ZFA STRUCTURAL ENGINEERS

Seismic Evaluation Report

Municipal Services Center North

55 Stony Point Road, Santa Rosa, CA, 95401

ZFA Project: 22406

November 2, 2022

Prepared For:

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TABLE OF CONTENTS

EXECUTIVE SUMMARY

The Administration Building at the Municipal Services Center North Building campus, located at 55 Stony Point Road in Santa Rosa, CA, has been reviewed for **Immediate Occupancy** performance level using the ASCE 41-17 Standard for Seismic Evaluation and Retrofit of Existing Buildings, Tier 1 and Tier 2 Evaluations. The building was reviewed using the original construction documents, structural Tier 1 checklists, and site visits. Non-structural elements were not included in the scope of this review. Items indicated as non-compliant by Tier 1 checklists were reviewed using Tier 2 evaluation procedures.

The review resulted in the following structural and geotechnical findings and recommendations for improvement in order of significance:

STRUCTURAL

- The tension braces utilized in the braced frames are not adequate to resist Tier 1 or Tier 2 calculated seismic demands. Since there are only two braces along each of the two lines, there is very little redundancy in the system and a failure would likely result in significant damage. New, larger braces are recommended at each frame. *Structural Priority: High*
- The columns utilized in the moment frames are not adequate to resist Tier 1 or Tier 2 calculated seismic demands. There is no redundancy in this system and a failure would likely result in significant damage. Strengthening of the structural steel columns is recommended. *Structural Priority: High*
- Adjacent lobby structure does not meet the minimum Tier 1 required clear separation to subject building for independent seismic performance. Additional analysis may be performed to estimate horizontal movement in a seismic event. Minor damage may occur due to pounding between structures during a seismic event; however, damage due to this condition is not anticipated to cause life safety structural concerns within the subject building. Further analysis of possible egress issues is recommended. *Structural Priority: Low*

GEOTECHNICAL

- The geotechnical report from the adjacent site indicates a low probability of liquefaction in the upper 21.5 feet of soil, however area liquefaction maps indicate the site is located in an area of moderate risk for liquefaction. Additional geotechnical investigation to a depth of 50 feet is recommended to meet Tier 1 requirements to rule out the potential for liquefaction at this site. *Structural Priority: Low*

SCHEDULE & COST

It is assumed the building will remain occupied therefore two phases were assumed for the construction schedule to allow approximately half the building to remain in use while work is being completed. Based on this approach the schedule was estimated to be 4 to 6 months. This includes time for relocation of staff, completion of structural work, and final TI items.

Based on the schedule and the conceptual structural retrofit approach to the total cost for the project is estimated at approximately $440,000 dollars. This cost does not include soft costs such as permit fees, design profession fees, special inspection fees, etc.

The following evaluation report details our findings.

INTRODUCTION

The purpose of this evaluation is to review and evaluate the structural systems of the subject building using criteria provided by ASCE 41-17. The evaluation criteria have been tailored for specific building types and desired levels of building performance. This standard is based on criteria developed from observation of structural and non-structural damage occurring in previous earthquakes and provides a means to identify general deficiencies based on anticipated behavior of specific building types.

The evaluation begins with a Screening Phase (Tier 1) to assess primary components and connections in the seismic force resisting system through the use of standard checklists and simplified structural calculations. Checklist items are general in nature and are intended to highlight building components that do not exceed conservative construction guidelines. If the element is compliant, it is anticipated to perform adequately under seismic loading without additional review or strengthening. Items indicated as non-compliant in a Tier 1 checklist are considered potential deficiencies that require further analysis.

A limited, deficiency-based Evaluation Phase (Tier 2) can then be used to review the items determined to be potential deficiencies by Tier 1 checklists and simplified calculations. Non-compliant items are evaluated for calculated linear seismic demand as determined by ASCE 41-17. If the elements are compliant per Tier 2 analysis, the Tier 1 deficiency is waived. However, if the element remains non-compliant after the more detailed Tier 2 analysis, repair or remediation of the deficiency is recommended.

In certain cases, a more detailed Systematic Evaluation (Tier 3) may be more appropriate for complex structures where a Tier 2 analysis may be considered significantly conservative. A Tier 3 structural evaluation generally requires a substantially greater level of effort than a Tier 2 review.

EVALUATION OVERVIEW

This seismic evaluation report for the existing buildings located at 55 Stony Point Road, Santa Rosa, CA, is based on the following:

- The American Society of Civil Engineers/ Structural Engineering Institute (ASCE/SEI 41-17) Standard for Seismic Evaluation and Retrofit of Existing Buildings - Tier 1 and Tier 2 (non-compliant items only), Immediate Occupancy level structural evaluation criteria.

- A site visit for general review of the structure performed on 7/27/2022. No destructive testing or removal of finishes was performed or included in scope.

- Review of following original drawings:
    - Structural drawings by Graham & Kellam Structural Engineers (1979) for the foundation.
    - Structural drawings by Christensen & Foster (1980) for the metal building.

- Existing material properties as indicated in Appendix C.

- Review of following geotechnical reports and hazard maps:
    - Geotechnical Exploration for Finley Community Park by ENGEO Inc. (Project No. 18584.000.001, June 1, 2021).
    - Liquefaction Susceptibility and Surface Fault Rupture hazard maps.

- Review of non-structural elements is not included in the scope of this review.

STRUCTURE OVERVIEW

General Description

The Administration building at the Municipal Service Center North was designed under the 1976 Uniform Building Code (UBC) as an office building. Correlating this information to the information presented in Table 1604.5 of the 2019 CBC, the building was constructed to a Risk Category II standard. The building is located on a flat commercial lot approximately 1.5 miles west of Highway 101 and approximately 0.9 miles north of the Highway 12 in Santa Rosa on Stony Point Road. The main entrance is facing south-west, away from Stony Point Road.

General Objective of Evaluation

The City of Santa Rosa (City) requested a seismic evaluation to identify potential seismic remediations required to bring the building into current standards by performing an ASCE 41 Tier 1 & Tier 2 evaluation to the Immediate Occupancy performance level. The City's stated goal is to determine if the current building is constructed to or can be modified to be an "Essential Services Building". According to Table 1604.5 of the 2019 California Building Code (CBC), essential facilities "designated emergency preparedness, communications and operations centers and other facilities required for emergency response" are assigned to Risk Category IV.

ASCE 41 contains predefined Seismic Hazard Levels. Basic Safety Earthquake (BSE) levels BSE-1E and BSE-2E are for use with the Basic Performance Objective for Existing Buildings. BSE-1N and BSE-2N for use with the Basic Performance Objective Equivalent to New Building Standards. Typically, an ASCE 41 Tier 1 & Tier 2 evaluation would utilize the BSE-1E level forces. However, there are various factors outlined in the 2019 California Existing Building Code (CEBC) that trigger a more in-depth analysis under the Basic Performance Objective Equivalent to New Building Standards. One of the factors is described in Section 506.4.3 of the CEBC:

> *Where a change of occupancy results in a building being assigned to a higher risk category, the building shall satisfy the requirements of Section 1613 of the California Building Code for the new risk category using full seismic forces.*

Section 303 of the CEBC outlines the requirements for structural design loads and evaluation and design procedures. The statement "*full seismic forces*" in Section 506.4.3 means Section 303.3.1 (Compliance with full seismic forces) must be used for the analysis. The criteria utilized for this evaluation is in accordance with Table 303.3.1, which requires BSE-1N earthquake hazard level.

By their nature, metal buildings are lighter than other construction methods. Because of this, wind loads sometimes govern the lateral design over the seismic forces, even in some high seismic zones. Utilizing current code (ASCE 7-16), an overall wind versus seismic loading comparison was performed and confirmed that wind does not govern the design for this building.

Structural Performance Objective

Per ASCE 41-17, a structural performance objective consists of a target performance level for structural elements in combination with a specific seismic hazard level. For seismic assessment of the subject building, the Basic Performance Objective for New Buildings (BPON) is used.

For the purposes of this review to the BPON, the specified level of structural performance is **Immediate Occupancy (1-A)** for this essential structure (Risk Category IV as defined by ASCE 7). The Immediate Occupancy Performance Level as described by ASCE/SEI 41-17: *'Structural Performance Level S-1 is defined as the post-earthquake damage state in which a structure remains safe to occupy and essentially retains its pre-earthquake strength and stiffness.'*

Additionally, metal buildings are susceptible to high levels of drift during seismic events. While drift may not cause significant structural damage, it may result in damages to non-structural elements. Low levels of drift can result in damages to finishes such as diagonal cracks in drywall. High levels of drift can result in damages to door frames that hinder the door from opening, broken windows, and damages to utility lines that may be critical to the function of the building. The lobby used as the entrance to the building may be unusable after high levels of drift. Drift can be mitigated by stiffening the lateral force resisting system. ASCE 41 does not explicitly outline an analysis procedure for evaluating drift. ZFA performed a drift analysis, utilizing current code (ASCE 7-16) values to compare the actual drift to the allowable drift specified in ASCE 7-16.

Site Seismicity (Earthquake Activity)

Per ASCE 41-17, 'seismicity', or the potential for ground motion, is classified into regions defined as Low, Moderate, or High. These regions are based upon mapped site accelerations S_s and S_1 which are then modified by site coefficients F_a and F_v to produce the Design Spectral Accelerations, S_{DS} (short period) and S_{D1} (1-second period). The successful performance of buildings in areas of high seismicity depends on a combination of strength, ductility of structural components, and the presence of a fully interconnected, balanced, and complete seismic force-resisting system. Where buildings occur in lower levels of seismicity, the strength and ductility required for better performance is significantly reduced and building components or connections with additional strength capacity can in some cases be adequate despite lacking ductility.

Based on a geotechnical report provided for an adjacent site, the soil profile of this building can be classified as *Site Class D* per ASCE 41-17 for use in determination of site coefficients F_a and F_v.

Per the site values indicated by USGS data and evaluated using seismic acceleration equations and tables of ASCE 41-17, the site is located in a region of **High Seismicity** with a design short-period spectral response acceleration parameter (S_{DS}) of 1.258g and a design spectral response acceleration parameter at a one second period (S_{D1}) of 0.817g. Per the table shown below, both of these parameters exceed the lower boundaries for high seismicity classification, 0.5g for S_{DS} and 0.2g for S_{D1}.

Table 1: Level of Seismicity Definitions (per ASCE 41-17 Table 2-4)

*Level of Seismicity**	S_{DS}	S_{D1}
Low	< 0.167g	< 0.067g
Moderate	≥ 0.167g < 0.500g	≥ 0.067g < 0.200g
High	≥ 0.500g	≥ 0.200g

*Where S_{DS} and S_{D1} values fall in different levels of seismicity, the higher level shall be used.

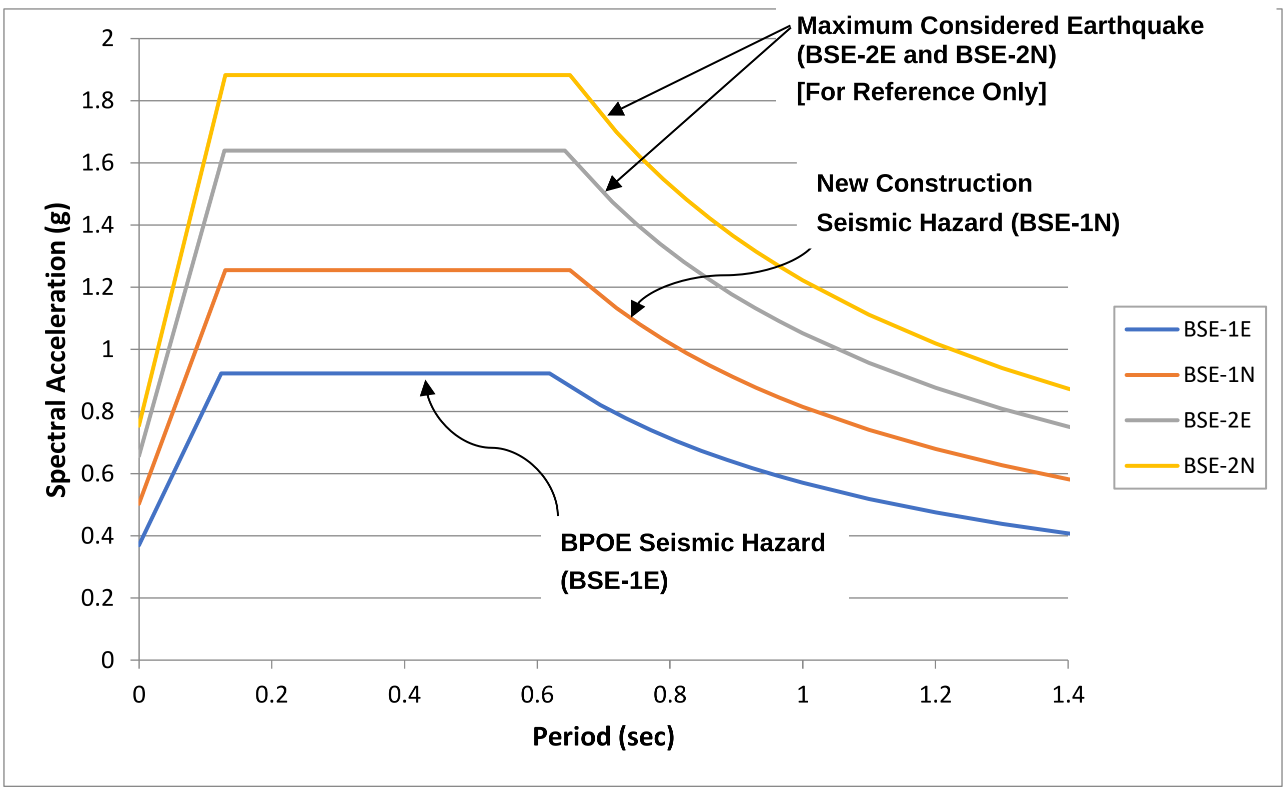
The spectral response parameters S_S and S_1 were obtained for the BSE-1N seismic hazard level for new structures (BPON). The acceleration values were adjusted for the maximum direction and site class in accordance with ASCE 41 Section 2.4.1 to determine the design values for the Tier 1 and Tier 2 analyses.

The following chart depicts the response spectra for the multiple seismic hazard levels defined by ASCE 41-17, two existing hazard levels and two hazard levels corresponding to code design of new structures (ASCE 7).

Table 2: Response Spectra Criteria

Seismic Hazard Level*	Building Code Reference	Design Spectral Acceleration $S_a(T)$
BSE-1E	ASCE 41-17 (20%/50yr)	0.92g
BSE-1N	ASCE 7-16 Design Basis Earthquake (DBE)	1.26g
BSE-2E	ASCE 41-17 (5%/50yr)	1.64g
BSE-2N	ASCE 7-16 Maximum Considered Earthquake (MCE)	1.88g

* Seismic hazard levels denoted with 'E' for existing buildings or 'N' for new building equivalency.



Change in Structural Risk Category

The City's stated goal to modify the building from an office building to an Essential Services Building represents a change in structural risk category. Per the California Existing Building Code (CEBC) a change in Risk Category triggers the building being evaluated, and strengthened as required, to current code.

Section 506.4.3 of the CEBC:

> *Where a change of occupancy results in a building being assigned to a higher risk category, the building shall satisfy the requirements of Section 1613 of the California Building Code for the new risk category using **full seismic forces**.*

Section 303 of the CEBC outlines the requirements for structural design loads and evaluation and design procedures. The statement "***full seismic forces***" in Section 506.4.3 means Section 303.3.1 (Compliance with full seismic forces) must be used for the analysis which is defined as BSE-1N (see Figure 3).

[BS] TABLE 303.3.1
PERFORMANCE OBJECTIVES FOR USE IN ASCE 41 FOR COMPLIANCE WITH FULL SEISMIC FORCES

RISK CATEGORY (Based on CBC Table 1604.5)	STRUCTURAL PERFORMANCE LEVEL FOR USE WITH BSE-1N EARTHQUAKE HAZARD LEVEL	STRUCTURAL PERFORMANCE LEVEL FOR USE WITH BSE-2N EARTHQUAKE HAZARD LEVEL
I	Life Safety (S-3)	Collapse Prevention (S-5)
II	Life Safety (S-3)	Collapse Prevention (S-5)
III	Damage Control (S-2)	Limited Safety (S-4)
IV	Immediate Occupancy (S-1)	Life Safety (S-3)

Figure 1: Table from 2019 CEBC

Structural System and Materials Description

General

The building is approximately 6,000 square feet, single-story, pre-engineered metal building. There is a small lobby between the subject building and adjacent metal building designed and constructed at the same time. The building is used for administration offices and has interior non-structural walls. All non-structural items are outside the scope of this report. The footprint is approximately 100-feet by 60-feet. The roof is approximately 108-feet by 68-feet. The additional roof area is a result of a 4-foot wide eave all around. (See Appendix A – Schematic Site Map).

Roof Framing

The building is approximately 13-feet 6-inches tall. The roof is covered with a non-structural metal deck over light-gauge metal "Z" shaped purlins spaced approximately 5-feet apart. The purlins are supported by perpendicular built-up steel "I" shaped beams spaced approximately 20-feet apart that span the full width of the building to exterior walls. The beams are supported by built-up steel "I" shaped columns, which transfer the roof loads to the foundations.

Walls

All around the building, the exterior walls are covered by non-structural metal panels with a clerestory light window at the top. The metal panels are supported by horizontal light-gauge metal "C" shaped purlins spanning between beam support columns, which are spaced approximately 20-feet apart. There are no interior structural walls.

Seismic Force-Resisting System

In the plan east-west direction, the seismic force resisting system consists of moment frames spaced approximately 20-feet apart (see Figure 1).

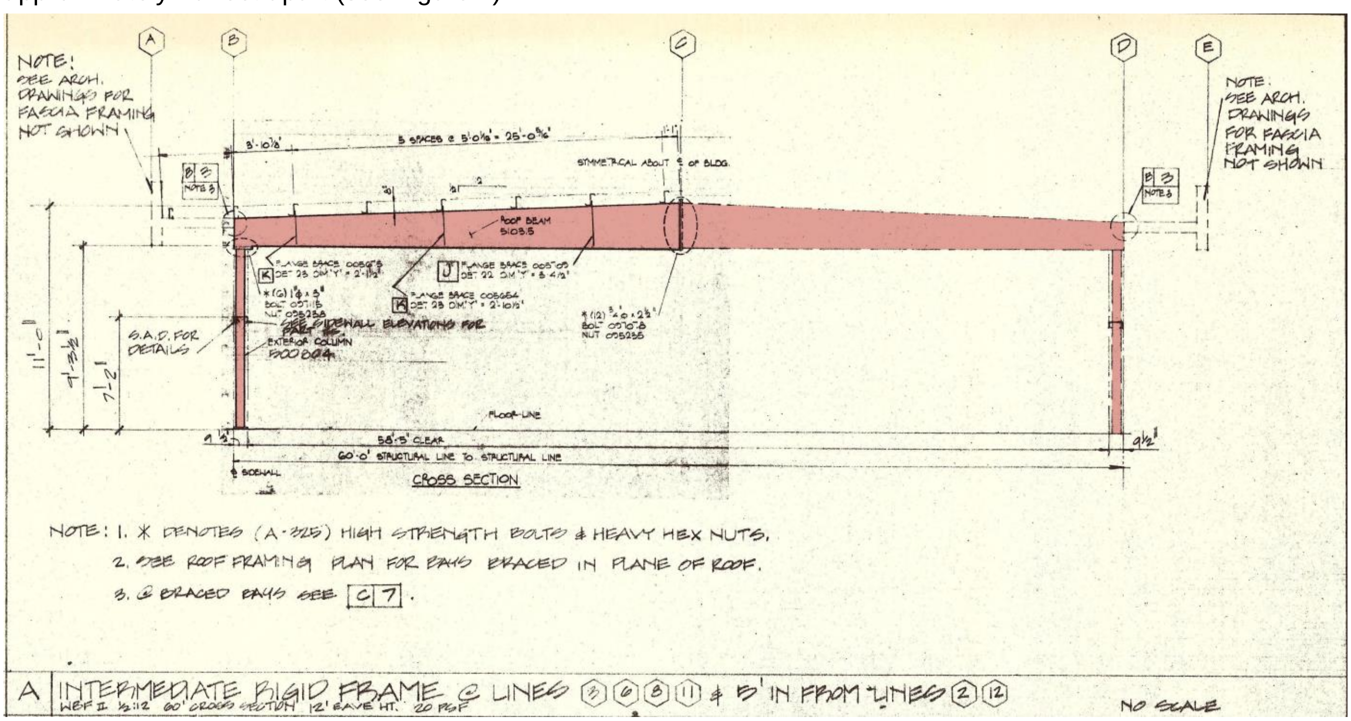


Figure 2: Existing Moment Frame Elevation

In the plan north-south direction, the seismic force resisting system consists of “X” tension rod braced frames at the exterior walls, approximately 60-feet apart. There are two bays of “X” tension rod braces along each of the two walls lines, for a total of four “X” braced frames in the plan north-south direction (see Figure 2).

