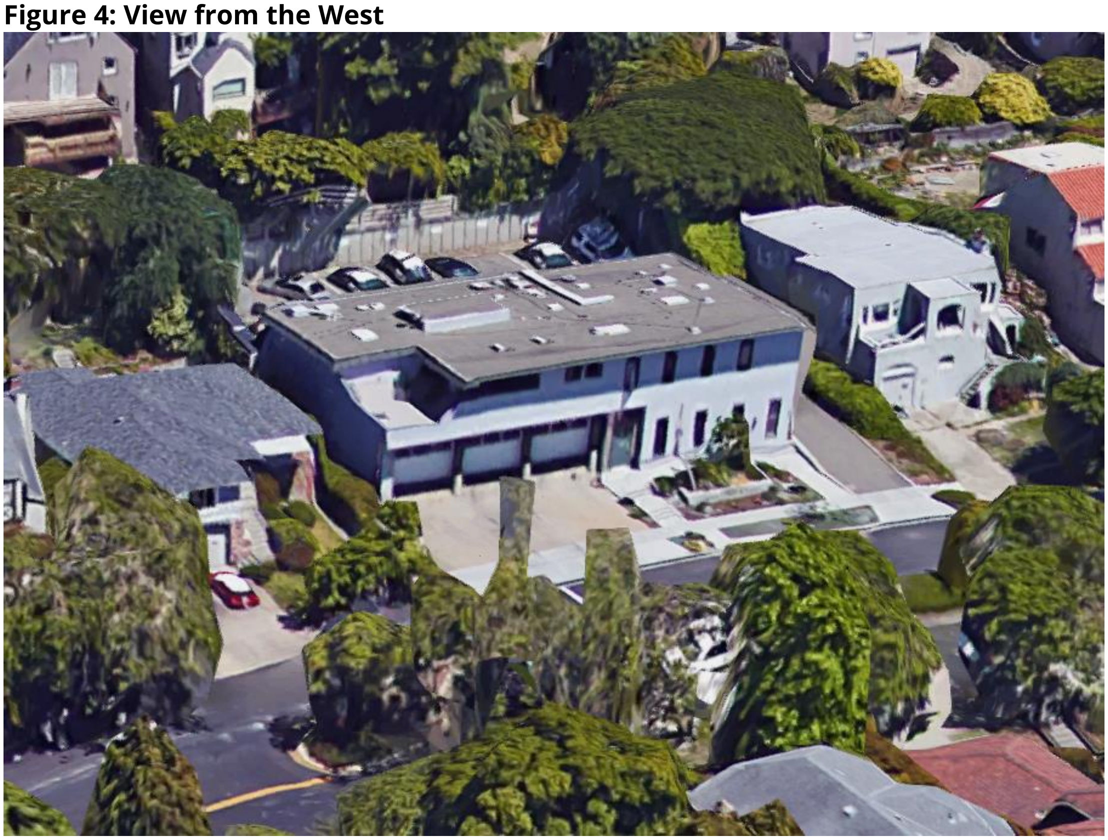
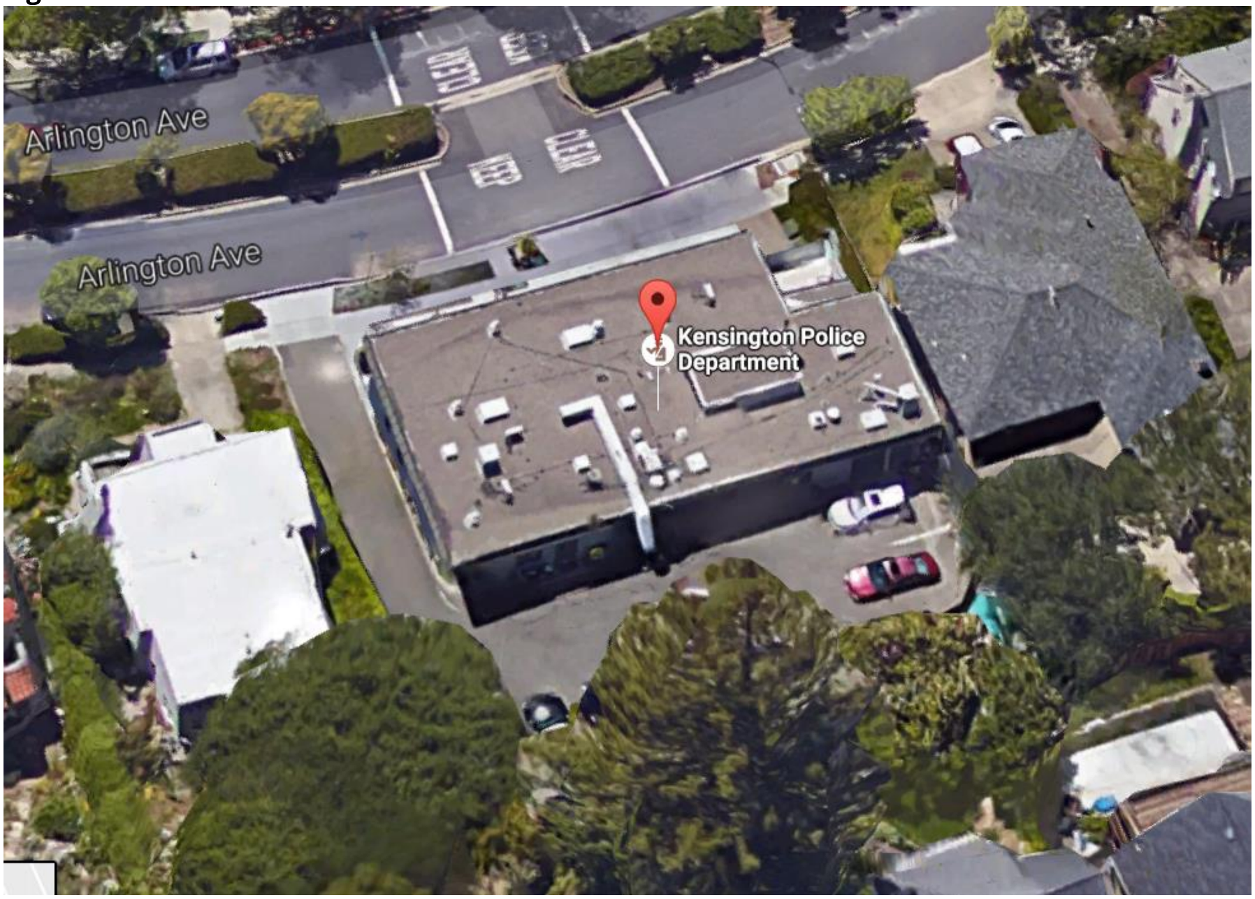