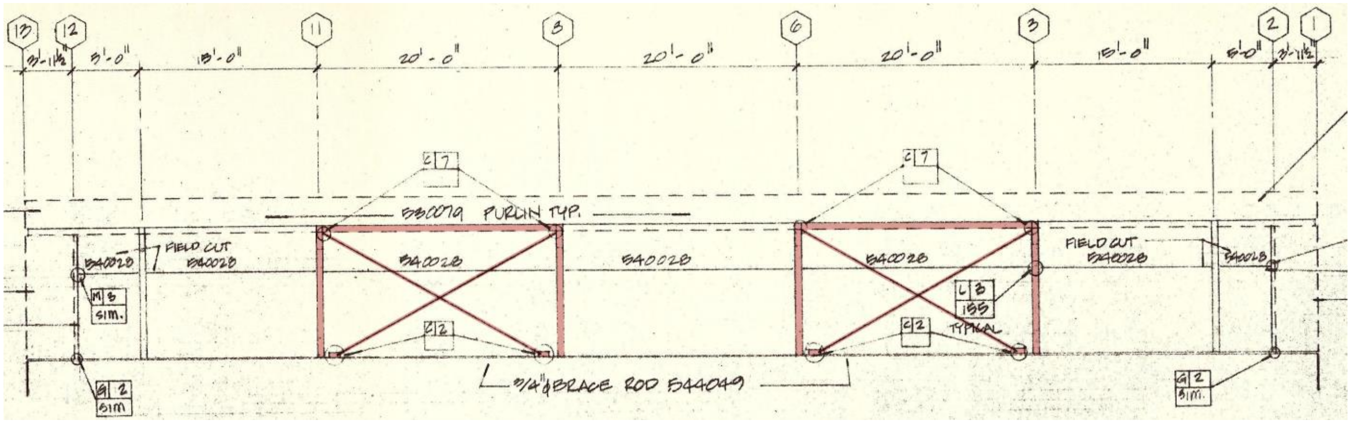


Figure 3: Existing Braced Frame Elevations

The roof diaphragm consists of “X” tension rod braces between moment frames. There are three bays of “X” tension rod braces between two bays of moment frames, for a total of six “X” tension rod braces in the roof plane. The light-gauge “Z” shaped purlins tie the roof together and transfer the loads to the “X” tension rod braces. The “X” tension rod braces occur in the same bay as the plan north-south “X” tension rod braced frames to transfer loads to the foundation. They also occur between moment frames in the plan east-west direction to transfer loads to the foundation.

Foundations

Foundations are spread concrete footings with a concrete slab on grade floor system. Pad footings occur at columns. Continuous footings occur between pad footings at the perimeter. Continuous tie footings occur parallel to moment frames at pad footings within the interior of the building perimeter.

Field Verification and Condition Assessment

The structure appears in generally good structural condition with minimal structural damage or deterioration apparent. It appears to be constructed in general accordance with the provided structural drawings.

Material Properties

Basic properties for existing structural materials found on existing building documentation or ASCE 41 code prescribed minimum structural values utilized in the analysis calculations can be found in Appendix C.

Building Type

Per ASCE/SEI 41-17, this building can be classified as **Building Type S3: Steel Light Frames**. As described by ASCE/SEI 41-17: *‘These buildings are pre-engineered and prefabricated with transverse rigid steel frames. They are one-story in height. The roof and walls consist of lightweight metal, fiberglass, or cementitious panels. The frames are designed for maximum efficiency, and the beams and columns consist of tapered, built-up sections with thin plates. The frames are built in segments and assembled in the field with bolted or welded joints. Seismic forces in the transverse direction are resisted by the rigid frames. Seismic forces in the longitudinal direction are resisted by wall panel shear elements or rod bracing. Diaphragm forces are resisted by untopped metal deck, roof panel shear elements, or a system of tension-only rod bracing. The foundation system may consist of a variety of elements.’*

Historical Performance

Modern steel moment frame systems came about in the 1960's when beam flanges and webs were welded directly to the columns to create fully restrained sections. Shear tabs bolted to the beam webs and welded to the columns later replaced welded beam webs. These welded-flange and bolted-web connections were used extensively from the 1970's through the early 1990's and are now known as the pre-Northridge connections.

The low-rise metal building industry pioneered the use of moment end-plate connections in the United States. Rigid frame construction is usually assumed for the design of the frames. The end-plate moment connection saw its first application in the 1960's, stemming from research in the 1950's. The early designs usually resulted in thick end-plates and large bolt diameters due mainly to simplified design assumptions and analyses of the connection. The connection slowly gained acceptance and was included in the AISC Manual of Steel Construction, 7th Ed. (1970).

These frames did not perform as well as expected during the 1994 Northridge earthquake. A significant number of the frames inspected after the earthquake exhibited visible cracking in the beam flange-to column welds resulting in brittle failures of the beam to column connection that could cause floors to collapse. Buildings that relied on deep beams that are stronger than the columns are more susceptible to this type of damage.

Currently moment frames are designed to force beam yielding away from the column and the connection by using strong columns compared to beams and reducing the beam section adjacent to the connection at columns. This connection allows the beam to yield and prevent brittle failures. Moment frame buildings are generally flexible and subject to large drifts. Their ductility is achieved through yielding of beams and/or shear yielding of column panel zones at beam-column connections. This inelastic behavior allows moment frames to sustain many cycles of loading and load reversals (seismic loading). The subject building was designed prior to the 1994 Northridge earthquake and appears rely on a deep beam system with limited redundancy. The frame connections from the beam to the columns are detailed in the standard method for pre-Northridge structures. As with all buildings of this type there is a risk of brittle failure of the frame connections.

Benchmark Buildings

In addition to classifying buildings by type of construction, ASCE 41 identifies 'Benchmark Buildings' for each type. The detailing of seismic force-resisting systems in Benchmark Buildings is generally considered to meet the performance requirements of ASCE 41. When a building is determined to meet Benchmark Building requirements through field verification of construction compliant with benchmark code requirements, only review of foundation and non-structural elements is required. Even though a building appears meets the benchmark criteria, a full analysis may still be recommended under certain circumstances.

For building type S3, the 2000 seismic design provisions are referenced as the oldest permitted standard. Since the subject building was designed in 1979, and per the provided documentation was designed under the 1976 Uniform Building Code, it does not meet the criteria of a Benchmark Building and a complete Tier 1 analysis is required.

FINDINGS AND RECOMMENDATIONS

Structural

The ASCE 41-17 Tier 1 Immediate Occupancy Basic Configuration and Building Type Specific Checklists indicate the primary building structure as non-compliant in five (5) areas for Immediate Occupancy Performance.

a. *ADJACENT BUILDINGS: The clear distance between the building being evaluated and any adjacent building is greater than 0.5%of the height of the shorter building in low seismicity, 1.0% in moderate seismicity, and 3.0% in high seismicity.*

FINDINGS: The single-story Lobby building adjacent to the single-story Administration building is constructed with approximately 4.5-inches of clear distance between structural columns. The lobby structure is approximately 10-feet tall, so the clear distance needs to be 3.6-inches minimum.

DISCUSSION: The structural framing meets the 3.0% clear distance requirement for high seismicity, but there are nonstructural finishes extending across the seismic gap. Since this is the main entrance to an Immediate Occupancy building, ZFA believes it is prudent to classify this as non-compliant.

RECOMMENDATION: Additional analysis may be performed to estimate horizontal movement in a seismic event. Minor damage may occur due to pounding between structures during a seismic event; however, damage due to this condition is not anticipated to cause structural concerns within the subject building. Egress issues are recommended to be further analyzed. Any MEP systems or communication systems cross this joint shall have flexible connection to accommodate differential movement.

b. *BRACE AXIAL STRESS CHECK: The axial stress in the diagonals, calculated using the Quick Check procedure of Section 4.4.3.4, is less than 0.50F_y.*

FINDINGS: The axial stress in the diagonals, calculated using the Quick Check procedure, is greater than 0.50F_y.

DISCUSSION: Further analysis of the braces was performed using the Tier 2 procedures. The results indicate the braces have insufficient strength to resist the seismic forces associated with the BPON standards however the connection of the braces to the structure can develop the seismic forces.

RECOMMENDATION: Since the connection to the structure is adequate, the braces and their connections can be replaced with larger diameter braces proportioned to resist the required seismic demand.

c. *Flexural Stress Check: The Average flexural stress in the moment frame columns and beams, calculated using the Quick Check Procedures of Section 4.4.3.9 is less than Fy.*

FINDINGS: The columns are 134% stressed using the Quick Check procedure

DISCUSSION: Further analysis was performed using the Tier 2 procedures for the entire moment frame was completed and the columns were found to be overstressed for both BSE-1N and BSE-2N analysis.

RECOMMENDATION: The frame columns should be strengthened by welding full height plates to the flanges. The plates, proportioned to both increase the strength and limit the drift below the allowable bound, would be relatively thick to provide enough stiffness in the column. See Conceptual Retrofit Discussion for a more detailed discussion of retrofitting the existing moment frames.

d. *MOMENT-RESISTING CONNECTIONS: All moment connections are able to develop the strength of the adjoining members or panel zones.*

FINDINGS: The moment connections are not able to develop the strength of the beam or the column using the Quick Check procedure.

DISCUSSION: Further analysis of non-compliant elements was performed using the Tier 2 procedures. The connections were determined to be partially restrained (PR) and a model was utilized to approximate the moment demands at the moment connections. The moment demands were compared against the moment capacity of the connection and the results indicate the connections can resist the moment demands.

RECOMMENDATION: Deficiency waived.

e. *COMPACT MEMBERS: All frame elements meet compact section requirements in accordance with AISC 360, Table B4.1.*

FINDINGS: The columns do not meet the compact section requirements.

DISCUSSION: The strength of the column was determined in accordance with ASCE 41 & AISC 360 and compared to the demands determined from the model. The results indicate the column is not adequate to resist the seismic demands.

RECOMMENDATION: See item c discussion above.

RETROFIT OPTIONS, SCHEDULE, AND COST DISCUSSION

An ASCE 41 Tier 1 and 2 analysis of the building for deficiencies in the lateral force resisting system indicates the building has deficiencies in both principal directions. The following retrofit options and additional considerations are triggered due to the change in structural risk category, which requires the building be brought into conformance with current code.

<u>Moment Frame Retrofit Discussion:</u>

Option 1: Strengthen existing frames with 1 ½” thick x 8” wide steel plates welded to each flange at each column for the full height. Additionally, the weld from the existing web to existing flange will need to be strengthened with 3/8” continuous fillet weld. See S-1.1 for conceptual details for this strengthening. This approach would maximize the open space in the floor plan. Based on the cost estimate provided by OC Insight, Appendix E, the estimated cost for the structural steel strengthening is $160,000. In addition to the steel costs there is an estimated $20,000 of interior finish repairs post strengthening for a total direct cost of approximately $180,000.

Option 2: In lieu of strengthening the existing moment frames, the building could be converted to a wood or metal stud framed shear wall system by adding new shear walls along each primary frame line. Each new shear wall would also require a new continuous concrete footing. This approach provides a structural system that will drift less than retrofitting the moment frame. Decreased drift will reduce deflection compatibility requirements for non-structural walls, MEP systems, etc. to meet current code. See S-1.2 for conceptual plan and detailing. The estimated cost for this retrofit approach is approximately $105,000. This cost estimate only covers the direct cost but based on the location of the moment frames and the current office conference room layout this approach would require a larger TI to incorporate the structural retrofit into the design. This TI would likely cost significantly more than the moment frame retrofit therefore Option 1 is recommended.

<u>Tension Rod Retrofit Discussion:</u>

For the tension only rod brace direction the retrofit will consist of upsize the existing rod bracing and the clevis/turnbuckle at each end. See S-1.3 for schematic elevations and details. The estimated cost for the structural steel scope of work is approximately $5,000 with an additional $25,000 of interior finishes repairs post strengthening.

<u>Schedule Discussion:</u>

The building is currently partially occupied, therefore the project schedule assumed phased construction to allow the people to remain in the building. The work would begin in the north portion of the building. Once complete, the people in occupied offices could be relocated and work in the south portion of the building until completion. Based on this approach, the scheduled was estimated at 4-6 months.

Cost Discussion:

The numbers including the discussion points above are direct costs only and do not include Markups and General Conditions. Since this is a public project, it requires a higher level of oversight for the general contractor. Due to the relatively small nature of the project and the assumed phasing this results in the general conditions representing a relatively high portion of the total project costs. Summary below.

Direct Cost			
Moment Frame retrofit Option 1 & wall repair		$ 180,000.00	
Braced Frame Retrofit & wall Repair		$ 30,000.00	
TI: Carpet, Paint, etc.		$ 15,000.00	
Total Net Direct Cost		**$ 225,000.00**	
General Markups			
Design Contigency	20.00%	$ 45,000.00	
Escalation to Midpoint	6.42%	$ 14,445.00	
Contractor Contigency	7.00%	$ 15,750.00	
General Conditions/Requirements	47.50%	$ 106,875.00	<-- Assuming 6 month
Contractor Overhead and Porfit	8.00%	$ 18,000.00	construction
Insurance	3.50%	$ 7,875.00	
Total Construction Cost		$ 432,945.00	

The scope of work for this report was limited to a structural review, however there are many nonstructural items that may require review and mitigation to meet current code requirements trigger due to the change in occupancy. The following represents a few of these considerations but is not an all-inclusive list:

- Drift capability for all interior non-load bearing metal stud walls
- Code compliant attachments for all mechanical, electrical, plumbing systems including seismic anchorage
- Fire sprinkler bracing in conformance with NFPA 13 requirements including seismic bracing

RELIABILITY OF SEISMIC EVALUATIONS

In general, structural engineers do not have the ability to predict the exact damage to a building as a result of an earthquake. There will be a wide variation of damage from building to building due to the variations in ground motion and varying types and quality of construction. In addition, engineers cannot predict the exact ground motions of the earthquake that may strike a given building. Design and evaluation of buildings are performed using general guidelines and information from past earthquakes. Engineers and the codes used for design and evaluation have been conservative when attempting to ensure that building design meets minimum standards of life safety. This effort is based on science and technology as well as on observations made from actual seismic events. Building design and evaluation codes are constantly evolving to better meet performance targets based on this information. Continued research will improve predictive methods and facilitate performance-based engineering. It has been estimated that, given design ground motions, a small percent of new buildings and a slightly greater percent of retrofit buildings may fail to meet their expected performance.

CLOSING

The seismic review and analyses associated with this evaluation were based on available original structural drawings, and the site reviews were based on that which was plainly visible. No attempt was made to uncover hidden conditions or perform any destructive or non-destructive testing. The items discussed in this report are subject to revision should more information become available.

This report is general in nature and does not imply that the recommendations listed above are the only structural requirements that must be made to the existing structure to meet current code criteria.

We understand you may have questions regarding this evaluation and are available for comment and explanations. Please call with any questions you may have. Thank you for choosing ZFA Structural Engineers to assist you with this building seismic review.

Kyle Bettencourt
Engineer
ZFA Structural Engineers

Luke Wilson, SE
Principal
ZFA Structural Engineers

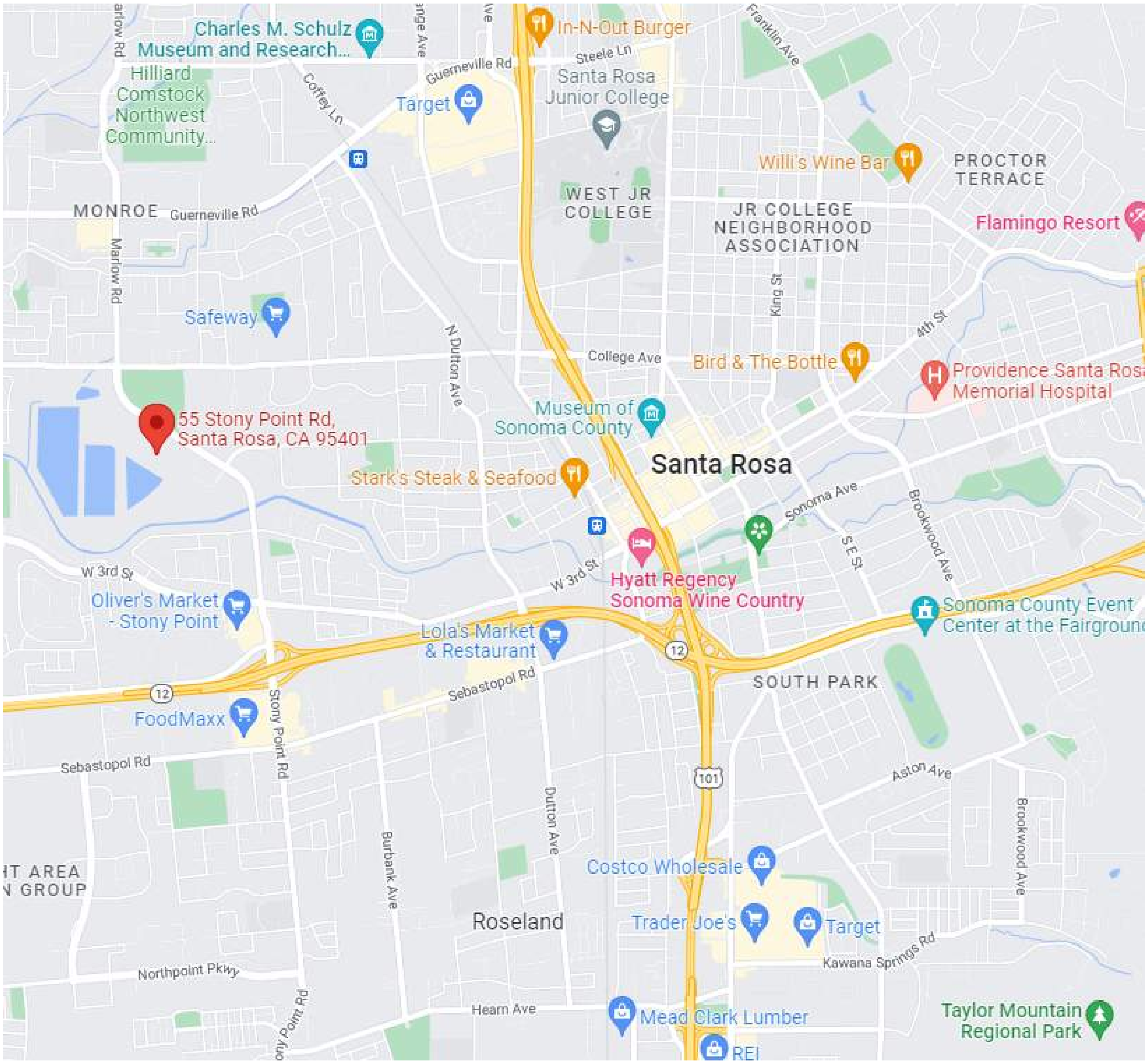
APPENDIX A – PHOTOGRAPHS

Photo 1: Northwest Corner

Photo 2: Northeast Corner

APPENDIX B – MAPS & PLANS

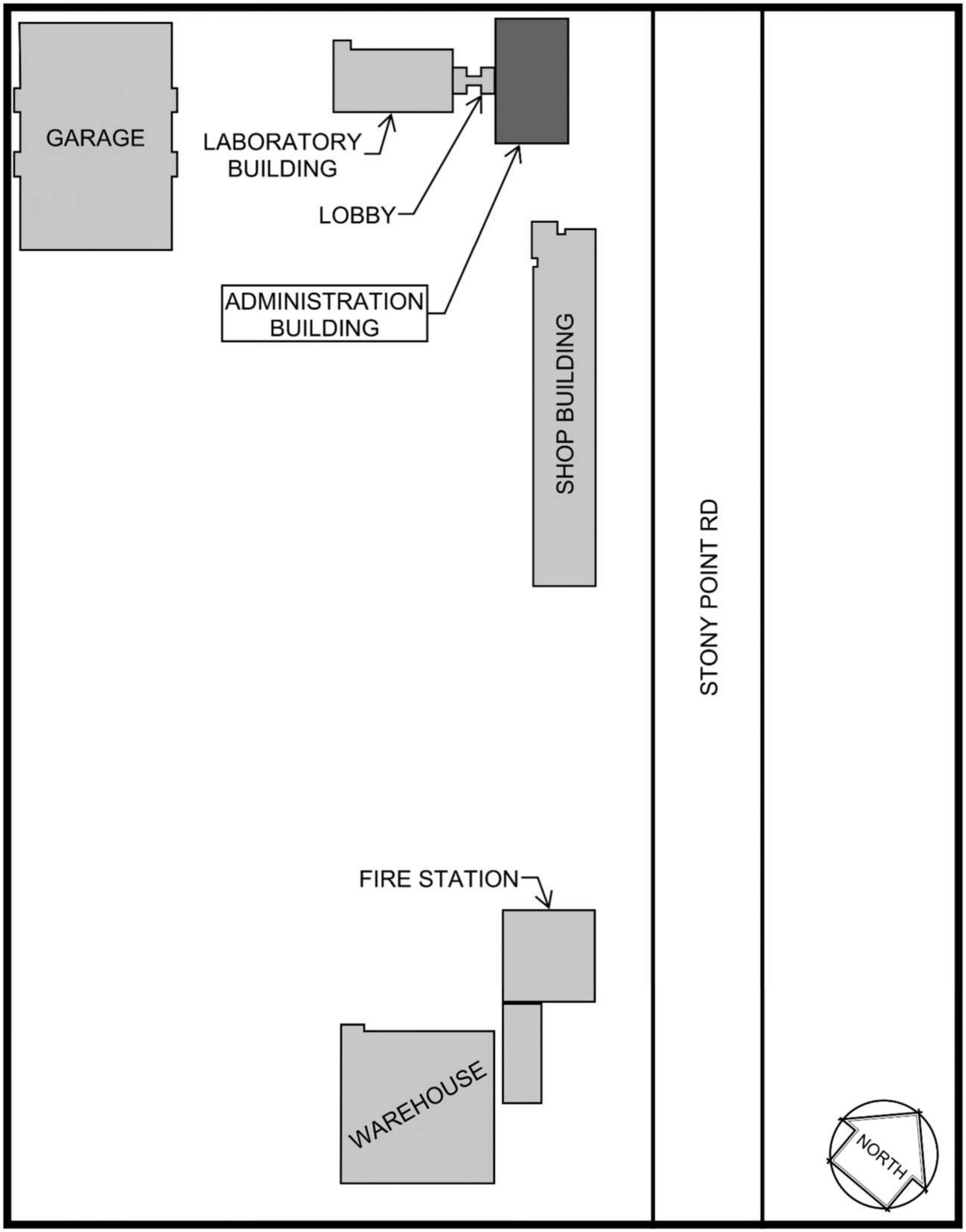
<u>Location Map</u>



Campus Map

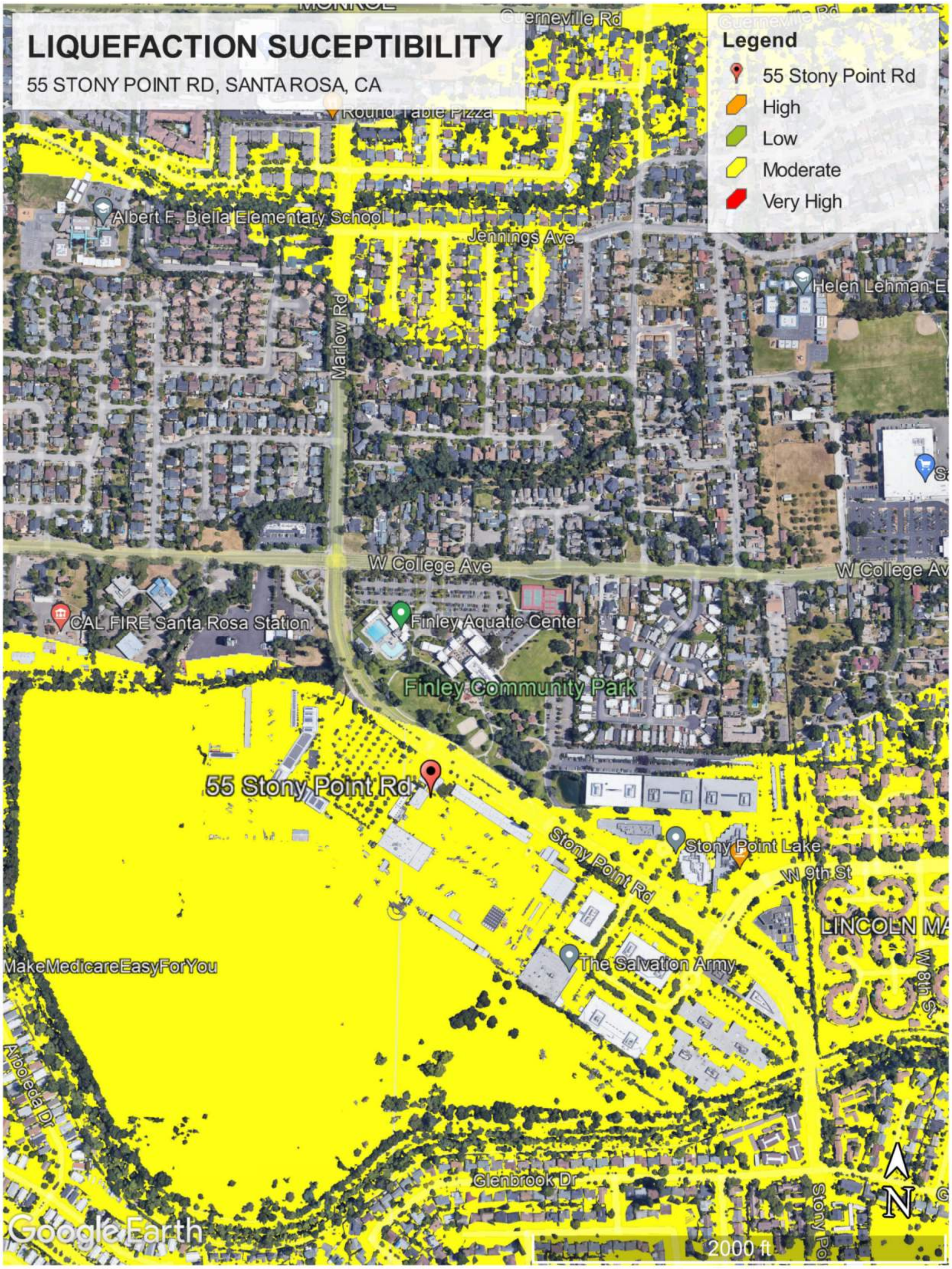

er department
55 Stony Point Rd
Stony Point Rd
The Salvation Arm

<u>Schematic Site Plan</u>



Liquefaction Susceptibility Map

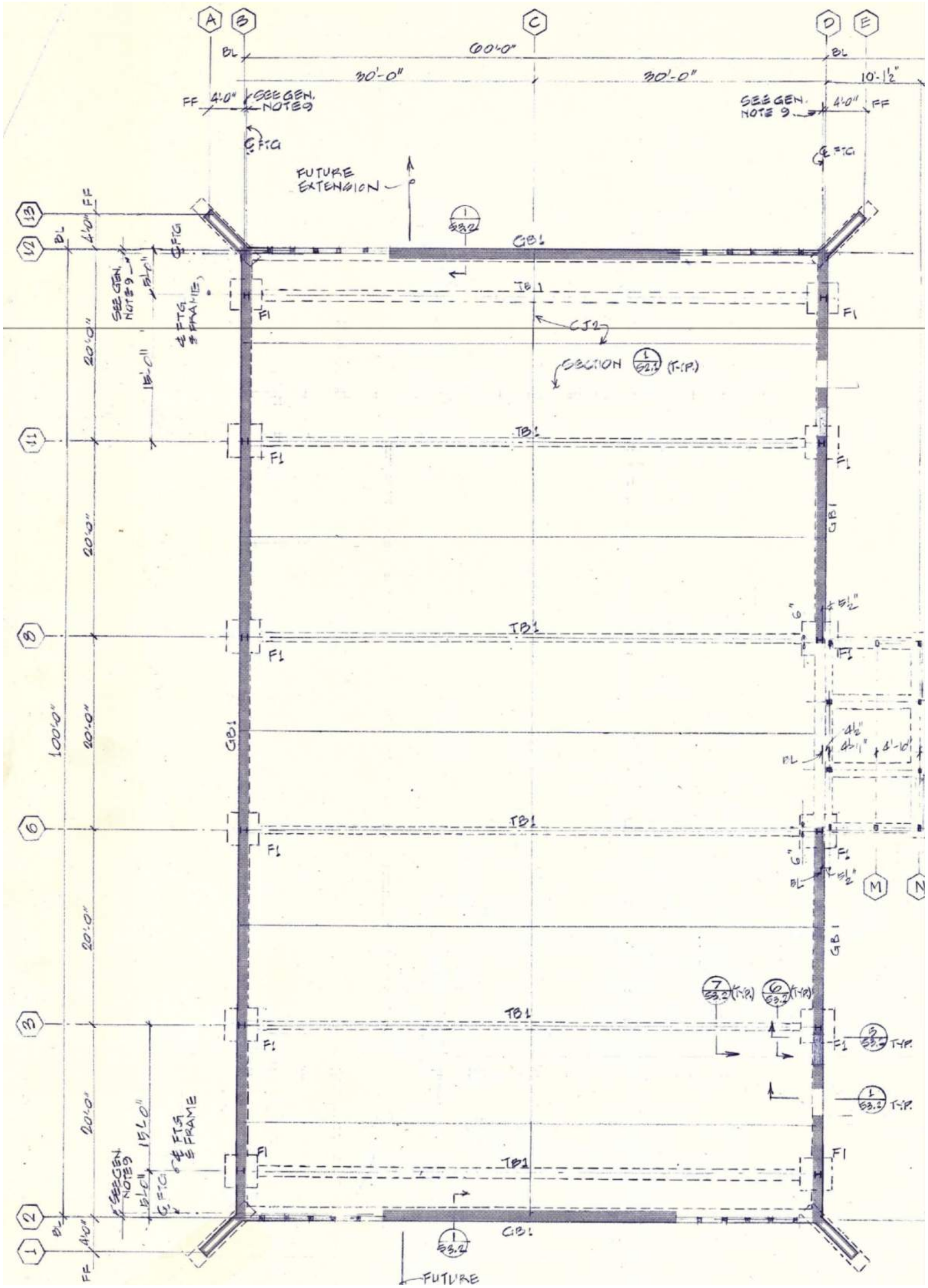
Per the Liquefaction Susceptibility Map below, the subject site is located in an area that has a moderate probability for liquefaction in a seismic event.



Surface Fault Rupture Map

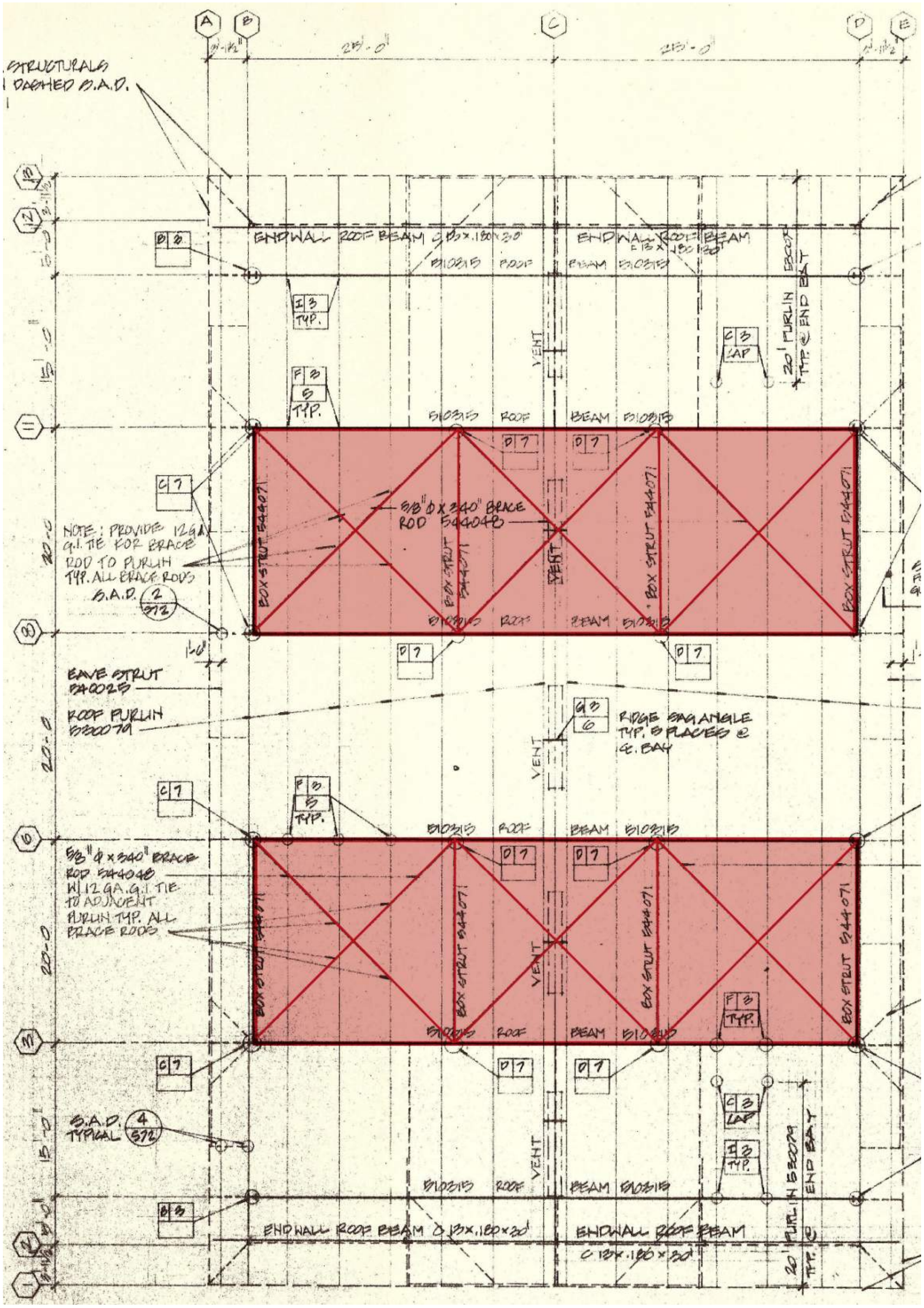
No faults are indicated in the vicinity of the subject site, indicating a negligible risk of ground surface rupture.

1979 Foundation Plan



<u>1980 Metal Building Roof Plan</u>

Braced diaphragms highlighted in red.



APPENDIX C – SUMMARY DATA SHEET

Summary Data Sheet

BUILDING DATA

Building Name:	Administration Building	Date:	August, 2022
Building Address:	55 Stony Point Drive, Santa Rosa, CA 95401		
Latitude:	38.44238 Longitude: -122.74957	By:	ZFA / KPB / LSW

Year Built:	1980	Year(s) Remodeled:	N/A	Original Design Code:	1976 UBC
Area (sf):	6000	Length (ft):	100	Width (ft):	60
No. of Stories:	1	Story Height (ft):	13.5	Total Height (ft):	13.5

USE ☐ Industrial ☑ Office ☐ Warehouse ☐ Hospital ☐ Residential ☐ Educational ☐ Other:

CONSTRUCTION DATA

Gravity Load Structural System:	Steel Beams and Columns		
Exterior Transverse Walls:	Cold Formed Steel Framed	Openings?	Yes
Exterior Longitudinal Walls:	Cold Formed Steel Framed	Openings?	Yes
Roof Materials/Framing:	Cold Formed Steel Purlins and Hot Rolled Steel Beams		
Intermediate Floors/Framing:	N/A		
Ground Floor:	Concrete Slab on Grade		
Columns:	Steel	Foundation:	Spread Footings
General Condition of Structure:	Good		
Levels Below Grade?	No		
Special Features and Comments:	-		

LATERAL-FORCE-RESISTING SYSTEM

	Longitudinal	Transverse
System:	Tension Rod Bracing	Moment Frames
Vertical Elements:	Tension Rod Bracing	Tension Rod Bracing
Diaphragms:	Tension Rod Bracing	Tension Rod Bracing
Connections:	Bolts & Welds	Bolts & Welds

EVALUATION DATA

BSE-1N Spectral Response Accelerations:	S_{DS}=	1.258	S_{D1}=	0.817
Soil Factors:	Class=	D	F_a= 1.0	F_v= 1.7
BSE-1E Spectral Response Accelerations:	S_{XS}=	0.925	S_{X1}=	0.572
Level of Seismicity:		High		
Performance Level:		Immediate Occupancy		
Building Period:	T=	0.141		
Spectral Acceleration:	S_a=	0.925		
Modification Factor:	$C_mC_1C_2$=	1.4	Building Weight: W=	110k
Pseudo Lateral Force:	$V=C_mC_1C_2S_aW$=	1.292W		

BUILDING CLASSIFICATION:	**S3 – Metal Building Frames**

REQUIRED TIER 1 CHECKLISTS	Yes	No
Basic Configuration Checklist	☑	☐
Building Type S3 Structural Checklist	☑	☐
Nonstructural Component Checklist	☐	☑
FURTHER EVALUATION REQUIREMENT:		

Material Properties

To account for uncertainty in the as-built data, a knowledge factor, κ, is determined according to ASCE 41 Table 6-1. Where material properties are listed in the existing construction documents, a knowledge factor of κ=0.9 is permitted and applied to the component capacities for deformation-controlled and force-controlled actions.