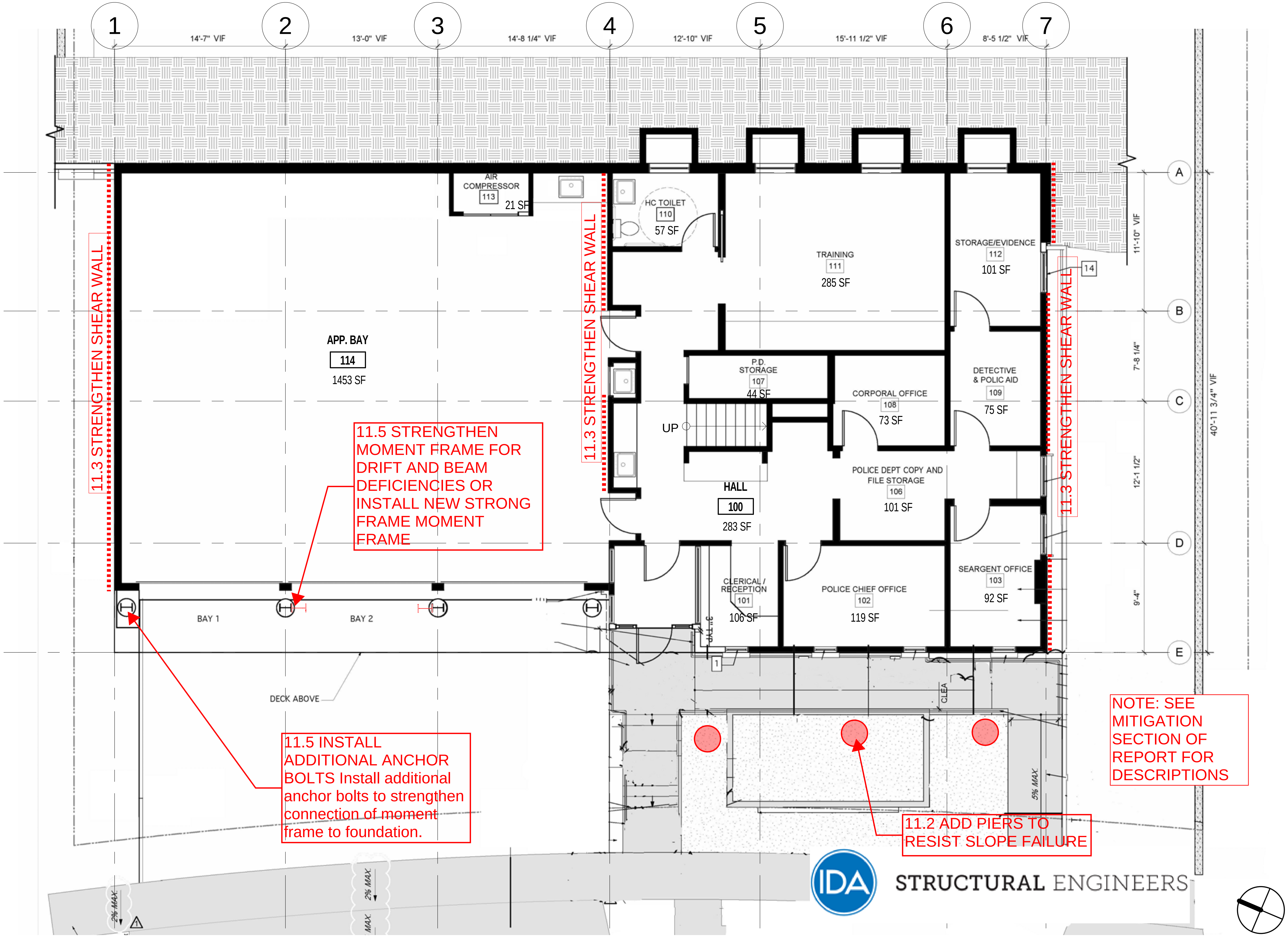


Figure 5: View from the East



APPENDIX A : SCHEMATIC MITIGATION PLAN



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

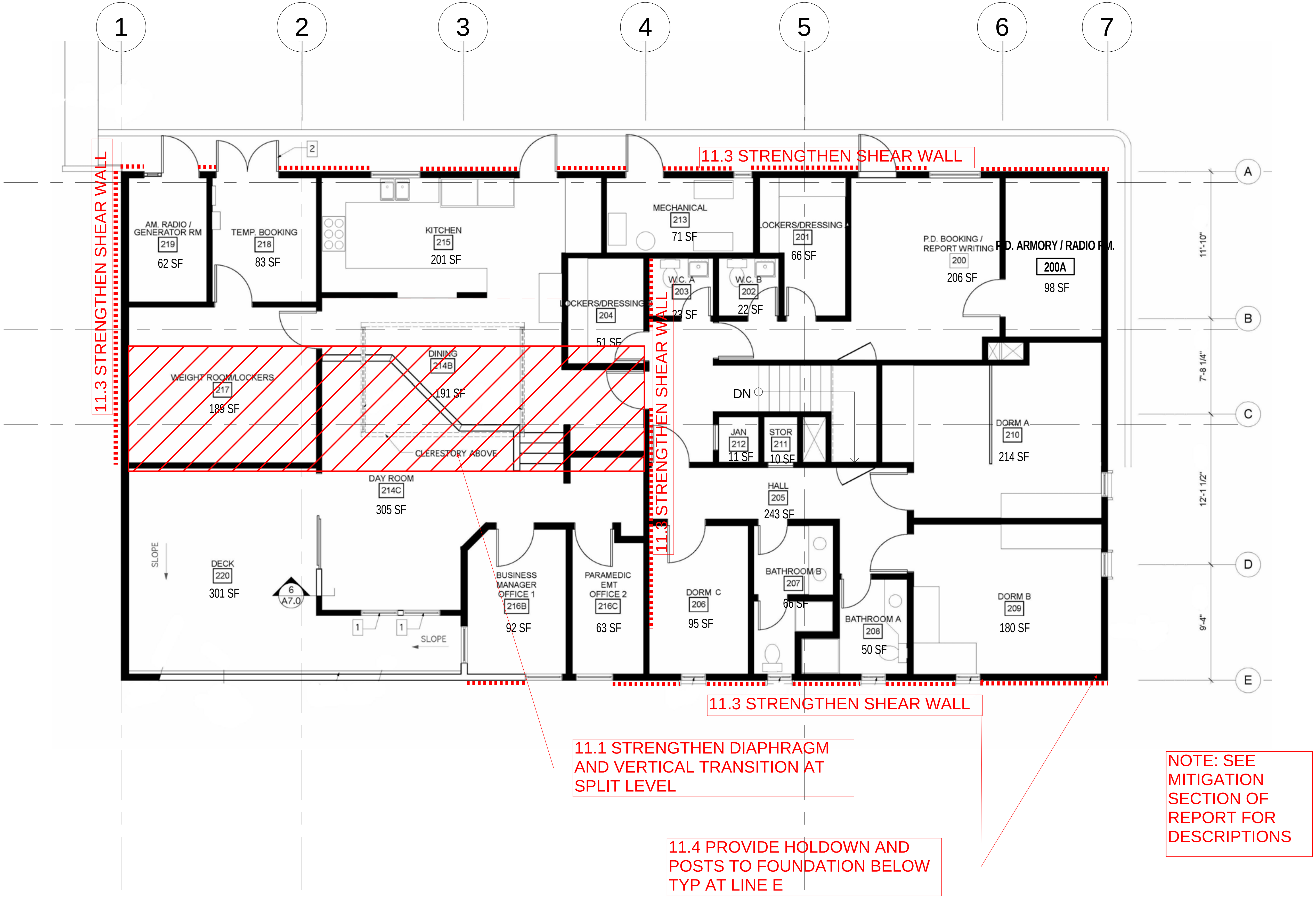
STRUCTURAL MITIGATION

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

Date: 2016/07/07

S1

Drawing No.



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

STRUCTURAL ENGINEERS

Preliminary design. Not for bidding or construction purposes.

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KENSINGTON FIRE STATION
KENSINGTON, CA

Sheet Title
EXISTING FLOOR PLAN - LEVEL 2
STRUCTURAL MITIGATION

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

Date: 2016/07/07

Drawing No.
S2

APPENDIX B :
TIER 1 CHECKLISTS

APPENDIX C
SUMMARY DATA SHEET

BUILDING DATA

Building Name: Kensington Public Safety Building   Date: 9/5/19

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 [ft^2 (m^2)]: 5800   Length [ft (m)]: 79' 2"   Width [ft (m)]: 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} = S_{D1} =

Soil Factors: Class = D F_a = 1.2 F_v = 1.5

BSE-1E Spectral Response Accelerations: S_{XS} = 1.13 S_{X1} = 0.491

Level of Seismicity: High   Performance Level: Immediate Occupancy

Building Period: T = 0.207 s

Spectral Acceleration: S_a = 0.942 s

Modification Factor: $C_m C_1 C_2$ = 1.1   Building Weight: W = 211 k

Pseudolateral Force: $V = C_m C_1 C_2 S_a W$ = 219 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.

Table 17-6 *(Continued)*. Collapse Prevention Structural Checklist for Building Type W2

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
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 (12.2 m) and have aspect ratios less than or equal to 4-to-1.	5.6.2	A.4.2.3
C NC N/A U	OTHER DIAPHRAGMS: The diaphragms do not consist of a system other than wood, metal deck, concrete, or horizontal bracing.	5.6.5	A.4.7.1

Note: C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.