			Default Value per ASCE 41, 4.2.3?	*Alternate Value Source?*
Concrete			Table (4-2)	
Slab on Grade:	f'_c=	3000psi	☐	1979 Drawings
Walls:	f'_c=	3000psi	☐	1979 Drawings
Reinforcing Steel			Table (4-3)	
#4 Bars and Smaller:	f_y=	40ksi	☐	1979 Drawings
#5 Bars and Larger:	f_y=	60ksi	☐	1979 Drawings
Structural Steel			Tables (4-4), (4-5)	
Beams	F_y=	36ksi	☑	
Columns	F_y=	36ksi	☑	

APPENDIX D – TIER 1 CHECKLISTS

TIER 1 CHECKLISTS

Table 17-3. Immediate Occupancy Basic Configuration Checklist
Note: C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.

Very Low Seismicity
Building System—General

☑ C ☐ NC ☐ N/A ☐ U	*LOAD PATH: The structure contains 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 tothe foundation. (Commentary: Sec. A.2.1.1. Tier 2: Sec. 5.4.1.1)* Complete (E) Drawings
☐ C ☑ NC ☐ N/A ☐ U	*ADJACENT BUILDINGS: The clear distance between the building being evaluated and any adjacent building is greater than 0.5%of the height of the shorter building in low seismicity, 1.0% in moderate seismicity, and 3.0% in high seismicity. (Commentary: Sec. A.2.1.2. Tier 2: Sec. 5.4.1.2)* 10ft(3.0%) = 3.6in | (E) Gap = 4.5in
☐ 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)* No Mezzanines

Building System—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)* 1-Story Building
☐ 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)* 1-Story Building
☐ 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)* 1-Story Building
☐ 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)* 1-Story Building
☐ C ☐ NC ☑ N/A ☐ U	*MASS:There is no change in effective mass of 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)* 1-Story Building
☐ 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)* Flexible Diaphragm

TIER 1 CHECKLISTS

Table 17-3. Immediate Occupancy Basic Configuration Checklist
Note: C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.

Low Seismicity (Complete the Following Items in Addition to the Items for Very 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 do not exist in the foundation soils at depths within 50 ft (15.2 m) under the building. (Commentary: Sec. A.6.1.1. Tier 2: 5.4.3.1)*
	See Discussion in Report
☑ C ☐ NC ☐ N/A ☐ U	*SLOPE FAILURE: The building site is located away from potential earthquake-induced slope failures or rockfalls so that it is 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)*
	Flat Site
☑ 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)*
	See Discussion in Report

Moderate and High Seismicity: Complete the Following Items in Addition to the Items for Low 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 0.6Sa. (Commentary: Sec. A.6.2.1. Tier 2: Sec. 5.4.3.3)*
	60/13.5 = 4.44 > 0.6(0.925) = 0.56
☑ 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)*
	Continuous Tie Footings per (E) Drawings.

TIER 1 CHECKLISTS

17.13 Immediate Occupancy Checklist for Building Type S3

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

Very Low and Low Seismicity

Seismic-Force-Resisting System

☐ C ☑ NC ☐ N/A ☐ U	*BRACE AXIAL STRESS CHECK: The axial stress in the diagonals, calculated using the Quick Check procedure of Section 4. 4. 3.4, is less than $0.50F_y$. (Commentary: Sec. A.3.3.1.2. Tier 2: Sec. 5.5.4.1)*
	See Tier 1 & Tier 2 calculations.
☐ 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. (Commentary: Sec. A.3.1.3.3. Tier 2: Sec. 5.5.2.1.2)*
	See Tier 1 & Tier 2 calculations.

Connections

☑ C ☐ NC ☐ N/A ☐ U	*TRANSFER TO STEEL FRAMES: Diaphragms are connected for transfer of seismic forces to the steel moment frames. (Commentary: Sec. A.5.2.2. Tier 2: Sec. 5.7.2)*
	There is a detailed connection.
☑ 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)*
	There is a detailed connection.

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 elastic moment ($F_y S$) of the adjoining members. (Commentary: Sec. A.3.1.3.4. Tier 2: Sec. 5.5.2.2.1)*
	The check for High Seismicity governs. See response in that section.
☐ 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)*
	No reentrant corners.
☐ 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)*
	No large openings in diaphragm.
☑ 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)*
	Horizontal bracing.

TIER 1 CHECKLISTS

Connections

☐ C ☐ NC ☑ N/A ☐ U	*ROOF PANELS: Where considered as diaphragm elements for lateral resistance, metal, plastic, or cementitious roof panels are positively attached to the roof framing to resist seismic forces. (Commentary: Sec. A.5.5.1. Tier 2: Sec. 5.7.5)*
	Roof Panels not used for Diaphragm
☐ C ☐ NC ☑ N/A ☐ U	*WALL PANELS: Where considered as shear elements for lateral resistance, metal, fiberglass, or cementitious wall panels are positively attached to the framing and foundation to resist seismic forces. (Commentary: Sec. A.5.5.2. Tier 2: Sec. 5.7.5)*
	Wall Panels not used for Shear Elements

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. (Commentary: Sec. A.3.1.3.4. Tier 2:Sec. 5.5.2.2.1)*
	See Tier 1 & Tier 2 calculations.
☐ C ☑ NC ☐ N/A ☐ U	*COMPACT MEMBERS: All frame elements meet compact section requirements in accordance with AISC 360, Table B4.1. (Commentary: Sec. A.3.1.3.8. Tier 2: Sec. 5.5.2.2.5)*
	See Tier 1 & Tier 2 calculations.
☑ 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)*
	There are no penetrations detailed or seen when reviewed on site.
☑ 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)*
	There is a brace beam detailed and reviewed on site.
☑ 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)*
	There are angle braces detailed and reviewed on site.

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)*
	See Tier 1 calculation.
☑ 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 following: 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)*
	See Tier 1 calculation.

TIER 1 CHECKLISTS

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.)*
	N/A; on shallow foundations.
☐ 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. (Commentary: Sec. A.6.2.4)*
	N/A; not a sloping site.

APPENDIX E – COST ESTIMATE

ZFA Structural Engineers
Santa Rosa Structural Evaluation

55 Stony Point Road, Santa Rosa, CA

ZFA Structural Engineers
SCHEMATIC RETROFIT
OCMI JOB #: 220665.000
31 October 2022

OC INSIGHT

COST ESTIMATE

INTRODUCTORY NOTES

This estimate is based on verbal direction from the client and the following items, received 26 October 2022:

Structural	S-1.1, S-1.12, S-1.3 (3 sheets)
Specifications and Reports	Seismic Evaluation Report prepared by ZFA Structural Engineers 10/21/22

The following items are excluded from this estimate:

- Professional fees.
- Building permits and fees.
- Inspections and tests.
- Furniture, fixtures & equipment, except as noted.
- Installation of owner furnished equipment.
- Construction change order contingency.
- Overtime.
- Hazardous material abatement/removal.
- Items referenced as NOT INCLUDED or NIC in estimate.

The midpoint of construction of September 2023 is based on:

- Construction start date of June 2023
- Estimated construction duration of 06 months

- This estimate is based on a Design-Bid-Build delivery method.
- This estimate is based on prevailing wage labor rates.
- This estimate is based on a detailed measurement of quantities. We have made allowances for items that were not clearly defined in the drawings. The client should verify these allowances.
- This estimate is based on a minimum of four competitive bids and a stable bidding market.
- This estimate should be updated if more definitive information becomes available, or if there is any change in scope.
- We strongly advise the client to review this estimate in detail. If any interpretations in this estimate appear to differ from those intended by the design documents, they should be addressed immediately.

PROJECT SUMMARY

ELEMENT	TOTAL COST	GFA	$/SF AREA
01. STRUCTURAL RETROFIT	$657,832	6,200	$106.10
TOTAL CONSTRUCTION COST	**$657,832**		

SCHEMATIC RETROFIT

OCMI JOB #: 220665.000 | 31 October 2022

DETAILED PROJECT SUMMARY

ELEMENT		TOTAL COST	GFA	$/SF AREA
01. STRUCTURAL RETROFIT		$292,001	6,200	$47.10
TOTAL NET DIRECT COST		**$292,001**		
GENERAL MARKUPS				
DESIGN CONTINGENCY	20.00%	$58,400		
ESCALATION TO MIDPOINT 09/2023	6.42%	$22,484		
CONTRACTOR CONTINGENCY	7.00%	$26,102		
GENERAL CONDITIONS/REQUIREMENTS	47.50%	$189,519		
CONTRACTOR OVERHEAD AND PROFIT	8.00%	$47,080		
INSURANCE	3.50%	$22,246		
TOTAL CONSTRUCTION COST		**$657,832**		

BUILDING SUMMARY

ELEMENT		TOTAL COST	$/SF AREA
A. SUBSTRUCTURE		$35,713	$5.76
B. SHELL		$168,393	$27.16
C. INTERIORS		$59,845	$9.65
D. SERVICES		$9,743	$1.57
E. EQUIPMENT AND FURNISHINGS			
F. SPECIAL CONSTRUCTION AND DEMOLITION		$18,307	$2.95
G. BUILDING SITEWORK			
NET DIRECT BUILDING COST		$292,001	$47.10
ESCALATION TO MIDPOINT 09/2023	6.42%	$22,484	$3.63
TOTAL BUILDING COST		**$657,832**	**$106.10**

GROSS FLOOR AREA: 6,200 SF

SCHEMATIC RETROFIT

OCMI JOB #: 220665.000 | 31 October 2022

DETAILED SUMMARY

ELEMENT		TOTAL COST	$/SF AREA
A10 FOUNDATIONS		$35,713	$5.76
A20 BASEMENT CONSTRUCTION			
B10 SUPERSTRUCTURE			
B20 EXTERIOR ENCLOSURE		$168,393	$27.16
B30 ROOFING			
C10 INTERIOR CONSTRUCTION		$44,448	$7.17
C20 STAIRS			
C30 INTERIOR FINISHES		$15,397	$2.48
D10 CONVEYING			
D20 PLUMBING			
D30 HVAC			
D40 FIRE PROTECTION			
D50 ELECTRICAL		$9,743	$1.57
E10 EQUIPMENT			
E20 FURNISHINGS			
F10 SPECIAL CONSTRUCTION			
F20 SELECTIVE BUILDING DEMOLITION		$18,307	$2.95
G10 SITE PREPARATION			
G20 SITE IMPROVEMENTS			
G30 SITE MECHANICAL UTILITIES			
G40 SITE ELECTRICAL UTILITIES			
G90 OTHER SITE CONSTRUCTION			
NET DIRECT BUILDING COST		$292,001	$47.10
ESCALATION TO MIDPOINT 09/2023	6.42%	$22,484	$3.63
TOTAL BUILDING COST		**$657,832**	**$106.10**

SCHEMATIC RETROFIT

OCMI JOB #: 220665.000 | 31 October 2022

DESCRIPTION	QUANTITY	UNIT	UNIT RATE	ESTIMATED COST
A. SUBSTRUCTURE				
A10 FOUNDATIONS				
A1010 STANDARD FOUNDATIONS				**$16,009**
A1030 SLAB ON GRADE				**$19,704**
				$35,713
A. SUBSTRUCTURE TOTAL				**$35,713**
B. SHELL				
B20 EXTERIOR ENCLOSURE				
B2010 EXTERIOR WALLS				**$168,393**
				$168,393
B. SHELL TOTAL				**$168,393**
C. INTERIORS				
C10 INTERIOR CONSTRUCTION				
C1010 PARTITIONS				**$44,448**
				$44,448
C30 INTERIOR FINISHES				
C3010 WALL FINISHES				**$5,721**
C3020 FLOOR FINISHES				**$4,531**
C3030 CEILING FINISHES				**$5,145**
				$15,397
C. INTERIORS TOTAL				**$59,845**
D. SERVICES				
D50 ELECTRICAL				
D5010 ELECTRICAL SERVICE AND DISTRIBUTION				**$9,743**
				$9,743
D. SERVICES TOTAL				**$9,743**
F. SPECIAL CONSTRUCTION AND DEMOLITION				
F20 SELECTIVE BUILDING DEMOLITION				
F2010 BUILDING ELEMENTS DEMOLITION				**$18,307**
F2020 HAZARDOUS COMPONENTS ABATEMENT				
				$18,307
F. SPECIAL CONSTRUCTION AND DEMOLITION TOTAL				**$18,307**

SCHEMATIC RETROFIT

OCMI JOB #: 220665.000 | 31 October 2022

DESCRIPTION	QUANTITY	UNIT	UNIT RATE	ESTIMATED COST
A. SUBSTRUCTURE				
A10 FOUNDATIONS				
A1010 STANDARD FOUNDATIONS				
Engineered fill, backfill and compact	1	LS	2,014.85	$2,015
Continuous footings, assembly	8	CY	1,717.50	$13,994
				$16,009
A1030 SLAB ON GRADE				
Reinforced concrete slab, 4" thick	259	SF	18.16	$4,708
Dowel into existing slab	108	EA	138.31	$14,996
				$19,704
A10 FOUNDATIONS TOTAL				**$35,713**
A. SUBSTRUCTURE TOTAL				**$35,713**
B. SHELL				
B20 EXTERIOR ENCLOSURE				
B2010 EXTERIOR WALLS				
Steel plates full height of columns	10	TON	13,548.65	$133,930
Miscellaneous angles and channels	1.5	TON	20,055.53	$29,738
Tension rod assemblies	0.24	TON	20,055.53	$4,725
				$168,393
B20 EXTERIOR CLOSURE TOTAL				**$168,393**
B. SHELL TOTAL				**$168,393**
C. INTERIORS				
C10 INTERIOR CONSTRUCTION				
C1010 PARTITIONS				
New shear walls				
Metal stud framing, 6"	373	SF	18.51	$6,907
Gypsum board, taped and finished	746	SF	5.65	$4,218
Connection to moment frame and footing, Allow	40	LF	150.00	$6,000
Wall sheathing	746	SF	4.87	$3,634
Replace demo'd walls				
Metal stud framing, 6"	576	SF	18.51	$10,660
Furring	456	SF	10.09	$4,599
Batt insulation, 6"	576	SF	2.75	$1,582
Gypsum board, taped and finished	1,212	SF	5.65	$6,848

SCHEMATIC RETROFIT

OCMI JOB #: 220665.000 | 31 October 2022

DESCRIPTION	QUANTITY	UNIT	UNIT RATE	ESTIMATED COST
				$44,448
C10 INTERIOR CONSTRUCTION TOTAL				**$44,448**
C30 INTERIOR FINISHES				
C3010 WALL FINISHES				
Paint	1,958	SF	2.41	$4,721
Repair damaged finishes, Allow.	1	LS	1,000.00	$1,000
				$5,721
C3020 FLOOR FINISHES				
Concrete, sealer	259	SF	2.36	$612
Floor finishes, Allow	259	SF	9.22	$2,391
Base				
Rubber	124	LF	4.26	$528
Repair damaged finishes, Allow.	1	LS	1,000.00	$1,000
				$4,531
C3030 CEILING FINISHES				
Reinstall existing ACT ceiling	710	SF	5.84	$4,145
Repair damaged finishes, Allow.	1	LS	1,000.00	$1,000
				$5,145
C30 INTERIOR FINISHES TOTAL				**$15,397**
C. INTERIORS TOTAL				**$59,845**
D. SERVICES				
D50 ELECTRICAL				
D5010 ELECTRICAL SERVICE AND DISTRIBUTION				
Convenience power				
Receptacles				
Duplex	8	EA	182.40	$1,459
Junction box	2	EA	91.40	$183
Conduit and wire	420	LF	18.24	$7,661
Demolition				
Electrical outlets	8	LS	55.00	$440
				$9,743

SCHEMATIC RETROFIT

OCMI JOB #: 220665.000 | 31 October 2022

DESCRIPTION	QUANTITY	UNIT	UNIT RATE	ESTIMATED COST
D50 ELECTRICAL TOTAL				**$9,743**
D. SERVICES TOTAL				**$9,743**
F. SPECIAL CONSTRUCTION AND DEMOLITION				
F20 SELECTIVE BUILDING DEMOLITION				
F2010 BUILDING ELEMENTS DEMOLITION				
Sawcut slab on grade	163	LF	15.19	$2,470
Remove slab on grade	259	SF	20.03	$5,192
Excavate footings	11	CY	97.86	$1,087
ACT, remove grid and salvage tile	710	SF	2.88	$2,044
Interior partition	576	SF	4.55	$2,623
Remove column furring	456	SF	5.55	$2,532
Remove existing frame bracing, Allow.	1	LS	2,000.00	$2,000
Haul	15%	LS	2,392.20	$359
				$18,307
F20 SELECTIVE BUILDING DEMOLITION TOTALS				**$18,307**
F. SPECIAL CONSTRUCTION AND DEMOLITION TOTAL				**$18,307**

APPENDIX F – STRUCTURAL CALCULATIONS

DETAILED DESIGN CRITERIA

BUILDING CODE

Governing Code:	2019 California Building Code
Authority Having Jurisdiction:	City of Santa Rosa

SEOR STAMP

BUILDING SYSTEM DESCRIPTION

Date of Construction:	1979
No. Stories:	1
Floor Area:	6000 ft^2
Roof Area:	7344 ft^2
Mean Building Height:	13.5 ft

SEISMIC DESIGN PARAMETERS

ASCE 41-17 Reference UNO:

Latitude:	38.442 deg	Longitude: -122.750 deg	
Soil Site Class =	D	Per Geotech Report, Site Class D otherwise	*USGS*
Risk Category:	IV		*ASCE 7 Table 1.5-1*
Diaphragm=	Flexible Diaphragm		
Building System, N-S:	S3	Steel Light Frame	*Table 3-1*
Building System, E-W:	S3	Steel Light Frame	*Table 3-1*
$C_{t, N\text{-}S}$ =	0.02	Approximate Period Parameter, Ct, N-S	*Section 4.4.2.4*
$C_{t, E\text{-}W}$ =	0.02	Approximate Period Parameter, Ct, E-W	*Section 4.4.2.4*
$\beta_{N\text{-}S}$ =	0.75	Approximate Period Parameter, β, N-S	*Section 4.4.2.4*
$\beta_{E\text{-}W}$ =	0.75	Approximate Period Parameter, β, E-W	*Section 4.4.2.4*
$T_{a, N\text{-}S}$=	0.141 sec	Approximate Fundamental Period, N-S	*Section 4.4.2.4*
$T_{a, E\text{-}W}$ =	0.141 sec	Approximate Fundamental Period, E-W	*Section 4.4.2.4*
T_L =	8.000 sec	Long Period Transition Period	*ASCE 7 Section 11.4.5*
Seismicity:	High		*Table 2-5*

TIER 1 SEISMIC EVALUATION PARAMETERS

Seismic Hazard Level:	BSE-1N	ASCE 7 DBE, 10%/50 years (475 year mean return)	
Performance Objective:	IO	S-1: Immediate Occupancy	*Table 2-1 & 2-2*
S_S =	1.258 g	Mapped spectral response acceleration parameter	*USGS*
S_1 =	0.481 g	Mapped spectral response acceleration parameter	*USGS*
S_{XS} =	1.258 g	Mapped spectral response acceleration parameter	*USGS*
S_{X1} =	0.817 g	Mapped spectral response acceleration parameter	*USGS*
$S_{a, N\text{-}S}$ =	1.258 g	Spectral Response Acceleration, N-S	*Section 4.4.2.3*
$S_{a, E\text{-}W}$ =	1.258 g	Spectral Response Acceleration, E-W	*Section 4.4.2.3*
$C_{N\text{-}S}$ =	1.300	Modification Factor	*Table 4-8*
$C_{E\text{-}W}$ =	1.300	Modification Factor	*Table 4-8*
$V_{N\text{-}S}$ =	1.635 *W	Pseudo-Seismic Base Shear, N-S	*Section 4.4.2.1*
$V_{E\text{-}W}$ =	1.635 *W	Pseudo-Seismic Base Shear, E-W	*Section 4.4.2.1*