Table 17-7. Immediate Occupancy Checklist for Building Type W2

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
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.	5.5.1.1	A.3.2.1.1
C NC N/A U	SHEAR STRESS CHECK: The shear stress in the shear walls, calculated using the Quick Check procedure of Section 4.4.3.3, is less than the following values: Structural panel sheathing 1,000 lb/ft (14.6 kN/m) Diagonal sheathing 700 lb/ft (10.2 kN/m) Straight sheathing 100 lb/ft (1.5 kN/m) All other conditions 100 lb/ft (1.5 kN/m)	5.5.3.1.1	A.3.2.7.1
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.	5.5.3.6.1	A.3.2.7.2
C NC N/A U	GYPSUM WALLBOARD OR PLASTER SHEAR WALLS: Interior plaster or gypsum wallboard is not used for shear walls on buildings more than one story high with the exception of the uppermost level of a multi-story building.	5.5.3.6.1	A.3.2.7.3
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.	5.5.3.6.1	A.3.2.7.4
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.	5.5.3.6.2	A.3.2.7.5
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.	5.5.3.6.3	A.3.2.7.6
C NC N/A U	CRIPPLE WALLS: Cripple walls below first-floor-level shear walls are braced to the foundation with wood structural panels.	5.5.3.6.4	A.3.2.7.7
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.	5.5.3.6.5	A.3.2.7.8
C NC N/A U	HOLD-DOWN ANCHORS: All shear walls have hold-down anchors attached to the end studs constructed in accordance with acceptable construction practices.	5.5.3.6.6	A.3.2.7.9
Connections			
C NC N/A U	WOOD POSTS: There is a positive connection of wood posts to the foundation.	5.7.3.3	A.5.3.3
C NC N/A U	WOOD SILLS: All wood sills are bolted to the foundation.	5.7.3.3	A.5.3.4
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.	5.7.4.1	A.5.4.1

continues

Table 17-7 *(Continued)*. Immediate Occupancy Checklist for Building Type W2

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
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.		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.		A.6.2.4
Low, Moderate, and High Seismicity (Complete the Following Items in Addition to the Items for Very Low Seismicity)			
Seismic-Force-Resisting System			
C NC N/A U	NARROW WOOD SHEAR WALLS: Narrow wood shear walls with an aspect ratio greater than 1.5-to-1 are not used to resist seismic forces.	5.5.3.6.1	A.3.2.7.4
Diaphragms			
C NC N/A U	DIAPHRAGM CONTINUITY: The diaphragms are not composed of split-level floors and do not have expansion joints.	5.6.1.1	A.4.1.1
C NC N/A U	ROOF CHORD CONTINUITY: All chord elements are continuous, regardless of changes in roof elevation.	5.6.1.1	A.4.1.3
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.	5.6.1.5	A.4.1.8
C NC N/A U	STRAIGHT SHEATHING: All straight-sheathed diaphragms have aspect ratios less than 1-to-1 in the direction being considered.	5.6.2	A.4.2.1
C NC N/A U	SPANS: All wood diaphragms with spans greater than 12 ft (3.6 m) consist of wood structural panels or diagonal sheathing.	5.6.2	A.4.2.2
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 (9.2 m) and have aspect ratios less than or equal to 3-to-1.	5.6.2	A.4.2.3
C NC N/A U	OTHER DIAPHRAGMS: The diaphragms do not consist of a system other than wood, metal deck, concrete, or horizontal bracing.	5.6.5	A.4.7.1
Connections			
C NC N/A U	WOOD SILL BOLTS: Sill bolts are spaced at 4 ft or less with acceptable edge and end distance provided for wood and concrete.	5.7.3.3	A.5.3.7

Note: C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.

professional to require further investigation shall be categorized as Noncompliant or Unknown. For evaluation statements classified as Noncompliant or Unknown, the design professional is permitted to choose to conduct further investigation using the corresponding Tier 2 evaluation procedure listed next to each evaluation statement.

17.4 STRUCTURAL CHECKLISTS FOR BUILDING TYPES S1: STEEL MOMENT FRAMES WITH STIFF DIAPHRAGMS AND S1A: STEEL MOMENT FRAMES WITH FLEXIBLE DIAPHRAGMS

For building systems and configurations that comply with the S1 or S1a building type description in Table 3-1, the Collapse Prevention Structural Checklist in Table 17-8 shall be completed where required by Table 4-6 for Collapse Prevention Structural Performance, and the Immediate Occupancy Structural Checklist in Table 17-9 shall be completed where required by Table 4-6 for Immediate Occupancy Structural Performance. Tier 1 screening shall include on-site investigation and condition assessment as required by Section 4.2.1.

Where applicable, each of the evaluation statements listed in this checklist shall be marked Compliant (C), Noncompliant (NC), Not Applicable (N/A), or Unknown (U) for a Tier 1 screening. Items that are deemed acceptable to the design professional in accordance with the evaluation statement shall be categorized as Compliant, whereas items that are determined by the design professional to require further investigation shall be categorized as Noncompliant or Unknown. For evaluation statements classified as Noncompliant or Unknown, the design professional is permitted to choose to conduct further investigation using the corresponding Tier 2 evaluation procedure listed next to each evaluation statement.

17.5 STRUCTURAL CHECKLIST FOR BUILDING TYPES S2: STEEL BRACED FRAMES WITH STIFF DIAPHRAGMS AND S2A: STEEL BRACED FRAMES WITH FLEXIBLE DIAPHRAGMS

For building systems and configurations that comply with the S2 or S2a building type description in Table 3-1, the Collapse Prevention Structural Checklist in Table 17-10 shall be completed where required by Table 4-6 for Collapse Prevention Structural Performance, and the Immediate Occupancy Structural Checklist in Table 17-11 shall be completed where required by Table 4-6 for Immediate Occupancy Structural Performance.

Table 17-8. Collapse Prevention Structural Checklist for Building Types S1 and S1a

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
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. Three bays	5.5.1.1	A.3.1.1.1
C NC N/A U	DRIFT CHECK: The drift ratio of the steel moment frames, calculated using the Quick Check procedure of Section 4.4.3.1, is less than 0.030.	5.5.2.1.2	A.3.1.3.1
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 4.4.3.6, is less than $0.30F_y$.	5.5.2.1.3	A.3.1.3.2
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 4.4.3.9, is less than F_y. Columns need not be checked if the strong column–weak beam checklist item is compliant.	5.5.2.1.2	A.3.1.3.3
Connections			
C NC N/A U	TRANSFER TO STEEL FRAMES: Diaphragms are connected for transfer of seismic forces to the steel frames.	5.7.2	A.5.2.2
C NC N/A U	STEEL COLUMNS: The columns in seismic-force-resisting frames are anchored to the building foundation.	5.7.3.1	A.5.3.1
Moderate Seismicity (Complete the Following Items in Addition to the Items for Low Seismicity)			
Seismic-Force-Resisting System			
C NC N/A U	REDUNDANCY: The number of bays of moment frames in each line is greater than or equal to 2.	5.5.1.1	A.3.1.1.1
C NC N/A U	INTERFERING WALLS: All concrete and masonry infill walls placed in moment frames are isolated from structural elements.	5.5.2.1.1	A.3.1.2.1
C NC N/A U	MOMENT-RESISTING CONNECTIONS: All moment connections can develop the strength of the adjoining members based on the specified minimum yield stress of steel.	5.5.2.2.1	A.3.1.3.4
High Seismicity (Complete the Following Items in Addition to the Items for 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 in accordance with AISC 341, Section A3.2.	5.5.2.2.1	A.3.1.3.4
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.	5.5.2.2.2	A.3.1.3.5
C NC N/A U	COLUMN SPLICES: All column splice details located in moment-resisting frames include connection of both flanges and the web.	5.5.2.2.3	A.3.1.3.6
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 frames is greater than 50%.	5.5.2.1.5	A.3.1.3.7
C NC N/A U	COMPACT MEMBERS: All frame elements meet section requirements in accordance with AISC 341, Table D1.1, for moderately ductile members.	5.5.2.2.4	A.3.1.3.8
Diaphragms (Stiff or Flexible)			
C NC N/A U	OPENINGS AT FRAMES: Diaphragm openings immediately adjacent to the moment frames extend less than 25% of the total frame length.	5.6.1.3	A.4.1.5
Flexible Diaphragms			
C NC N/A U	CROSS TIES: There are continuous cross ties between diaphragm chords.	5.6.1.2	A.4.1.2
C NC N/A U	STRAIGHT SHEATHING: All straight-sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered.	5.6.2	A.4.2.1
C NC N/A U	SPANS: All wood diaphragms with spans greater than 24 ft (7.3 m) consist of wood structural panels or diagonal sheathing.	5.6.2	A.4.2.2

continues

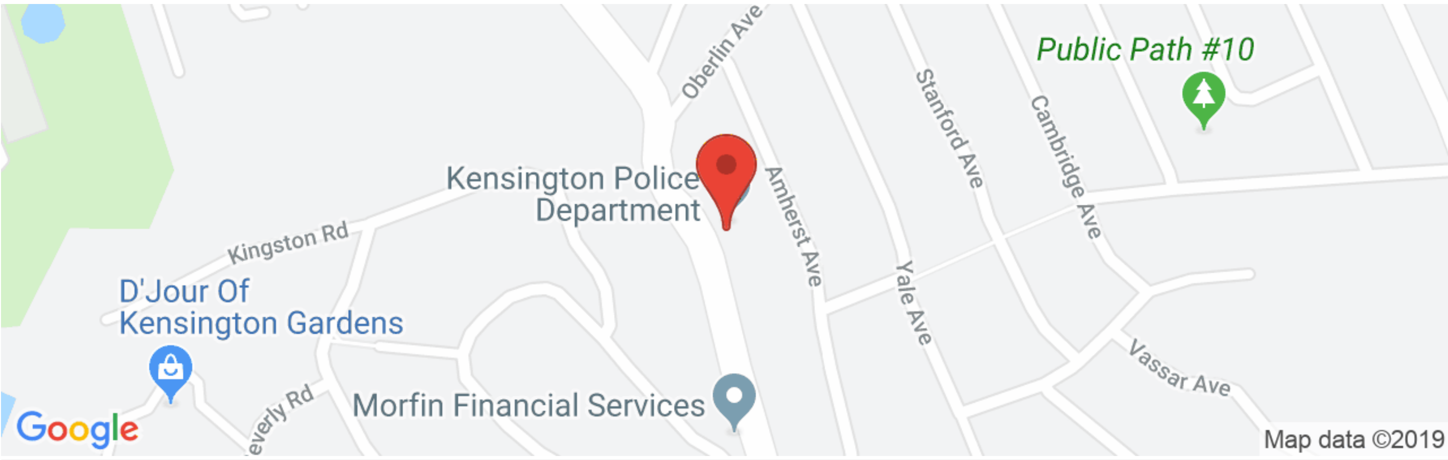
STRUCTURAL ENGINEERS

APPENDIX C :
TIER 2 EVALUATIONS

Kensington Public Safety Building

217 Arlington Ave, Kensington, CA 94707, USA

Latitude, Longitude: 37.9062238, -122.27874710000003



Date	9/5/2019, 4:01:27 PM
Design Code Reference Document	ASCE41-17
Custom Probability	
Site Class	C - Very Dense Soil and Soft Rock

Type	Description	Value
Hazard Level		BSE-2N
S_S	spectral response (0.2 s)	2.277
S_1	spectral response (1.0 s)	0.88
S_{XS}	site-modified spectral response (0.2 s)	2.733
S_{X1}	site-modified spectral response (1.0 s)	1.232
F_a	site amplification factor (0.2 s)	1.2
F_v	site amplification factor (1.0 s)	1.4
ssuh	max direction uniform hazard (0.2 s)	2.886
crs	coefficient of risk (0.2 s)	0.9
ssrt	risk-targeted hazard (0.2 s)	2.598
ssd	deterministic hazard (0.2 s)	2.277
s1uh	max direction uniform hazard (1.0 s)	1.104
cr1	coefficient of risk (1.0 s)	0.891
s1rt	risk-targeted hazard (1.0 s)	0.984
s1d	deterministic hazard (1.0 s)	0.88