DETAILED DESIGN CRITERIA

TIER 2 / 3 SEISMIC EVALUATION PARAMETERS

Seismic Hazard Level:	BSE-1N	ASCE 7 DBE, 10%/50 years (475 year mean return)	
Performance Objective:	IO	S-1: Immediate Occupancy	*Table 2-1 & 2-2*
S_S =	1.258 g	Mapped spectral response acceleration parameter	*USGS*
S_1 =	0.481 g	Mapped spectral response acceleration parameter	*USGS*
S_{XS} =	1.258 g	Mapped spectral response acceleration parameter	*USGS*
S_{X1} =	0.817 g	Mapped spectral response acceleration parameter	*USGS*
$C_{1, N-S}$ =	1.000	Inelastic-to-elastic displacement factor	*Equation 7-22*
$C_{2, N-S}$ =	1.000	Hysteresis shape factor	*Equation 7-23*
Alternate $(C_1C_2)_{N-S}$ =	1.400	2 ≤ mmax < 6	*Table 7-3*
Use Alternate $(C_1C_2)_{N-S}$?	Yes		
$(C_1C_2)_{N-S}$ =	1.400		
$C_{m, N-S}$ =	1.000	Effective mass factor	*Table 7-4*
$C_{1, E-W}$ =	1.000	Inelastic-to-elastic displacement factor	*Equation 7-22*
$C_{2, E-W}$ =	1.000	Hysteresis shape factor	*Equation 7-23*
Alternate $(C_1C_2)_{E-W}$ =	1.400	2 ≤ mmax < 6	*Table 7-3*
Use Alternate $(C_1C_2)_{E-W}$?	Yes		
$(C_1C_2)_{E-W}$ =	1.400		
$C_{m, E-W}$ =	1.000	Effective mass factor	*Table 7-4*
$S_{a,N-S}$ =	1.255	Spectral Response Acceleration	*Section 2.4.1.7*
$S_{a,E-W}$ =	1.255	Spectral Response Acceleration	*Section 2.4.1.7*
V_{N-S} =	1.757 *W	Pseudo-Seismic Base Shear, N-S	*Equation 7-21*
V_{E-W} =	1.757 *W	Pseudo-Seismic Base Shear, E-W	*Equation 7-21*

TIER 2 / 3 SEISMIC EVALUATION PARAMETERS (HAZARD LEVEL B)

Seismic Hazard Level:	BSE-2N	ASCE 7 MCE	
Performance Objective:	LS	S-3: Life Safety	*Table 2-1 & 2-2*
S_S =	1.887 g	Mapped spectral response acceleration parameter	*USGS*
S_1 =	0.721 g	Mapped spectral response acceleration parameter	*USGS*
S_{XS} =	1.887 g	Mapped spectral response acceleration parameter	*USGS*
S_{X1} =	1.225 g	Mapped spectral response acceleration parameter	*USGS*
$C_{1, N-S}$ =	1.000	Inelastic-to-elastic displacement factor	*Equation 7-22*
$C_{2, N-S}$ =	1.000	Hysteresis shape factor	*Equation 7-23*
Alternate $(C_1C_2)_{N-S}$ =	1.400	2 ≤ mmax < 6	*Table 7-3*
Use Alternate $(C_1C_2)_{N-S}$?	Yes		
$(C_1C_2)_{N-S}$ =	1.400		
$C_{m, N-S}$ =	1.000	Effective mass factor	*Table 7-4*
$C_{1, E-W}$ =	1.000	Inelastic-to-elastic displacement factor	*Equation 7-22*
$C_{2, E-W}$ =	1.000	Hysteresis shape factor	*Equation 7-23*
Alternate $(C_1C_2)_{E-W}$ =	1.400	2 ≤ mmax < 6	*Table 7-3*
Use Alternate $(C_1C_2)_{E-W}$?	Yes		
$(C_1C_2)_{E-W}$ =	1.400		
$C_{m, E-W}$ =	1.000	Effective mass factor	*Table 7-4*
$S_{a,N-S}$ =	1.883	Spectral Response Acceleration	*Section 2.4.1.7*
$S_{a,E-W}$ =	1.883	Spectral Response Acceleration	*Section 2.4.1.7*
V_{N-S} =	2.636 *W	Pseudo-Seismic Base Shear, N-S	*Equation 7-21*
V_{E-W} =	2.636 *W	Pseudo-Seismic Base Shear, E-W	*Equation 7-21*

MATERIAL STRENGTH AND SPECIFICATIONS

ASCE 41-17 Reference UNO:

CONCRETE:

Knowledge Factor, κ	0.75	Concrete Knowledge Factor	*Table 6-1*
Foundations, f'_c =	3000 psi	Default Lower Bound: 3000 psi - 4000 psi	*Table 10-2*
Foundations, f'_{ce} =	4500 psi		*Table 10-1*
Slab on grade, f'_c =	3000 psi	Default Lower Bound: 3000 psi - 5000 psi	*Table 10-2*
Slab on grade, f'_{ce} =	4500 psi		*Table 10-1*

DETAILED DESIGN CRITERIA

CONCRETE REINFORCING:

Knowledge Factor, κ	0.75	Reinforcing Knowledge Factor	*Table 6-1*
Reinforcing Steel, f_y =	60 ksi	Default Lower-Bound: 40, 50, 60, 65, 70 ksi	*Table 10-3*
Reinforcing Steel, f_{ye} =	75 ksi		*Table 10-1*
Reinforcing Steel, f_y =	90 ksi	Default Lower-Bound: 70, 80, 90, 75, 80 ksi	*Table 10-3*
Reinforcing Steel, f_{ye} =	112.5 ksi		*Table 10-1*
Reinforcing Steel ties, f_y =	40 ksi	Default Lower-Bound: 40, 50, 60, 65, 70 ksi	*Table 10-3*
Reinforcing Steel ties, f_{ye} =	50 ksi		*Table 10-1*
Reinforcing Steel ties, f_y =	70 ksi	Default Lower-Bound: 70, 80, 90, 75, 80 ksi	*Table 10-3*
Reinforcing Steel ties, f_{ye} =	87.5 ksi		*Table 10-1*

STRUCTURAL STEEL:

Knowledge Factor, κ	0.75	Structural Steel Knowledge Factor	*Table 6-1*
W-Shapes, f_y =	44.0 ksi	(1961-1990) A36 Group 1	*Table 9-1*
W-Shapes, f_u =	62.0 ksi		*Table 9-1*
W-Shapes, f_{ye} =	48.4 ksi		*Table 9-3*
W-Shapes, f_{ue} =	68.2 ksi		*Table 9-3*
Angles and channels, f_y =	44.0 ksi	(1961-1990) A36 Group 1	*Table 9-1*
Angles and channels, f_u =	62.0 ksi		*Table 9-1*
Angles and channels, f_{ye} =	48.4 ksi		*Table 9-3*
Angles and channels, f_{ue} =	68.2 ksi		*Table 9-3*
Typical plates, f_y =	44.0 ksi	(1961-1990) A36 Group 1	*Table 9-1*
Typical plates, f_u =	62.0 ksi		*Table 9-1*
Typical plates, f_{ye} =	48.4 ksi		*Table 9-3*
Typical plates, f_{ue} =	68.2 ksi		*Table 9-3*
Base plates & MF plates, f_y =	44.0 ksi	(1961-1990) A36 Group 1	*Table 9-1*
Base plates & MF plates, f_u =	62.0 ksi		*Table 9-1*
Base plates & MF plates, f_{ye} =	48.4 ksi		*Table 9-3*
Base plates & MF plates, f_{ue} =	68.2 ksi		*Table 9-3*
Gusset plates, f_y =	44.0 ksi	(1961-1990) A36 Group 1	*Table 9-1*
Gusset plates, f_u =	62.0 ksi		*Table 9-1*
Gusset plates, f_{ye} =	48.4 ksi		*Table 9-3*
Gusset plates, f_{ue} =	68.2 ksi		*Table 9-3*

STEEL CONNECTORS:

Knowledge factor, κ	0.75	Steel Connectors Knowledge Factor	*Table 6-1*
Shear stud connectors, f_u =	65 ksi	ASTM A108	
Machine Bolts, f_u =	58 ksi	ASTM A307	
High Strength Bolts, f_u =	120 ksi	ASTM A325	
Anchor Bolts, f_y =	36 ksi	ASTM F1554, Grade 36 or ASTM A307	
Anchor Bolts, f_u =	58 ksi	ASTM F1554, Grade 36 or ASTM A307	
Threaded Rods, f_y =	36 ksi	ASTM F1554, Grade 36 or ASTM A307	
Threaded Rods, f_u =	58 ksi	ASTM F1554, Grade 36 or ASTM A307	
Weld F_{EXX} =	70 ksi	Weld Strength	

COLD FORMED METAL FRAMING:

Knowledge Factor, κ	0.75	Reinforcing Knowledge Factor	*Table 6-1*
33 & 43 mils (20 & 18 ga), fy =	33 ksi	ASTM A1003, Grade ST33H or ST33L	
54, 68, & 97 mils (16, 14, & 12 ga), fy =	50 ksi	ASTM A1003, Grade ST50H or ST50L	

WEIGHT TAKEOFF (PSF)

EXISTING ROOF

CBC Live Load Category: 26. Roof: ordinary [Table 1607.1]

Material		Thickness (in)	Deck	Joists	Girders	Seismic
Roofing:	Cap Sheet		3.0	3.0	3.0	3.0
Decking:	Metal Deck		1.5	1.5	1.5	1.5
Insulation:	Rigid		1.0	1.0	1.0	1.0
Framing:	Z-Purlins @ 5'-0"oc		2.0	2.0	2.0	2.0
Framing:	Steel Beams @ 20'-0"oc				1.0	1.0
MEP:	Typical			2.0	2.0	2.0
Miscellaneous			2.5	1.5	1.5	1.5
Dead Load			**10.0**	**11.0**	**12.0**	**12.0**
Live Load			**20.0**	**20.0**	**20.0**	**0.0**
Total Load			**30.0**	**31.0**	**32.0**	**12.0**

EXISTING EXTERIOR WALLS

CFS Infill Between Steel Columns	**10.0**

Seismic Weight Takeoff

Level	Diaphragm Surfice Wt (psf)	Diaphragm Area (ft^2)	Diaphragm Weight (kips)	Exterior Walls Surfice Wt (psf)	Exterior Walls Trib (ft)	Exterior Walls Length (ft)	Exterior Walls Weight (kips)	Level Weight (kips)
Roof	12	7344	88	10	4.6	320	15	**103**
		Σ	88			Σ	15	**103**

SEISMIC HAZARD ANALYSIS (Tier 1)

ASCE 41-17 §2.4

Site Coordinates

Latitude =	38.4424	deg	55 Stony Point Assessment
Longitude =	-122.7496	deg	Santa Rosa, California

Spectral Response Acceleration Parameters

ASCE 41-17 §2.4.1.4

Site Class =	D		Site Soil Classification
S_S =	1.258	g	Mapped Short-period Spectral Response Acceleration
S_1 =	0.481	g	Mapped 1-sec period Spectral Response Acceleration
S_{XS} =	1.258	g	Short-period Spectral Response Acceleration at BSE-1N
S_{X1} =	0.817	g	1-sec period Spectral Response Acceleration at BSE-1N

SEISMIC FORCE

ASCE 41-17 §4.5.2

Building Properties

Type =	S3		Building Type	*ASCE 41-17 Table 3-1*
Height, h_n =	13.50	ft	Height above base to roof level	
Stories =	1		Number of stories	
Weight =	102.9	k	Seismic Weight of Building	

Building Period

ASCE 41-17 §4.5.2.4

C_t =	0.02		Period Adjustment Factor	
β =	0.75		Empirical Fundamental Period Adjustment Factor	
T =	0.141	sec	Fundamental Period	$=C_t{*}h_n^{\beta}$

Pseudo-Seismic Force

ASCE 41-17 §4.5.2.1

S_a =	1.258	g	Spectral Response Acceleration	$=S_{x1}/T < S_{xs}$
C =	1.30		Modification Factor	*Table 4-7*
V =	**1.64**	***W**	**Pseudo-Seismic Force in Terms of Weight**	*=C*Sa*W*
V =	**168.3**	**k**	**Pseudo-Seismic Force**	

SEISMIC HAZARD ANALYSIS (Tier 2)

ASCE 41-17 §2.4.1

Site Coordinates

Latitude =	38.442	deg	55 Stony Point Assessment
Longitude =	-122.750	deg	Santa Rosa, California

Spectral Response Acceleration Parameters

ASCE 41-17 §2.4.1.4

Site Class =	D		Site Soil Classification
T_L =	8.00	sec	Long Period Transition Period
Hazard Level:	BSE-1N		
S_S =	1.258	g	Mapped Short-period Spectral Response Acceleration
S_1 =	0.481	g	Mapped 1-sec period Spectral Response Acceleration
S_{XS} =	1.258	g	Short-period Spectral Response Acceleration at BSE-1N
S_{X1} =	0.817	g	1-sec period Spectral Response Acceleration at BSE-1N
Hazard Level:	BSE-2N		
S_S =	1.887	g	Mapped Short-period Spectral Response Acceleration
S_1 =	0.721	g	Mapped 1-sec period Spectral Response Acceleration
S_{XS} =	1.887	g	Short-period Spectral Response Acceleration at BSE-2N
S_{X1} =	1.225	g	1-sec period Spectral Response Acceleration at BSE-2N

General Horizontal Response Spectrum

ASCE 41-17 §2.4.1.7

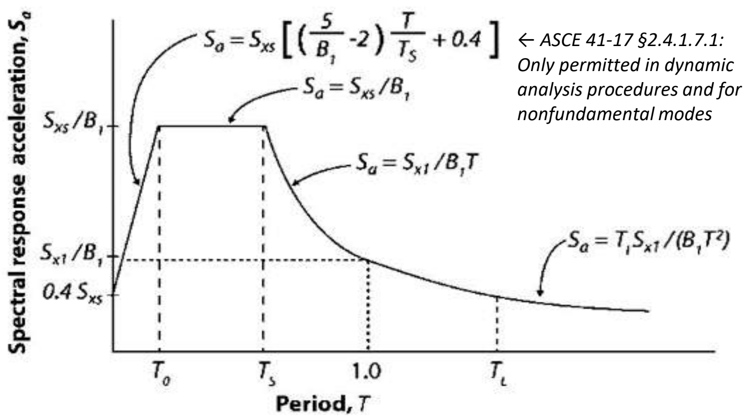
β =	5%		Effective Viscous Damping Ratio	
B_1 =	1.00		Damping Coefficient	*= 4 / (5.6 - ln(100β))*
Hazard Level:	BSE-1N			
T_S =	0.65	sec	Period at Constant Velocity Region	$= S_{X1} / S_{XS}$
T_0 =	0.13	sec	Period at Constant Acceleration Region	$= 0.2\ T_S$
$0.4S_{XS}$=	0.50	g	Peak Ground Acceleration	
S_{XS}/B_1 =	1.26	g	Short period Spectral Response Acceleration	
S_{X1}/B_1 =	0.81	g	1-sec period Design Spectral Response Acceleration	
Hazard Level:	BSE-2N			
T_S =	0.65	sec	Period at Constant Velocity Region	$= S_{X1} / S_{XS}$
T_0 =	0.13	sec	Period at Constant Acceleration Region	$= 0.2\ T_S$
$0.4S_{XS}$=	0.75	g	Peak Ground Acceleration	
S_{XS}/B_1 =	1.88	g	Short period Spectral Response Acceleration	
S_{X1}/B_1 =	1.22	g	1-sec period Design Spectral Response Acceleration	

SEISMIC HAZARD ANALYSIS (Tier 2)

ASCE 41-17 §2.4.1

Spectral Acceleration at Building Period

ASCE 41-17 §2.4



ASCE 41-17 Figure 2-1: General Horizontal Response Spectrum

T =	0.141	sec	Building period	*Per LSP calcs*
S_a =	**1.255**	g	**BSE-1N Spectral Acceleration at Building Period**	
S_a =	**1.883**	g	**BSE-2N Spectral Acceleration at Building Period**	

LINEAR STATIC PROCEDURE (LSP) — *ASCE 41-17 §7.4.1*

# Stories =	1		Number of stories in building	

Period Determination for LSP - Method 2 - Empirical — *ASCE 41-17 §7.4.1.2*

Building Type =	S3		Steel Light Frame	
C_t =	0.020		Factor for adjustment of period	*ASCE 41-17 §7.4.1.2.2*
β =	0.75		Factor for adjustment of period	
h_n =	13.5	ft	Roof Height	
T =	0.141	sec	Building period in N-S direction	*ASCE 41-17 Eq. 7-18*

Pseudo-Seismic Force for LSP — *ASCE 41-17 §7.4.1.3.1*

$V = C_1 C_2 C_m S_a W$			Pseudo-Lateral Force	*ASCE 41-17 Eq. 7-21*
W =	102.9	kips	Effective Seismic Weight	

Hazard = BSE-1N, Performance Objective = S-1: Immediate Occupancy

C_1 =	1.00		Modification Factor, Inelastic Displacements	*ASCE 41-17 Eq. 7-22*
C_2 =	1.00		Modification Factor, Cyclic Behavior	*ASCE 41-17 Eq. 7-23*
$C_1 C_2$ =	1.40		Alternative Value for Modification Factors	*ASCE 41-17 Table 7-3*
Use alternate C_1C_2?	Yes			
C_m =	1.0		Effective Mass Factor	*ASCE 41-17 Table 7-4*
$S_a(T)$ =	1.255	g	Spectral Response Acceleration	*for T = 0.14sec*
$V_{N/S}$ =	**1.757**	***W**	**Pseudo-Lateral Force**	
$V_{N/S}$ =	**180.8**	**kips**	**Pseudo-Lateral Force**	

Hazard = BSE-2N, Performance Objective = S-3: Life Safety

C_1 =	1.00		Modification Factor, Inelastic Displacements	*ASCE 41-17 Eq. 7-22*
C_2 =	1.00		Modification Factor, Cyclic Behavior	*ASCE 41-17 Eq. 7-23*
$C_1 C_2$ =	1.40		Alternative Value for Modification Factors	*ASCE 41-17 Table 7-3*
Use alternate C_1C_2?	Yes			
C_m =	1.0		Effective Mass Factor	*ASCE 41-17 Table 7-4*
$S_a(T)$ =	**1.883**	g	Spectral Response Acceleration	*for T = 0.14sec*
$V_{N/S}$ =	**2.636**	*W	Pseudo-Lateral Force	
$V_{N/S}$ =	**271.3**	kips	Pseudo-Lateral Force	

Tier 1

BRACE AXIAL STRESS CHECK: The axial stress in the diagonals, calculated using the Quick Check procedure of Section 4. 4. 3.4, is less than $0.50F_y$. (Commentary: Sec. A.3.3.1.2. Tier 2: Sec. 5.5.4.1)

4.4.3.4 *Diagonal Bracing.* The average axial stress in diagonal bracing elements, f_j^{avg}, shall be calculated in accordance with Eq. (4-9).

$$f_j^{avg} = \frac{1}{M_s}\left(\frac{V_j}{sN_{br}}\right)\left(\frac{L_{br}}{A_{br}}\right) \quad (4\text{-}9)$$

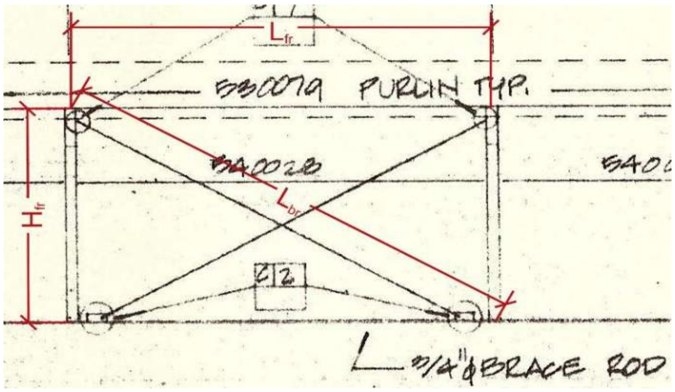
where

L_{br} = Average length of the braces (ft);
N_{br} = Number of braces in tension and compression if the braces are designed for compression, number of diagonal braces in tension if the braces are designed for tension only;
s = Average span length of braced spans (ft);
A_{br} = Average area of a diagonal brace (in.^{2});
V_j = Maximum story shear at each level (kip); and
M_s = System modification factor; M_s shall be taken from Table 4-9.

Table 4-9. M_s Factors for Diagonal Braces

		Level of Performance		
Brace Type	**d/t[b]**	**CP[a]**	**LS[a]**	**IO[a]**
Tube[b]	$<90/(F_{ye})^{1/2}$	7.0	4.5	2.0
	$>190/(F_{ye})^{1/2}$	3.5	2.5	1.25
Pipe[c]	$<1{,}500/F_{ye}$	7.0	4.5	2.0
	$>6{,}000/F_{ye}$	3.5	2.5	1.25
Tension-only		3.5	2.5	1.25
Cold-formed steel strap-braced wall		3.5	2.5	1.25
All others		7.0	4.5	2.0

Note: $F_{ye} = 1.25F_y$; expected yield stress.
[a] CP = Collapse Prevention, LS = Life Safety, IO = Immediate Occupancy.
[b] Depth-to-thickness ratio.
[c] Interpolation to be used for tubes and pipes.



Frame Length (L_{fr}) = 20.00 ft
Frame Height (H_{fr}) = 9.25 ft
L_{br} = 22.04 ft
N_{br} = 4 braces
s = 20.00 ft
Brace Diameter (D_{br}) = 0.75 in
A_{br} = 0.44 in^2
V_j = 168 kips
M_S = 1.25 (tension only)
f_j^{avg} = 84.0 ksi

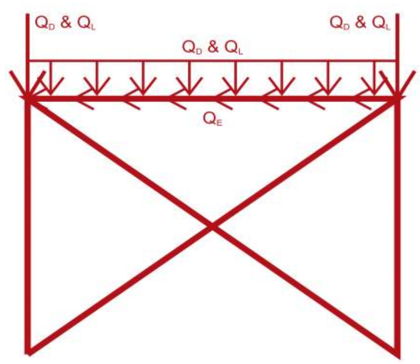
$0.50F_y$ = 18.0 ksi

Acceptance Ratio = 4.66 > 1.0, therefore NC

Tier 2

<u>Braced Frame RISA Model Loading</u>

Trib Beam Length (TL_B) = 30 ft
Trib Beam Width (TW_B) = 20 ft
Trib Purlin Width (TW_p) = 5 ft
Building Length (BL) = 100 ft
Building Width (BW) = 60 ft
Frame Width (w) = 20 ft
Frame Height (H) = 13 ft
Number of Frames (n) = 2 frames



Vertical Gravity Loads

Roof Dead Load (DL) =	12	psf
Roof Live Load (RLL) =	20	psf
RLL Reduction (R_1) =	0.60	
Q_D = DL(TL_B)(TW_B) =	**7200**	**lb**
Q_L = RLL(R_1)(TL_B)(TW_B) =	**7200**	**lb**
Q_D = DL(TW_p) =	**60**	**plf**
Q_L = RLL(R_1)(TW_p) =	**60**	**plf**

Horizontal Lateral Loads

Base Shear (V_1) =	1.76	W
Base Shear (V_2) =	2.64	W
Roof Weight (w_R) =	12	psf
Roof Weight (W_R) =	36.0	k
Wall Weight (w_W) =	10	psf
Wall Weight (W_W) =	10.4	k
Seismic Weight (W) =	1160	plf
$Q_{E1} = V_1 W$ =	**2038**	**plf**
$Q_{E2} = V_2 W$ =	**3057**	**plf**

Braces *Deformation Controlled per ASCE 41-17 §9.5.2.4.1*

According to §9.5.2.3.2 determine strength per §9.4.2.3.2…

The tension strength of structural steel components shall be the lowest value obtained from the limit states of yielding in the gross section or rupture in the net section. The tension strength shall be determined using equations for nominal strength, P_n, given in AISC 360, Chapter D, except that F_{ye} shall be substituted for F_y if the component is deformation controlled, $Q_{CE} = Q_y = T_{CE}$; or F_{yLB} shall be substituted for F_y if the component is force controlled, $Q_{CL} = T_{CL}$.

AISC 360-16 §D.2 TENSILE STRENGTH

Φ =	1.00		ASCE 41-17 §9.5.2.3.2
Lower Bound Strength (F_{yLB}) =	36	ksi	
Expected Strength Factor =	1.1		ASCE 41-17 Table 9-3
Expected Strength (F_{ye}) =	39.6	ksi	
Ultimate Strength (F_u) =	58	ksi	
Brace Diameter (D_{br}) =	0.75	in	
Gross Area (A_g) =	0.44	in^2	
Net Area (A_n) =	0.44	in^2	
Shear Lag Factor (U) =	1.0		
Effective Net Area (A_e) =	0.44	in^2	
Tensile Yielding in the Gross Section			
Design Tensile Strength (ΦP_{ny}) =	17.5	k	AISC 360-16 (D2-1)
Tensile Rupture in the Net Section			
Design Tensile Strength (ΦP_{nr}) =	25.6	k	AISC 360-16 (D2-2)

Expected Strength of Brace in Tension (Q_{CE}) =	**17.5**	**k**
Knowledge Factor (k) =	0.75	
IO m-Factor (m_{IO}) =	1.25	Table 9-6
LS m-Factor (m_{LS}) =	5	Table 9-6
$m_{IO}kQ_{CE}$ =	**16.4**	**k**
$m_{LS}kQ_{CE}$ =	**65.6**	**k**

Deformation Controlled Action per RISA (Q_{UD1}) =	**46.9**	**k**
Deformation Controlled Action per RISA (Q_{UD2}) =	**70.3**	**k**

Acceptance Ratio = $Q_{UD1}/(m_{IO}kQ_{CE})$ = 2.86 > 1.0, therefore FAILS

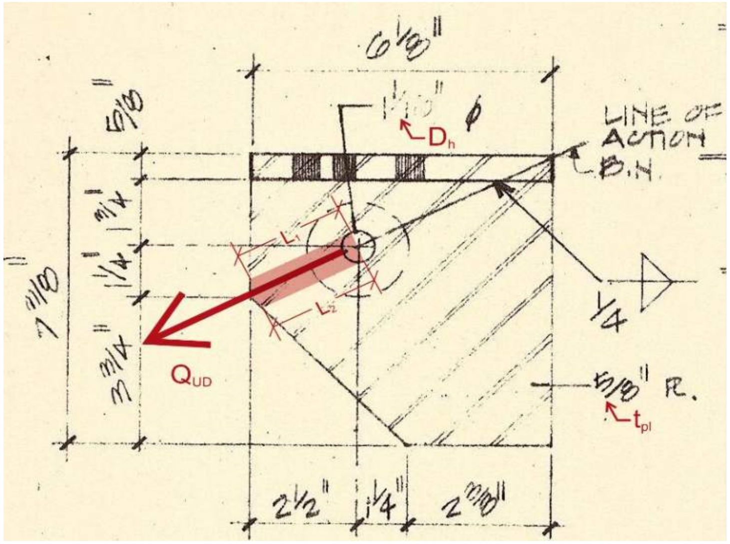
Acceptance Ratio = $Q_{UD2}/(m_{LS}kQ_{CE})$ = 1.07 > 1.0, therefore FAILS

NOTE: Brace fails, assume Clevis fails by observation. New Clevis would be required for new brace size anyway.

Connections *Force Controlled per ASCE 41-17 §9.5.2.4.1*

According to §9.5.2.3.2 determine strength per AISC 360 & the *Steel Construction Manual …*

AISC 360-16 §J4.3 - STRENGTH OF ELEMENTS IN SHEAR



NOTE:
1/4in fillet weld each side at top and 3/8in filled weld each side at side OK by observation.

Φ = 1.00 ASCE 41-17 §9.5.2.3.2

Lower Bound Strength (F_{yLB}) = 36 ksi

Ultimate Strength (F_u) = 58 ksi

Plate Thickness (t_{pl}) = 0.625 in

Hole Diameter (D_h) = 1.06 in

Side Length (L_1) = 2.53 in

Side Length (L_2) = 2.62 in

Gross Area (A_{gv}) = 3.22 in^2 $= t_{pl}(L_1+L_2)$

Net Area (A_{nv}) = 2.89 in^2 $= t_{pl}(L_1+L_2-2(D_h/4))$

Shear Yielding of the Element

$\Phi R_n = \Phi 0.6 F_{yLB} A_{gv}$ = 69.5 k AISC 360-16 (J4-3)

Shear Rupture of the Element

$\Phi R_n = \Phi 0.6 F_u A_{nv}$ = 100 k AISC 360-16 (J4-4)

Lower-Bound Strength of Plate (Q_{CL}) = 69.5 k

Knowledge Factor (k) = 0.75

kQ_{CL} = 52.1 k

Seismic Action per RISA (Q_E) = 46.9 k

IO: X = 1.3 C_1C_2 = 1.40 J = 1

Force Controlled Action ($Q_{UF(IO)}$) = 43.5 k

Acceptance Ratio = $Q_{UF(IO)}/(kQ_{CL})$ = 0.84 < 1.0, therefore OK

Seismic Action per RISA (Q_E) = 70.3 k

LS: X = 1.3 C_1C_2 = 1.40 J = 2

Force Controlled Action ($Q_{UF(LS)}$) = 32.7 k

Acceptance Ratio = $Q_{UF(LS)}/(kQ_{CL})$ = 0.63 < 1.0, therefore OK

Tier 1

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 Fy. (Commentary: Sec. A.3.1.3.3. Tier 2: Sec. 5.5.2.1.2)

4.4.3.9 Flexural Stress in Columns and Beams of Steel Moment Frames. The average flexural stress in the columns and beams of steel frames at each level shall be computed in accordance with Eq. (4-14).

$$f_j^{\text{avg}} = V_j \frac{1}{M_s}\left(\frac{n_c}{n_c - n_f}\right)\left(\frac{h}{2}\right)\frac{1}{Z} \qquad (4\text{-}14)$$

where

n_c = Total number of frame columns at the level, j, under consideration.
n_f = Total number of frames in the direction of loading at the level, j, under consideration.
V_j = Story shear computed in accordance with Section 4.4.2.2.
h = Story height (in.).
Z = For columns, the sum of the plastic section moduli of all the frame columns at the level under consideration. For beams, it is the sum of the plastic section moduli of all the frame beams with moment-resisting connections. If a beam has moment-resisting connections at both ends, then the contribution of that beam to the sum is twice the plastic section modulus of that beam (in^3).
M_s = System modification factor; M_s shall be taken as equal to 9.0 for buildings being evaluated to the Collapse Prevention Performance Level, equal to 6.0 for buildings being evaluated to the Life Safety Performance Level, and equal to 2.5 for buildings being evaluated to the Immediate Occupancy Performance Level for columns and beams satisfying the checklist items for compactness and column axial stress. If the columns or beams do not satisfy the checklist statements for compactness and column axial stress for the Immediate Occupancy Performance Level, then this item must be marked "Noncompliant".

n_c = 12 columns
n_f = 6 frames
V_j = 168 kips
h = 111 in
M_s = 2.50

Column Shape:
Z_{col} = 13.4 in^3
ΣZ_{col} = 161 in^3
$f_{j\,(col)}^{avg}$ = 46.5 ksi

Beam Shape:
Z_{bm} = 73.4 in^3 (smallest Z for tapered beam)
ΣZ_{bm} = 881 in^3
$f_{j\,(bm)}^{avg}$ = 8.5 ksi

F_y = 44.0 ksi

Acceptance Ratio$_{(col)}$ = 1.06 > 1.0, therefore NC
Acceptance Ratio$_{(bm)}$ = 0.19 < 1.0, therefore C

5.5.2.1.2 Drift Check. An analysis shall be performed in accordance with Section 5.2.4, and the adequacy of the moment–frame and slab–column frame elements, including P-delta effects and their associated connections, shall be evaluated in accordance with Section 5.2.5.

The drift limits presented in ASCE 7-16 are checked against the ASCE 7-16 loads for RC II and RC IV in the following calculations.

CBC 2019, ASCE 7-16 CHAPTER 11, 12, 13 SEISMIC DESIGN CRITERIA

Soil Site Class:	D	Table 20.3-1		
Response Spectral Acc. (0.2 sec) S_s =	1.887g	= 188.7%g		Figure 22-1, 22-3, 22-5, and 22-6
Response Spectral Acc. (1.0 sec) S_1 =	0.721g	= 72.1%g		Figure 22-2, 22-4, 22-5, and 22-6
Site Coefficient F_a =	1.00			Table 11.4-1
Site Coefficient F_v =	1.70			Table 11.4-2
Max Considered Earthquake Acc. S_{MS} =	$F_a.S_s$	= 1.887		(11.4-1)
Max Considered Earthquake Acc. S_{M1} =	$F_v.S_1$	= 1.226		(11.4-2)
S_{DS} =	$2/3(S_{MS})$	= 1.258	(at 5% Damped Design)	(11.4-3)
S_{D1} =	$2/3(S_{M1})$	= 0.817		(11.4-4)
T_S =	0.650 seconds			
Building Risk Category:	II	Standard		Table 1.5-1
Redundancy Factor ρ =	1.0			Section 12.3.4
Design Category Consideration:	Flexible Diaphragm			Section 12.3
Seismic Design Category for 0.1sec	D			Table 11.6-1
Seismic Design Category for 1.0sec	D			Table 11.6-2
S1 < .75g	NA			Section 11.6
Since Ta < .8Ts (see below), SDC =	D	exception of Section 11.6 does not apply		

CBC - Comply with Seismic Design Category D

12.8 Equivalent Lateral Force Procedure

Seismic Force Resisting System:	C. MOMENT-RESISTING FRAME SYSTEMS			T-12.2-1
	4. Steel ordinary moment frames			
C_t =	0.028	x = 0.80		T-12.8-2
Building Height H_n =	14 ft		Limited Building Height (ft) = NPi	
C_u =	1.400	for S_{D1} of 0.817g	Table 12.8-1	
Approx Fundamental Period, T_a =	$C_t(h_n)^x$	= 0.225	12.8-7	T_L = 8 Sec
Calculated T shall not exceed ≤	Cu*Ta	= 0.314		Use T = 0.225 Sec
0.8Ts =	$0.8(S_{D1}/S_{DS})$	= 0.520	exception of Section 11.6 does not apply	
Is structure Regular & ≤ 5 stories?	No			12.8.1.3
S_{DS} to determine C_s & E_v =	1.258g			11.4-3
Response Modification Coef. R =	3.5			Table-12.2-1
Overstrength Factor Ω_o =	2.5			Table-12.2-1 Footnote b
Deflection Amplification Factor C_d =	3			Table-12.2-1
Seismic Importance Factor I_e =	1.00			Table 1.5-1

Seismic Base Shear V = C_sW

$$C_s = \frac{S_{DS}}{R/I_e} = 0.359 \qquad (12.8\text{-}2)$$

or need not to exceed, $$C_s = \frac{S_{D1}}{(R/I_e)T} = 3.118 \quad \text{For } T \le T_L \qquad (12.8\text{-}3)$$

or $$C_s = \frac{S_{D1}T_L}{T^2(R/I_e)} = \text{N/A} \quad \text{For } T > T_L \qquad (12.8\text{-}4)$$

C_s shall not be less than…	0.055	≥ 0.01		(12.8-5)
Min C_s =	$0.5S_1I_e/R$	= 0.309	For S_1 ≥ 0.6g	(12.8-6)
Use C_s =	0.359			

Design Base Shear V (ULT) = 0.359 W
Design Base Shear V (ASD) = 0.252 W

CBC 2019, ASCE 7-16 CHAPTER 11, 12, 13 SEISMIC DESIGN CRITERIA

Soil Site Class:	D	Table 20.3-1		
Response Spectral Acc. (0.2 sec) S_s =	1.887g	= 188.7%g		Figure 22-1, 22-3, 22-5, and 22-6
Response Spectral Acc. (1.0 sec) S_1 =	0.721g	= 72.1%g		Figure 22-2, 22-4, 22-5, and 22-6
Site Coefficient F_a =	1.00			Table 11.4-1
Site Coefficient F_v =	1.70			Table 11.4-2
Max Considered Earthquake Acc. S_{MS} =	$F_a.S_s$	= 1.887		(11.4-1)
Max Considered Earthquake Acc. S_{M1} =	$F_v.S_1$	= 1.226		(11.4-2)
S_{DS} =	$2/3(S_{MS})$	= 1.258	(at 5% Damped Design)	(11.4-3)
S_{D1} =	$2/3(S_{M1})$	= 0.817		(11.4-4)
T_S =	0.650 seconds			
Building Risk Category:	IV	Essential		Table 1.5-1
Redundancy Factor ρ =	1.0			Section 12.3.4
Design Category Consideration:	Flexible Diaphragm			Section 12.3
Seismic Design Category for 0.1sec	D			Table 11.6-1
Seismic Design Category for 1.0sec	D			Table 11.6-2
S1 < .75g	NA			Section 11.6
Since Ta < .8Ts (see below), SDC =	D	exception of Section 11.6 does not apply		
	CBC - Comply with Seismic Design Category D			

12.8 Equivalent Lateral Force Procedure

Seismic Force Resisting System:	C. MOMENT-RESISTING FRAME SYSTEMS				T-12.2-1
	4. Steel ordinary moment frames				
C_t =	0.02		x = 0.80		T-12.8-2
Building Height H_n =	14 ft		Limited Building Height (ft) =	NPi	
C_u =	1.400	for S_{D1} of	0.817g	Table 12.8-1	
Approx Fundamental Period, T_a =	$C_t(h_n)^x$	= 0.160	12.8-7	T_L = 8 Sec	
Calculated T shall not exceed ≤	Cu*Ta	= 0.225		Use T = 0.225 Sec	
0.8Ts =	$0.8(S_{D1}/S_{DS})$	= 0.520	exception of Section 11.6 does not apply		
Is structure Regular & ≤ 5 stories?	No				12.8.1.3
S_{DS} to determine C_s & E_v =	1.258g				11.4-3
Response Modification Coef. R =	3.5				Table-12.2-1
Overstrength Factor Ω_o =	2.5				Table-12.2-1 Footnote b
Deflection Amplification Factor C_d =	3				Table-12.2-1
Seismic Importance Factor I_e =	1.50				Table 1.5-1
Seismic Base Shear V =	C_sW				
C_s =	$\dfrac{S_{DS}}{R/I_e}$	=0.539			(12.8-2)
or need not to exceed, C_s =	$\dfrac{S_{D1}}{(R/I_e)T}$	= 1.559	For T≤ T_L		(12.8-3)
or C_s =	$\dfrac{S_{D1}T_L}{T^2(R/I_e)}$	N/A	For T > T_L		(12.8-4)
C_s shall not be less than…	0.083	≥ 0.01			(12.8-5)
Min C_s =	$0.5S_1I_e/R$	= 0.155	For S_1 ≥ 0.6g		(12.8-6)
Use C_s =	0.539				
Design Base Shear V (ULT) =	**0.539 W**				
Design Base Shear V (ASD) =	**0.377 W**				