Type	Description	Value
Hazard Level		BSE-1N
S_{XS}	site-modified spectral response (0.2 s)	1.822
S_{X1}	site-modified spectral response (1.0 s)	0.821

Type	Description	Value
Hazard Level		BSE-2E
S_S	spectral response (0.2 s)	1.997
S_1	spectral response (1.0 s)	0.74
S_{XS}	site-modified spectral response (0.2 s)	2.396
S_{X1}	site-modified spectral response (1.0 s)	1.035
f_a	site amplification factor (0.2 s)	1.2
f_v	site amplification factor (1.0 s)	1.4

Type	Description	Value
Hazard Level		BSE-1E
S_S	spectral response (0.2 s)	0.942
S_1	spectral response (1.0 s)	0.327
S_{XS}	site-modified spectral response (0.2 s)	1.13
S_{X1}	site-modified spectral response (1.0 s)	0.491
F_a	site amplification factor (0.2 s)	1.2
F_v	site amplification factor (1.0 s)	1.5

Type	Description	Value
Hazard Level		T-Sub-L Data
T-Sub-L	Long-period transition period in seconds	8

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 4.4.2.3
- W, the total dead load
- BSE-1E at Immediate Occupancy

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 — $S_{X1} := 0.491$

Short period spectral acceleration of the BSE-1E Design — $S_{XS} := .942$

Factor per table 4-8 — $M_s := 1.5$ — Immediate Occupancy Level of Performance

Determine T

Coefficient to determine building period, from Section 4.4.2.4 — $C_t := 0.020$

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 4.4.2.4 — $T := C_t \cdot \left(\dfrac{h_n}{1\ ft}\right)^{\beta} = 0.207$

Minimum base dimension $base := 40.66\ ft$

$S_a := min\left(\frac{S_{X1}}{T}, S_{XS}\right) = 0.942$

$0.6 \cdot S_a = 0.57$ $\frac{base}{h_n} = 1.81$

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 4.4.3.3

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 :

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

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

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

$SeismicWeight_{2ndFlr} = 139.24$ ***kip***

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

$SeismicWeight_{Roof} = 71.04$ ***kip***

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

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

Total seismic weight of structure

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

Psuedo seismic force per 4.4.2.1 Eq. 4-1

$$V := C \cdot S_a \cdot W = 219 \textbf{ kip}$$

Factor per 4.4.2.2

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

$$k = 1$$

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

Vertical distribution of psuedo seismic force per 4.4.2.2 Eq 4-3a

$$F_x := \frac{w_x \cdot h_x^{\,k}}{\sum_{i=1}^{Floors} w_i \cdot h_i^{\,k}} \cdot V = \begin{bmatrix} 111 \\ 108 \end{bmatrix} \textbf{kip}$$

Story shear at story level j

$$V_j := \sum_{x=j}^{Floors} F_x = \begin{bmatrix} 219 \\ 108 \end{bmatrix} \mathbf{kip}$$

Shear stress in shear walls in north sourth direction

$$v_{NS_j} := \frac{1}{M_s} \cdot \frac{V_j}{A_{wNS_j} \cdot \frac{1}{\mathbf{ft}}} = \begin{bmatrix} 1388.18 \\ 1397.12 \end{bmatrix} \mathbf{plf}$$

Shear stress in shear walls in east west direction

$$v_{EW_j} := \frac{1}{M_s} \cdot \frac{V_j}{A_{wEW_j} \cdot \frac{1}{\mathbf{ft}}} = \begin{bmatrix} 1722.31 \\ 953.68 \end{bmatrix} \mathbf{plf}$$

ASCE 41 Quick check limit:

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

The shear stress in the shear walls,calculated using the Quick Check procedure of Section 4.4.3.3 is less than than allowable value of 1000plf

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

$S_a = 0.94$

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

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

$$V = 218.64 \ \textbf{kip}$$

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

$$F_x := \frac{w_x \cdot h_x^{\ k}}{\sum_{i=1}^{Floors} w_i \cdot h_i^{\ k}} \cdot V = \begin{bmatrix} 111 \\ 108 \end{bmatrix} \textbf{kip}$$

Story shear at story level j

$$V_j := \sum_{x=j}^{Floors} F_x = \begin{bmatrix} 219 \\ 108 \end{bmatrix} \textbf{kip}$$

Diaphragm inertial force

$$F_{p_x} := \frac{\sum_{i=1}^{Floors} F_i}{\sum_{i=1}^{Floors} w_i} \cdot w_x = \begin{bmatrix} 145 \\ 74 \end{bmatrix} \textbf{kip}$$

Calculation of available Shear Wall Length

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

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

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

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

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

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

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

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

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

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

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

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

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

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 :

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

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

Shear to Walls along grid E:

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

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

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 :

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

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

Shear to Walls along grid 7 :

$$R_{7\_Roof} := \frac{37\ ft \cdot 0.5}{79\ ft} \cdot V_x = 33.02\ kip$$

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

Second Floor

North South Direction

$V_x := 286\ kip$ Input Story Shear

Shear to Walls along grid A :

$$R_{A\_2ndFlr} := \frac{V_x}{2} = 143\ kip$$

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

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 :

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

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

Shear to Walls along grid 7 :

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

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

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:

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

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

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

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

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

Shear wall capacity along Grid 7:

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

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

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

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

$V_c := \frac{286}{2}\ kip$ Shear in the column (kip). The column shear forces are calculated using the story forces in accordance with Section 4.5.2.2

$V_c = 143\ kip$

$k_b := \frac{I_b}{L}$ for the representative beam

$k_c := \frac{I_c}{h}$ for the representative column

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

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

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

$\dfrac{Q_{ud\_beam}}{Q_{CE\_beam} \cdot m \cdot k} = 0.64$ Non Compliant

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

$\dfrac{Q_{ud\_col}}{m \cdot Q_{CE\_col}} = 0.4$ 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 is not adequate. The moment frame doesn't comply the drift check.