Seismic Weight Takeoff

Level	Diaphragm			Exterior Walls					Level Weight (kips)
	Area (ft^2)	DL (psf)	**Weight (kips)**	DL (psf)	Trib Ht (ft)	Length (ft)	Area (ft^2)	**Weight (kips)**	
Roof	7344	12	**88**	10	6.8	320	2160	**22**	**110**
Σ			**88**					**22**	**110**

Seismic Story Force Distribution (ASCE 7-16 Chapter 12)

S_{DS} = 1.258
I_E = 1.00
RC = II

Redundancy (ρ) = 1.0
Base Shear (V) = 0.359W
V = 39.4 kips (ULT)

Period (T_a) = 0.225 (12.8-7)
Periord Factor (k) = 1.0 (12.8.3)

ULT Story Force Vertical Distribution (ASCE 7-16 12.8.3)							
Level	**w_x (kips)**	**h_x (ft)**	**h_x^k**	**$w_x h_x^k$**	**F_x (kips)**	**ρF_x (kips)**	**Cv_x (%)**
Roof	110	13.5	13.5	1481	39	39	100.0
Σ	109.7 kips			1481	39.4 kips	39.4 kips	

Seismic Story Force Distribution (ASCE 7-16 Chapter 12)

S_{DS} = 1.258
I_E = 1.50
RC = IV

Redundancy (ρ) = 1.0
Base Shear (V) = 0.539W
V = 59.2 kips (ULT)

Period (T_a) = 0.225 (12.8-7)
Periord Factor (k) = 1.0 (12.8.3)

ULT Story Force Vertical Distribution (ASCE 7-16 12.8.3)							
Level	**w_x (kips)**	**h_x (ft)**	**h_x^k**	**$w_x h_x^k$**	**F_x (kips)**	**ρF_x (kips)**	**Cv_x (%)**
Roof	110	13.5	13.5	1481	59	59	100.0
Σ	109.7 kips			1481	59.2 kips	59.2 kips	

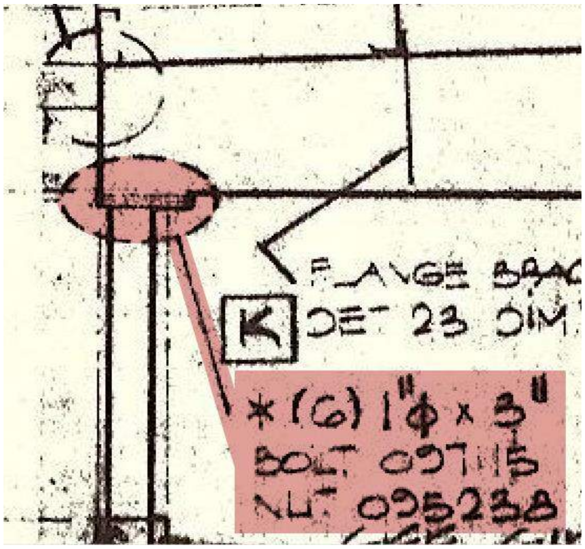
Drift Check

Determine Amplified Drift	*RC II*		*RC IV*	
Seismic Base Shear (V) =	39.4	k (ULT)	59.2	k (ULT)
Roof Area (A) =	7344	ft^2	7344	ft^2
Uniform Seismic Shear (u) =	5.37	psf	8.06	psf
Tributary Width (w) =	20	ft	20	ft
Seismic Load to Moment Frame (v) =	0.107	klf	0.161	klf
Apply 'v' to RISA model to determine frame drift…				
Moment Frame Drift =	1.911	in	2.876	in
Deflection Amplification Factor (C_d) =	3		3	
Seismic Importance factor (I_e) =	1.5		1.5	
Amplified Drift =	**3.82**	**in**	**5.75**	**in**

Check Allowable Drift (ASCE 7-16 Table 12.12-1)				
Structure:	All other structures		All other structures	
Allowable Story Drift (Δ_a) =	0.020	h_{sx}	0.010	h_{sx}
Story Height (h_{sx}) =	10.75	ft	10.75	ft
Δ_a =	**2.58**	**in**	**1.29**	**in**
Acceptance Ratio =	1.48		4.46	

Tier 1

MOMENT-RESISTING CONNECTIONS: All moment connections are able to develop the strength of the adjoining members or panel zones. (Commentary: Sec. A.3.1.3.4. Tier 2: Sec. 5.5.2.2.1)



A.3.1.3.4 Moment-Resisting Connections. All moment connections are able to develop the strength of the adjoining members based on the specified minimum yield stress of steel for moderate seismicity and 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 for high seismicity.

Adjoining Column Strength

Z_{col} =	49.5	in^3	
F_y =	44	ksi	
R_y =	1.5		[per AISC 341-16 Table A3.1]
Expected Strength (E) =	72.6	ksi	[110%R_yF_y]
Member Strength (M_u) =	**3591**	**k-in**	[EZ_{col}]

Adjoining Beam Strength

Z_{bm} =	73.4	in^3	(smallest Z for tapered beam)
F_y =	44	ksi	
R_y =	1.5		[per AISC 341-16 Table A3.1]
Expected Strength (E) =	72.6	ksi	[110%R_yF_y]
Member Strength (M_u) =	**5331**	**k-in**	[EZ_{bm}]

AISC Steel Design Guide Series 16

Flush and Extended Multiple-Row Moment End-Plate Connections

Flow Chart from Design Guide…

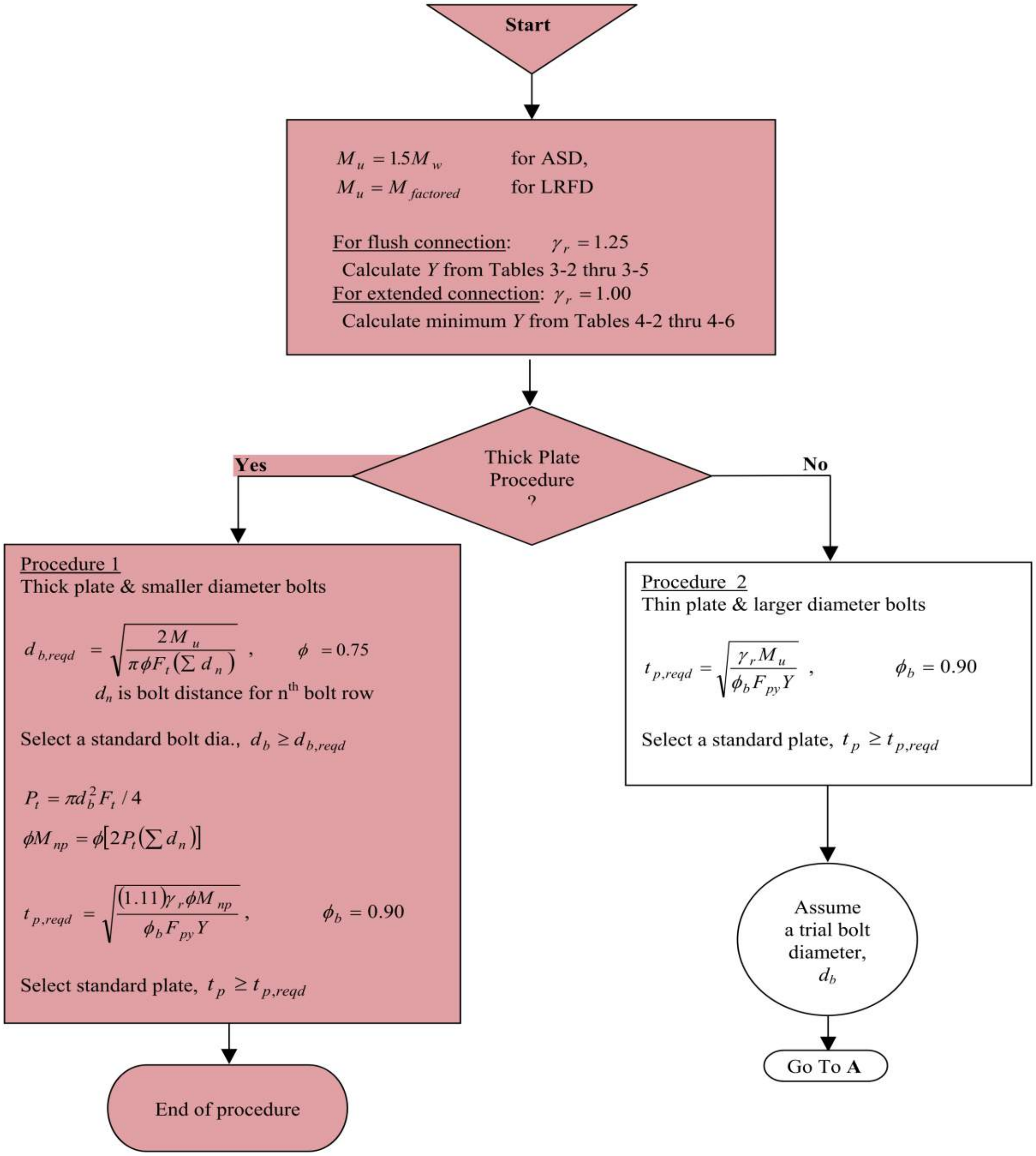
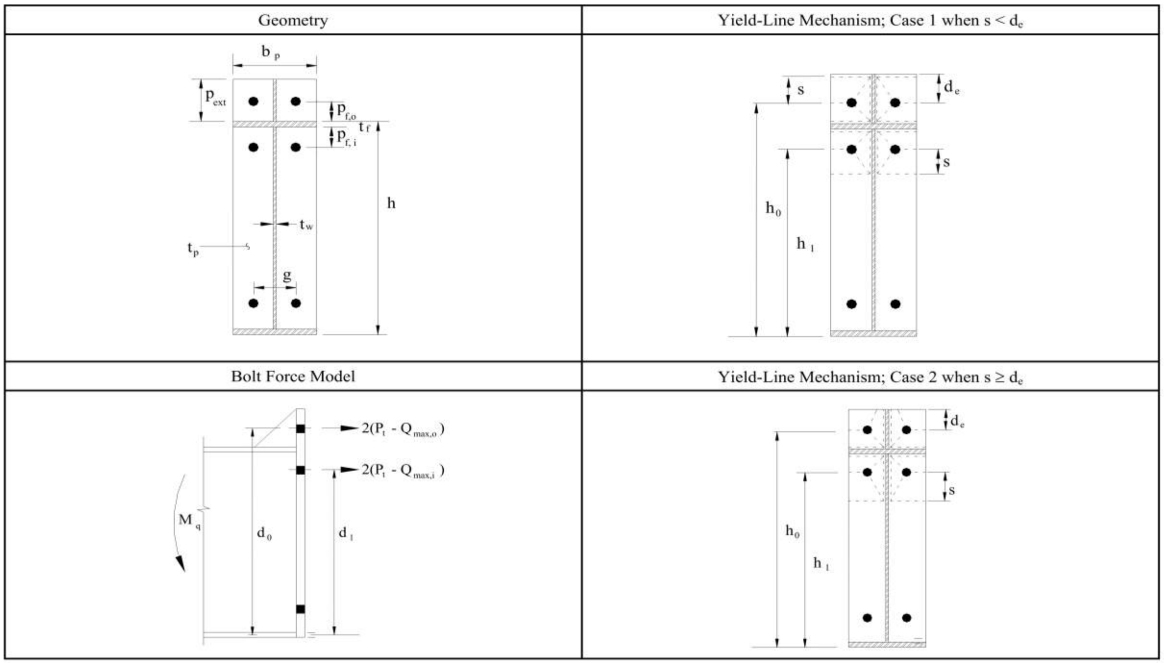


Figure 2-4 Flow-Chart: Bolted end-plate connection design.

Connection Geometry

Table 4-3 Summary of Four-Bolt Extended Stiffened Moment End-Plate Analysis



b_p = 6.5 in　　g = 3.5 in　　s = 2.38 in　　$s = \frac{1}{2}\sqrt{b_p g}$

h_0 = 11.25 in　　$p_{f,i}$ = 2 in　　d_e = 2 in

h_1 = 6.75 in　　$p_{f,o}$ = 2 in　　$d_e < s$　　therefore, Case 1

Connection Capacity for Bolt

Bolt Diameter (d_b) =	1.00	in	[per detail & verified in field]
Reduction Factor (Φ) =	1		[ASCE 41]
Bolt Type =	A325		[verified in field]
Bolt Tensile Strength (F_{tLB}) =	120	ksi	
Distance to First Bolt (d_1) =	6.75	in	[verified in field]
Distance to Second Bolt (d_2) =	11.25	in	[verified in field]
Σd_n =	18.00	in	
Moment Capacity (ΦM_n) =	**3393**	**k-in**	

Connection Capacity for Plate

Plate Thickness (t_p) =	0.875	in	[verified in field]
Plate Yeild Strength (F_{yLB}) =	44	ksi	

$$Y = \frac{b_p}{2}\left[h_1\left(\frac{1}{p_{f,i}} + \frac{1}{s}\right) + h_0\left(\frac{1}{s} + \frac{1}{p_{f,o}}\right)\right] + \frac{2}{g}\left[h_1\left(p_{f,i} + s\right) + h_0\left(s + p_{f,o}\right)\right]$$

Yeild-Line Mechanism Parameter (Y) =	98.9	in	
Reduction Factor (Φ) =	1		[ASCE 41]
Moment Capacity (ΦM_n) =	**3331**	**k-in**	

Compare Strength of Adjoining Members to Moment Connection Capacity

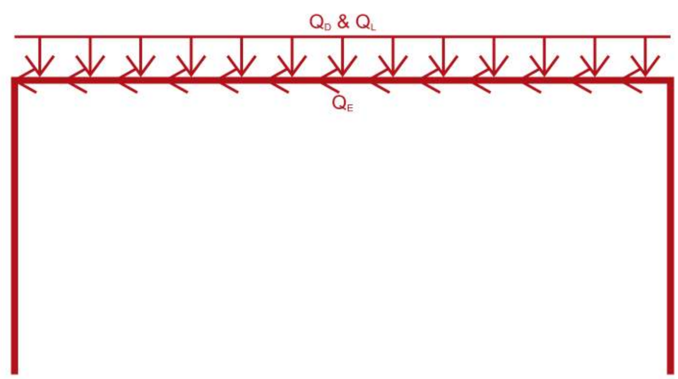
$\Phi M_{n(bolt)}$ = 3393 k-in
$\Phi M_{n(plate)}$ = 3331 k-in

$\Phi M_{n(conn)}$ = 3331 k-in

$M_{u(col)}$ = 3591 k-in **DCR = 1.08 > 1.0 therefore, NC**
$M_{u(bm)}$ = 5331 k-in **DCR = 1.60 > 1.0 therefore, NC**

Tier 2

5.5.2.2.1 Moment-Resisting Connections. The demands on the noncompliant connections shall be computed in accordance with Section 5.2.4, and the connections shall be evaluated in accordance with Section 5.2.5.



Moment Frame RISA Model Loading

Frame Width (w) = 60 ft
Tributary Width (TW) = 20 ft
Frame Height (H) = 13 ft

Vertical Gravity Loads

Roof Dead Load (DL) = 12 psf
Roof Live Load (RLL) = 20 psf
RLL Reduction (R_1) = 0.60
Q_D = DL(TW) = 240 plf
Q_L = RLL(R_1)(TW) = 240 plf

Horizontal Lateral Loads

Base Shear (V) = 1.76 W
Roof Weight (w_R) = 12 psf
Roof Weight (W_R) = 14.4 k
Wall Weight (w_W) = 10 psf
Wall Weight (W_W) = 1.3 k
Seismic Weight (W) = 262 plf
Q_E = VW = 460 plf

Connection Type

Connection: Bolted End Plate
Type: PR (per Table 9-5)
M_{CE} = 3331 k-in (per above)
= 278 k-ft
K_θ = 55518 k-ft/rad

See RISA output for analysis in accordance with §5.2.4

Reactions at Moment Connections

Q_D = 27.1 k-ft

Q_L = 27.1 k-ft

Q_{E1} = 138.1 k-ft

Q_{E2} = 207.2 k-ft

Q_{UD1} = 192.3 k-ft = **2308 k-in**

Q_{UD2} = 261.4 k-ft = **3137 k-in**

Q_{CE} = 3331 k-in

IO m-Factor (m_{IO}) = 1.25 Conservative from Table 9-6 for PR Bolted Flange Plate

LS m-Factor (m_{LS}) = 1.5 Conservative from Table 9-6 for PR Bolted Flange Plate

Knowledge Factor (k) = 0.75

$m_{IO}kQ_{CE}$ = 3123 k

$m_{LS}kQ_{CE}$ = 3747 k

Acceptance Ratio = $Q_{UD1}/m_{IO}kQ_{CE}$ = 0.74 < 1.0, therefore OK

Acceptance Ratio = $Q_{UD2}/m_{LS}kQ_{CE}$ = 0.84 < 1.0, therefore OK

Tier 1

COMPACT MEMBERS: All frame elements meet compact section requirements in accordance with AISC 360, Table B4.1. (Commentary: Sec. A.3.1.3.8. Tier 2: Sec. 5.5.2.2.5)

General Design Criteira

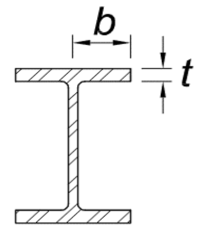
Modulus of Elasticity (E) = 29000 ksi

Yeild Strength (F_y) = 44 ksi

Case = 11 (per Table B4.1)

Compact if $b/t < 0.38\sqrt{(E/F_y)}$

$<$ 9.76



Column:

b = 3 in

t = 0.25 in

b/t = 12.0 > 9.76 therefore NC

Small Tapered Beam:

b = 3.25 in

t = 0.38 in

b/t = 8.7 < 9.76 therefore C

Large Tapered Beam:

b = 4.56 in

t = 0.38 in

b/t = 12.2 > 9.76 therefore NC

A.3.1.3.8 Compact Members. All frame elements meet section requirements in accordance with AISC 341, Table D1.1, for "moderately ductile" members for Collapse Prevention and for "highly ductile" members for Immediate Occupancy, except for building type S-3, where frame elements meet compact section requirements in accordance with AISC 360, Table B4.1.

Noncompact frame elements may experience premature local buckling before development of their full moment capacities. Members that do not meet these criteria may experience premature local buckling before development of their full moment capacities. This problem can lead to poor inelastic behavior and ductility.

The adequacy of the frame elements can be demonstrated by a Tier 2 evaluation using reduced *m*-factors in consideration of reduced capacities for noncompact sections.

Noncompact members can be eliminated by adding appropriate steel plates. Stiffening elements (e.g., braced frames, shear walls, or additional moment frames) can be added throughout the building to reduce the expected frame demands.

Tier 2

5.5.2.2.5 Compact Members. An analysis shall be performed in accordance with Section 5.2.4, and the adequacy of all noncompliant beams and columns that are part of a moment frame shall be evaluated in accordance with Section 5.2.5.