COLUMN AXIAL STRESS CHECK USING QUICK CHECK PROCEDURE

$f_y := 36\ ksi$ A36 steel

$0.30\ f_y = 10.8\ ksi$

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

$V := 143\ kip$ Pseudo Seismic force

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

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

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

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

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

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

FLEXURAL STRESS CHECK USING QUICK CHECK PROCEDURE OF SECTION 4.4.3.9:

$n_c := 4$ 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

$V_j := 143\ \mathbf{kip}$ Story shear computed in accordance with Section 4.4.2.2

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

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

$Z_b := 6 \cdot 114\ \mathbf{in}^3 = 684\ \mathbf{in}^3$ 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

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

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

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

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

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 :

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

$A_w := t_w \cdot (d - 2 \cdot t_f) = 3.24\ in^2$

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

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

Strength of beam web-to-column connection weld :

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

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

$m := 1.0$

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

Demand Capacity Ratio

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

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

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 4.4.2.3
- W, the total dead load
- BSE-2E at Life Safety

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-2E	$S_{X1} := 1.035$	
Short period spectral acceleration of the BSE-1E Design	$S_{XS} := 2.396$	
Factor per table 4-8	$M_s := 3.0$	Life Safety Occupancy Level of Performance

Determine T

Coefficient to determine building period, from Section 4.4.2.4	$C_t := 0.020$
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 4.4.2.4

$$T := C_t \cdot \left(\frac{h_n}{1\ ft}\right)^{\beta} = 0.207$$

Minimum base dimension $base := 40.66\ ft$

$$S_a := min\left(\frac{S_{X1}}{T}, S_{XS}\right) = 2.396$$

$0.6 \cdot S_a = 1.44$ $\frac{base}{h_n} = 1.81$

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 4.4.3.3

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 :

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

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

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

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

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

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

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 4.4.2.1 Eq. 4-1

$$V := C \cdot S_a \cdot W = 556\ \mathbf{kip}$$

Factor per 4.4.2.2

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

$$k = 1$$

$x := 1 .. Floors$

$j := 1 .. Floors$

Vertical distribution of psuedo seismic force per 4.4.2.2 Eq 4-3a

$$F_x := \frac{w_x \cdot h_x{}^k}{\sum_{i=1}^{Floors} w_i \cdot h_i{}^k} \cdot V = \begin{bmatrix} 282 \\ 274 \end{bmatrix} \mathbf{kip}$$

Story shear at story level j $$V_j := \sum_{x=j}^{Floors} F_x = \begin{bmatrix} 556 \\ 274 \end{bmatrix} \textbf{kip}$$

Shear stress in shear walls in north sourth direction $$v_{NS_j} := \frac{1}{M_s} \cdot \frac{V_j}{A_{wNS_j} \cdot \frac{1}{\textbf{ft}}} = \begin{bmatrix} 1765.43 \\ 1776.8 \end{bmatrix} \textbf{plf}$$

Shear stress in shear walls in east west direction $$v_{EW_j} := \frac{1}{M_s} \cdot \frac{V_j}{A_{wEW_j} \cdot \frac{1}{\textbf{ft}}} = \begin{bmatrix} 2190.36 \\ 1212.85 \end{bmatrix} \textbf{plf}$$

ASCE 41 Quick check limit:

$ShearstressAllowable := 1000\ \textbf{plf}$

The shear stress in the shear walls,calculated using the Quick Check procedure of Section 4.4.3.3 is less than than allowable value of 1000plf

Tier 2 analysis of shear wall

Total seismic weight of the building $W = 211\ \textbf{kip}$

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

$C_1 C_2 := 1.4$ Table 7-3

$C_m := 1.0$ Table 7-4

$S_a = 2.4$

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

$$V := C_1 C_2 \cdot C_m \cdot S_a \cdot W$$

$$V = 707.78\ \textbf{kip}$$

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

$$F_x := \frac{w_x \cdot h_x^{\ k}}{\sum_{i=1}^{Floors} w_i \cdot h_i^{\ k}} \cdot V = \begin{bmatrix} 359 \\ 349 \end{bmatrix} \textbf{kip}$$

Story shear at story level j

$$V_j := \sum_{x=j}^{Floors} F_x = \begin{bmatrix} 708 \\ 349 \end{bmatrix} \textbf{kip}$$

Diaphragm inertial force

$$F_{p_x} := \frac{\sum_{i=1}^{Floors} F_i}{\sum_{i=1}^{Floors} w_i} \cdot w_x = \begin{bmatrix} 470 \\ 238 \end{bmatrix} \; \mathbf{kip}$$

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 :

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

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

Shear to Walls along grid E:

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

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

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 :

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

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

Shear to Walls along grid 7 :

$$R_{7\_Roof} := \frac{37\ ft \cdot 0.5}{79\ ft} \cdot V_x = 33.02\ kip$$

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

Second Floor

North South Direction

$V_x := 286\ kip$ Input Story Shear

Shear to Walls along grid A :

$$R_{A\_2ndFlr} := \frac{V_x}{2} = 143\ kip$$

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

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

$m := 3.8$ 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

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

(10d nails @ 6" oc edge nailing)

$m \cdot k \cdot Q_{CE\_AWall} = 2616.3\ \mathbf{plf}$

$m \cdot k \cdot Q_{CE\_AWall} \cdot L_{Second\_GridA} = 119.3\ \mathbf{kip}$

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

$$\frac{Q_{ad}}{m \cdot k \cdot Q_{CE\_AWall} \cdot L_{Second\_GridA}} = 0.59$$ NG