<u>Determine m-Factor per Table 9-6</u>

<u>Column:</u>

Flange Width (b_f) = 6 in

Flange Thickness (t_f) = 0.25 in

Web Height (h) = 7.5 in

Web Thickness (t_w) = 0.125 in

Modulus of Elasticity (E) = 29000 ksi

Lower Bound Strength (F_{yLB}) = 44 ksi

Expected Strength Factor = 1.1 ASCE 41-17 Table 9-3

Expected Strength (F_{ye}) = 48.4 ksi

Assume m-factor = 1.25 (conservative for both IO & LS)

$b_f/(2t_f)$ = 12.0

h/t_w = 60.0 < 96.5 therefore, compact

Large Tapered Beam:

Flange Width (b_f) =	9.125	in	$b_f/(2t_f)$ =	12.2
Flange Thickness (t_f) =	0.375	in		
Web Height (h) =	29.25	in	h/t_w =	117.0
Web Thickness (t_w) =	0.25	in		
Modulus of Elasticity (E) =	29000	ksi		
Lower Bound Strength (F_{yLB}) =	44	ksi		
Expected Strength Factor =	1.1		ASCE 41-17 Table 9-3	
Expected Strength (F_{ye}) =	48.4	ksi		

Slenderness to determine m-factor… m-factor

$0.30\sqrt{(E/F_{ye})}$ =	7.3	<	12.2	1.25
$0.38\sqrt{(E/F_{ye})}$ =	9.3	<	12.2	
$2.45\sqrt{(E/F_{ye})}$ =	60.0	<	117.0	1.25
$3.75\sqrt{(E/F_{ye})}$ =	92.0	<	117.0	

m-factor = 1.25

Column Compression Capacity per AISC 360-16 Chapter E

Deformation Controlled Φ =	1.00		ASCE 41-17 Chapter 9
K =	0.7		
L =	13	ft	
r =	1.648	in	(strong axis)
KL/r =	66.3		

Flexural Buckling (§ E3)

Modulus of Elasticity (E) =	29000	ksi	
Expected Strength (F_{ye}) =	48.4	ksi	
$4.71\sqrt{E/F_{ye}}$ =	115.3	>	66.3 , therefore use (E3-2)
Elastic Buckling Stress (F_e) =	65.2	ksi	(E3-4)
Critical Stress (F_{cr}) =	35.5	ksi	(E3-2)
Column Area (A_g) =	3.94	in	
Compressive Strength (ΦP_n) =	139.7	k	

Torsional Buckling (§ E4)

Modulus of Elasticity (E) =	29000	ksi
Expected Strength (F_{ye}) =	48.4	ksi

Use E4 (b) (i)…

C_w =	135.2	in
G =	11500	ksi
J =	0.066	in^4
I_x =	49.5	in^4
I_y =	9.0	in^4

K_z = 1.0

Elastic Buckling Stress (F_e) = 40.2 ksi (E4-4)

Critical Stress (F_{cr}) = 29.2 ksi (E3-2)

Column Area (A_g) = 3.94 in

Compressive Strength (ΦP_n) = 115.1 k

Expected Axial Compressive Strength of Column (Q_{CE}) = 115.1 k

m-Factor = 1.25

Knowledge Factor (k) = 0.75

mkQ_{CE} = 107.9 k

See RISA output for analysis in accordance with §5.2.4

Axial Force in Column

Q_D = 7.11 k

Q_L = 7.11 k

Q_{E1} = 4.8 k

Q_{E2} = 7.2 k

Q_{UD1} = 19.0 k

Q_{UD2} = 21.4 k

Acceptance Ratio = $Q_{UD1}/(mkQ_{CE})$ = 0.18 < 1.0, therefore OK

Acceptance Ratio = $Q_{UD2}/(mkQ_{CE})$ = 0.20 < 1.0, therefore OK

Column Flexural Capacity per AISC 360-16 Chapter F

Φ = 1.00 ASCE 41-17 Chapter 9

Flanges = Noncompact

Web = Compact

Applicable Section in Chapter F = F3 E = 29000 ksi

Unbraced Length (L_b) = 9.25 ft F_{yLB} = 44 ksi

I_x = 49.5 in^4	S_x = 12.4 in^3	Z_x = 13.4 in^3	r_x = 3.5 in
I_y = 9.0 in^4	S_y = 3.0 in^3	Z_y = 4.5 in^3	r_y = 1.5 in
A = 3.94 in^2	C_w = 135.6 in^6	J = 0.07 in^4	

F3.1 - Lateral-Torsional Buckling — Provisions of Section F2.2 apply.

Limiting Length (L_p) = 68.3 in (F2-5)

L_p = 5.7 ft

$L_b > L_p$

Limiting Length (L_r) = 165.6 in

L_r = 13.8 ft

L_b < L_r

therefore, use (b)...

M_{max} = 103.9 k-ft

M_A = 26.24 k-ft

M_B = 54.74 k-ft

M_C = 77.66 k-ft

C_b = 1.64

Plastic Moment (M_p) = 588.9 k-in (F2-1)

ΦM_n = 588.9 k-in (F2-2)

F3.2 - Compression Flange Local Buckling

Plastic Moment (M_p) = 588.9 k-in (F2-1)

λ = 12.0

λ_{pf} = 9.76

λ_{rf} = 25.7

ΦM_n = 559.5 k-in (F3-1)

Expected Flexural Strength of Column (Q_{CE}) = 560 k-in

m-Factor = 1.25

Knowledge Factor (k) = 0.75

mkQ_{CL} = 524.5 k-in

See RISA output for analysis in accordance with §5.2.4

Bending Moment in Column

Q_D = 162 k-in

Q_L = 162 k-in

Q_{E1} = 1659 k-in

Q_{E2} = 2489 k-in

Q_{UD1} = 1983 k-in

Q_{UD2} = 2812 k-in

Acceptance Ratio = $Q_{UD1}/(mkQ_{CL})$ = 3.78 > 1.0, therefore FAILS

Acceptance Ratio = $Q_{UD2}/(mkQ_{CL})$ = 5.36 > 1.0, therefore FAILS

Design of Member for Combined Forces per AISC 360-16 Chapter H

Applicable Section in Chapter H = H1

BSE-1N

P_r/P_c = 0.18 < 0.2 therefore, H1.1(b) M_r/M_c = 3.78

Combined Acceptance Ratio = 3.87 > 1.0, therefore FAILS

BSE-2N

P_r/P_c = 0.20 < 0.2 therefore, H1.1(b) M_r/M_c = 5.36

Combined Acceptance Ratio = 5.46 > 1.0, therefore FAILS

Large Tapered Beam Compression Capacity per AISC 360-16 Chapter E

Deformation Controlled Φ =	1.00		ASCE 41-17 Chapter 9
K =	0.6		
L =	60	ft	
r =	2.19	in	(strong axis)
KL/r =	197.3		

Flexural Buckling (§ E3)

Modulus of Elasticity (E) =	29000	ksi	
Expected Strength (F_{ye}) =	48.4	ksi	
$4.71\sqrt{E/F_{ye}}$ =	115.3	<	197.3 , therefore use (E3-3)
Elastic Buckling Stress (F_e) =	7.4	ksi	(E3-4)
Critical Stress (F_{cr}) =	6.5	ksi	(E3-3)
Beam Area (A_g) =	12.91	in	
Compressive Strength (ΦP_n) =	83.25	k	

Torsional Buckling (§ E4)

Modulus of Elasticity (E) =	29000	ksi	
Expected Strength (F_{ye}) =	48.4	ksi	
Use E4 (b) (i)…			
C_w =	7204	in	
G =	11500	ksi	
J =	0.445	in^4	
I_x =	1335	in^4	
I_y =	47.5	in^4	
K_z =	1.0		
Elastic Buckling Stress (F_e) =	6.6	ksi	(E4-4)
Critical Stress (F_{cr}) =	2.2	ksi	(E3-2)
Beam Area (A_g) =	12.91	in	
Compressive Strength (ΦP_n) =	28.75	k	

Expected Axial Compressive Strength of Beam (Q_{CE}) = 28.75 k
m-Factor = 1.25
Knowledge Factor (k) = 0.75
mkQ_{CE} = 26.9 k

See RISA output for analysis in accordance with §5.2.4

Axial Force in Beam

Q_D = 2.81 k
Q_L = 2.81 k
Q_{E1} = -14.3 k
Q_{E2} = -21.4 k
Q_{UD1} = 8.7 k
Q_{UD2} = 15.8 k

Acceptance Ratio = $Q_{UD1}/(mkQ_{CE})$ = 0.32 < 1.0, therefore OK
Acceptance Ratio = $Q_{UD2}/(mkQ_{CE})$ = 0.59 < 1.0, therefore OK

Beam Flexural Capacity per AISC 360-16 Chapter F

Φ = 1.00 ASCE 41-17 Chapter 9
Flanges = Noncompact
Web = Compact
Applicable Section in Chapter F = F3 E = 29000 ksi
Unbraced Length (L_b) = 10 ft F_{yLB} = 44 ksi

I_x = 1335 in^4	S_x = 110.9 in^3	Z_x = 121.0 in^3	r_x = 10.2 in
I_y = 47.5 in^4	S_y = 10.4 in^3	Z_y = 16.0 in^3	r_y = 1.9 in
A = 12.91 in^2	C_w = 7204 in^6	J = 0.45 in^4	

F3.1 - Lateral-Torsional Buckling

Provisions of Section F2.2 apply.

Limiting Length (L_p) = 86.7 in (F2-5)
L_p = 7.2 ft
L_b > L_p
Limiting Length (L_r) = 201.9 in
L_r = 16.83 ft
L_b < L_r
therefore, use (b)...
M_{max} = 207.1 k-ft
M_A = 47.01 k-ft
M_B = 176.9 k-ft
M_C = 203.5 k-ft

C_b = 1.31
Plastic Moment (M_p) = 5325 k-in (F2-1)
ΦM_n = 5325 k-in (F2-2)

F3.2 - Compression Flange Local Buckling
Plastic Moment (M_p) = 5325 k-in (F2-1)
λ = 12.2
λ_{pf} = 9.76
λ_{rf} = 25.7
ΦM_n = 5035 k-in (F3-1)

Expected Flexural Strength of Beam (Q_{CE}) = 5035 k-in
m-Factor = 1.25
Knowledge Factor (k) = 0.75
mkQ_{CL} = 4721 k-in

See RISA output for analysis in accordance with §5.2.4

Bending Moment in Beam

Q_D = 76.0 k-in
Q_L = 76.0 k-in
Q_{E1} = 55.1 k-in
Q_{E2} = 82.7 k-in
Q_{UD1} = 207.1 k-in
Q_{UD2} = 234.7 k-in

Acceptance Ratio = $Q_{UD1}/(mkQ_{CL})$ = 0.04 < 1.0, therefore OK
Acceptance Ratio = $Q_{UD2}/(mkQ_{CL})$ = 0.05 < 1.0, therefore OK

Design of Member for Combined Forces per AISC 360-16 Chapter H
Applicable Section in Chapter H = H1

BSE-1N
P_r/P_c = 0.32 > 0.2 therefore, H1.1(a) M_r/M_c = 0.04
Combined Acceptance Ratio = 0.36 < 1.0 OK

BSE-2N
P_r/P_c = 0.59 > 0.2 therefore, H1.1(a) M_r/M_c = 0.05
Combined Acceptance Ratio = 0.63 < 1.0 OK

Tier 1

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)

The capacity of the braced diaphragm is controlled by the capacity of the brace rods in tension, by observation. The ability of the forces to transfer to the frames and braced diaphragm is controlled by the capacity of the Z-purlin connection to the frame beams, by observation. The lowest value of these two capacities is the capacity of the diaphragm.

Braced Diaphragm

AISC 360-16 §D.2 TENSILE STRENGTH

Φ =	1.00		ASCE 41-17 Chapter 9
Lower Bound Strength (F_{yLB}) =	36	ksi	
Ultimate Strength (F_u) =	58	ksi	
Brace Diameter (D_{br}) =	0.63	in	
Gross Area (A_g) =	0.31	in^2	
Net Area (A_n) =	0.31	in^2	
Shear Lag Factor (U) =	1.0		
Effective Net Area (A_e) =	0.31	in^2	
Tensile Yielding in the Gross Section			
Design Tensile Strength (ΦP_{ny}) =	11.0	k	AISC 360-16 (D2-1)
Tensile Rupture in the Net Section			
Design Tensile Strength (ΦP_{nr}) =	17.8	k	AISC 360-16 (D2-2)
ΦP_n =	**11.0**	**k**	

Diaphragm Strength from Directional Component of Brace

Number of Bays (N) =	2	bays	
Number of Braces (n) =	2	braces at connection per bay	
Directional Component of Capacity (c) =	0.707		
Strength of Braces (B_S) =	**31.2**	**k**	**[ΦP_{nr}(N)(n)(c)]**

Z-Purlin Diaphragm

AISC 360-16 §J3.6 SHEAR STRENGTH OF BOLTS

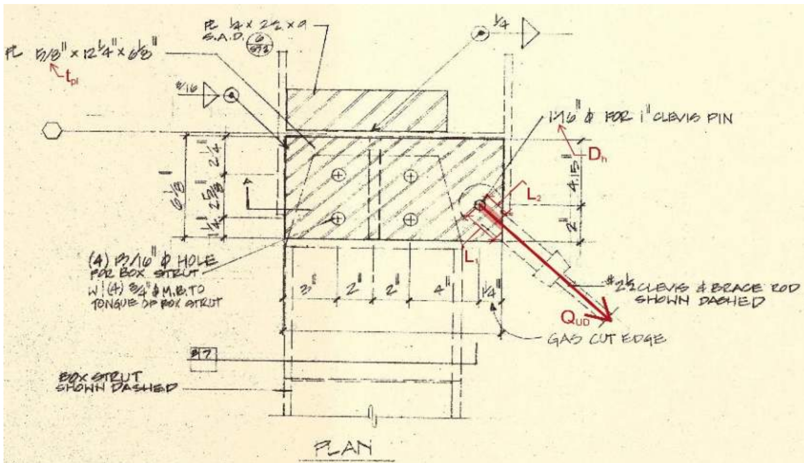
Φ =	1.00		ASCE 41-17 Chapter 9
Bolt Diameter (d_b) =	0.50	in	
Bolt Area (A_b) =	0.196	in^2	
Nominal Shear Strength (F_{nv}) =	27	ksi	
Shear Strength of Bolt (ΦR_n) =	5.3	k/bolt	

Diaphragm Strength from Z-Purlins Connected to Frames

Number of Z-Purlins per frame (n) =	12 purlins	
Number of Bolts per Z-Purlin (N) =	2 bolts/purlin	
Strength of Bolts (B_S) =	**127 k**	**[ΦP_{nr}(N)(n)(c)]**

Connections *Force Controlled per ASCE 41-17 §9.5.2.4.1*

According to §9.5.2.3.2 determine strength per AISC 360 & the *Steel Construction Manual* …



AISC 360-16 §J4.3 - STRENGTH OF ELEMENTS IN SHEAR

Φ =	1.00	ASCE 41-17 §9.5.2.3.2
Lower Bound Strength (F_{yLB}) =	36 ksi	
Ultimate Strength (F_u) =	58 ksi	
Plate Thickness (t_{pl}) =	0.625 in	
Hole Diameter (D_h) =	1.06 in	
Side Length (L_1) =	1.50 in	
Side Length (L_2) =	2.00 in	
Gross Area (A_{gv}) =	2.19 in^2	$= t_{pl}(L_1+L_2)$
Net Area (A_{nv}) =	1.86 in^2	$= t_{pl}(L_1+L_2-2(D_h/4))$

Shear Yeilding of the Element

$\Phi R_n = \Phi 0.6 F_{yLB} A_{gv}$ =	47.3 k	AISC 360-16 (J4-3)

Shear Rupture of the Element

$\Phi R_n = \Phi 0.6 F_u A_{nv}$ =	64.6 k	AISC 360-16 (J4-4)

Lower-Bound Strength of Plate (Q_{CL}) =	**47.3 k**
Knowledge Factor (k) =	0.75
kQ_{CL} =	**35.4 k**

AISC Steel Construction Manual - Table 15-4 (Clevises)

Size =	#2.5		
Available Strength =	18.8	k	(LRFD)
Φ_{LRFD} =	0.5		
Available Strength =	37.6	k	(ULT)
Φ =	1.00		ASCE 41-17 §9.5.2.3.2
Lower Bound Strength (R_{LB}) =	37.6	k	

Lower-Bound Strength of Clevis (Q_{CL}) =	**37.6**	**k**
Knowledge Factor (k) =	0.75	
kQ_{CL} =	**28.2**	**k**

Strength of Diaphragm < Strength of Connection , therefore C

*connection is able to develop strength of diapahragm

Tier 1

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 following: 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)

Tensile Capacity of Column

Cross Section Area (A_s) = 3.94 in^2
Yeild Strength (F_y) = 44 ksi
Tensile Strength = 173.3 k

Uplift Capacity of Foundation

Weight of Concrete (γ) = 150 pcf
Type of Foundation: Square Pad Footing (F1)
Plan Dimensions (s) = 3.5 ft
Depth (d) = 18 in
W1 = 2756 lb
Type of Foundation: Grade Beam (GB1)
Width (b) = 1.17 ft
Depth (d) = 18 in
Tributary Length (L) = 3.5 ft each side
W2 = 1838 lb
Type of Foundation: Tie Beam (TB1)
Width (b) = 1.33 ft
Depth (d) = 8 in
Tributary Length (L) = 2.5 ft
W3 = 333 lb
Type of Foundation: Interior Slab-on-Grade
Depth (d) = 4 in
Tributary Area (A) = 34.2 ft^2
W4 = 1712 lb
Weight of Foundation (U_F) = 6.64 k

Tensile Capacity of Column (T_c) = 173.3 k
Tensile Capacity of Splice (T_s) = N/A (no column splices)
Uplift Capacity of Foundation (U_F) = 6.6 k

Q_{UF} = 6.64 k

Anchorage Capacity

Concrete Breakout in Tension (ACI 318-14 §17.4.2)

f'_c =	3000 psi	$\psi_{ec,N}$ =	1.0	*(Eq. 17.4.2.4)*
h_{ef} =	5.0 in	$\psi_{ed,N}$ =	1.0	*(Eq. 17.4.2.5a)*
1.5 h_{ef} =	7.5 in	$\psi_{c,N}$ =	1.0	*(17.4.2.6)*
s =	4 in	$\psi_{cp,N}$ =	1.0	*(17.4.2.7)*
h_a =	18.0 in	$A_{Nco} = 9(h_{ef})^2$ =	225 in^2	*(Eq. 17.4.2.1c)*
λ_a =	1.0	A_{Nc}=	225 in^2	
k_c =	24	Anchor diameter, d_a =	1.00 in	

$N_b = k_c \lambda_a \sqrt{f'_c}\, h_{ef}^{1.5}$ = 14.7 k *(Eq. 17.4.2.2a)*

$N_b = 16 \lambda_a \sqrt{f'_c}\, h_{ef}^{(5/3)}$ = 12.8 k *(Eq. 17.4.2.2b)*

$N_{cbg} = (A_{Nc}/A_{Nco})(\psi_{ec,N})(\psi_{ed,N})(\psi_{c,N})(\psi_{h,N})N_b$ = 14.7 k *(Eq. 17.4.2.1b)*

Φ = 1.00

ΦN_{cb} = 14.7 k

$0.75\Phi N_{cb}$ = 11.0 k (seismic)

Acceptance Ratio = 0.60 < 1.0 OK

Concrete Pullout in Tension (ACI 318-14 §17.4.3)

f'_c =	3000 psi	Anchor diameter, d_a =	1.00 in	
$\psi_{c,P}$ =	1.0	A_{brg} =	1.16 in^2	
Anchors =	4	$N_p = 8A_{brg}f'_c$ =	27.8 k	*(Eq. 17.4.3.4)*

$N_{pn} = (\psi_{c,P})N_p$ = 27.8 k *(Eq. 17.4.3.1)*

Φ = 1.0

ΦN_{pn} = 27.8 k/anchor

$0.75\Phi N_{pn}$ = 20.9 k/anchor (seismic)

$\Sigma\Phi N_{pn}$ = 111 k

$0.75\Sigma\Phi N_{pn}$ = 83.5 k (seismic)

Acceptance Ratio = 0.08 < 1.0 OK

Max Acceptance Ratio = **0.60 < 1.0 therefore, C**