Shear wall capacity along Grid E

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

$m \cdot k \cdot Q_{CE\_AWall} = 2616.3 \ \textbf{plf}$

$m \cdot k \cdot Q_{CE\_AWall} \cdot L_{Second\_GridE} = 96.72 \ \textbf{kip}$

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

$\dfrac{Q_{ad}}{m \cdot k \cdot Q_{CE\_AWall} \cdot L_{Second\_GridE}} = 0.73$ NG

East West Direction

Shear wall capacity along Grid 1

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

$m \cdot k \cdot Q_{CE\_AWall} = 2616.3 \ \textbf{plf}$

$m \cdot k \cdot Q_{CE\_AWall} \cdot L_{Second\_Grid1} = 62.79 \ \textbf{kip}$

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

$\dfrac{Q_{ad}}{m \cdot k \cdot Q_{CE\_AWall} \cdot L_{Second\_Grid1}} = 0.6$ NG

Shear wall capacity along Grid 4:

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

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

$m \cdot k \cdot \left(Q_{CE\_AWall} + Q_{CE\_CWall}\right) \cdot 17 \ \textbf{ft} + m \cdot k \cdot Q_{CE\_AWall} \cdot 5.66 \ \textbf{ft} = 113.36 \ \textbf{kip}$

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

$\dfrac{Q_{ad}}{m \cdot k \cdot \left(Q_{CE\_AWall} + Q_{CE\_CWall}\right) \cdot 17 \ \textbf{ft} + m \cdot k \cdot Q_{CE\_AWall} \cdot 5.66 \ \textbf{ft}} = 0.65$ NG

Shear wall capacity along Grid 7:

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

$m \cdot k \cdot Q_{CE\_AWall} \cdot L_{Second\_Grid7} = 97.04\ \mathbf{kip}$

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

$\dfrac{Q_{ad}}{m \cdot k \cdot Q_{CE\_AWall} \cdot L_{Second\_Grid7}} = 0.34$ OK

<u>Second Floor</u>

<u>North South Direction</u>

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

<u>East West Direction</u>

Shear wall capacity along Grid 1

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

$m \cdot k \cdot Q_{CE\_AWall} = 2616.3\ \mathbf{plf}$

$m \cdot k \cdot Q_{CE\_AWall} \cdot L_{Ground\_Grid1} = 96.15\ \mathbf{kip}$

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

$$\frac{Q_{ad}}{m \cdot k \cdot Q_{CE\_AWall} \cdot L_{Ground\_Grid1}} = 0.79$$ NG

Shear wall capacity along Grid 4:

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

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

$$m \cdot k \cdot \left(Q_{CE\_BWall} + Q_{CE\_DWall}\right) \cdot 8 \ \boldsymbol{ft} + m \cdot k \cdot \left(Q_{CE\_DWall} \cdot 12 \ \boldsymbol{ft} + Q_{CE\_BWall} \cdot 18 \ \boldsymbol{ft}\right) = 245.83 \ \boldsymbol{kip}$$

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

$$\frac{Q_{ad}}{m \cdot k \cdot \left(Q_{CE\_BWall} + Q_{CE\_DWall}\right) \cdot 8 \ \boldsymbol{ft} + m \cdot k \cdot \left(Q_{CE\_DWall} \cdot 12 \ \boldsymbol{ft} + Q_{CE\_BWall} \cdot 18 \ \boldsymbol{ft}\right)} = 0.6$$ NG

Shear wall capacity along Grid 7:

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

$$m \cdot k \cdot Q_{CE\_AWall} \cdot L_{Ground\_Grid7} = 76.74 \ \boldsymbol{kip}$$

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

$$\frac{Q_{ad}}{m \cdot k \cdot Q_{CE\_AWall} \cdot L_{Ground\_Grid7}} = 0.87$$ NG

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

Life Safety Structural Checklist for Building Types S1A: Steel Moment Frames with Flexible Diaphragms

Section 4.4.3.1 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: $D_r := \frac{(k_b + k_c)}{k_b \cdot k_c} \cdot \frac{h}{12 \cdot E} \cdot V_c = 0.0576$

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

Tier 2 evaluation of Drift :

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

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

Expected Strength of Beams :

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

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

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

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

$\dfrac{Q_{ud\_beam}}{Q_{CE\_beam} \cdot m \cdot k} = 0.64$ Non Compliant

Expected Strength of Columns:

Note: Assuming negligible axial load on the columns

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

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

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

$m := 6.0$ Table 9-6, Columns-Flexure, IO

$\dfrac{Q_{ud\_col}}{m \cdot Q_{CE\_col}} = 0.13$ 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 is not adequate. The moment frame doesn't comply the drift check.

COLUMN AXIAL STRESS CHECK USING QUICK CHECK PROCEDURE

$f_y := 36\ ksi$ A36 steel

$0.30\ f_y = 10.8\ ksi$

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

$V := 143\ kip$ Pseudo Seismic force

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

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

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

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

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

$p_{ot} < 0.30\ f_y$ *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

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

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

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

$Z_b := 6 \cdot 114\ \mathbf{in}^3 = 684\ \mathbf{in}^3$ 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

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

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

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

$d_c := 12.2\ \mathbf{in}$ Column depth W12x58

$d_b := 12\ \mathbf{in}$ Depth of W12x40 beam

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

$E := 29000\ \mathbf{ksi}$ Modulus of elasticity

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

$t_p := \frac{1}{2}\ \mathbf{in} \cdot 2 + 0.36\ \mathbf{in}$ 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

$V_{CE} = 328.52\ \mathbf{kip}$

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

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

The plastic moment capacity of beam:

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

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

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

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 :

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

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

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

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

Strength of beam web-to-column connection weld :

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

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

$m := 1.0$

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

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)

$m := 2.5$ 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