STRUCTURAL ANALYSIS OF THE HIGH BAY STEEL REACTION FRAMES

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SENIOR PROJECT REPORT

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1.0 Introduction

1.1 Purpose

The purpose of this report is to calculate and document the limit states and overall capacity of the 1970's vintage steel reaction frame in High Bay Laboratory at California Polytechnic State University, San Luis Obispo's College of Architecture and Environmental Design. A reaction frame is used for large scale structural component testing and requires high strength and stiffness, when compared to the structural test specimens, in order to obtain accurate results. A reaction frame with high strength and stiffness will allow for specimen testing to failure and prevent yielding and excess deflection in the reaction frames. Since there are no remaining plans of the reaction frame, member cross sections and connections were identified based on visual inspections. RISA 3D, a structural analysis software tool, and hand calculations were used to confirm the demand on members of the frame with an actuator applying 23.6 kip lateral force cyclically at the top of the frame. The selected demand is based on existing double-acting actuator with a compression capacity of 110 kips, and 23.6 kip tension capacity.

1.2 Scope of Report

This report includes an investigation of the existing reaction frames, strong floor, and respective connection's capacity for quasi-static cyclical testing. There are many uncertainties in this report, such as material properties, which were determined with knowledge of typical construction practices circa 1970 or by assuming code minimum values. For code references used in this report, reference Section 3.3. The design of specimens for future tests are limited to the strength of the existing system, highlighting the value of this report to researches in Cal Poly's Architectural Engineering department.

1.3 Report Overview

The final deliverable for this project is a set of calculations that will serve as an archive to be used in future experimental projects conducted in the High Bay laboratory. The report opens with the verification of existing conditions, followed by estimating the capacity of the existing steel reaction frames, strong floor, and their respective connections using an ultimate strength limit state approach. It concludes with a summary of the governing component of the reaction frame system as well as suggestions for upgrading the system and actuator in the future.

1.4 Future Work

The original intent of this overall project was to design, test, and repair concrete wall specimens. It was necessary to ensure the reaction frame, by applying loads to the test specimens, does not yield prior to the wall specimen failures. The concrete shear wall specimens were used to determine deflection criteria and how to stiffen the reaction frames.

2.0 Verification of Existing Conditions

2.1 Reaction Frame Setup

The current testing setup was constructed using two adjacent reaction frames (Figure 1), which are set 3-ft apart and bolted into a sleeves embedded in the strong floor (Figure 9). A third reaction frame currently is attached to a large horizontal beam, which provides out-of-plane stability. For simplicity, both the third reaction frame and horizontal beam connecting the three frames will not be included in the analysis. Photographs in Appendix A.3 represent the current as-built condition of the frames. It should be noted that the vertical placement of the horizontal beam will vary based on the desired experimental setup for the structure being tested. Drawings and calculations represent the desired configuration for testing described in Section 1.4.

2.2 Member Sizes

The steel reaction frame and strong floor were constructed during the 1970s. Steel reaction frame members (Figure 1) were measured to the nearest 1/16-in using a measuring tape and were compared to sizes in the 7th edition Steel Construction Manual (AISC 360-73). Steel structural member sections were identified based on web thickness, web depth, flange thickness, and flange width. In cases where geometry was indistinguishable, the member with the smallest capacity was chosen (i.e. W12x40 vs W12x80, W12x40 was selected). Therefore, the analyses in this document may be considered conservative.

It was assumed that the reaction frame was constructed using the following members, as determined with AISC 360-73:

- Horizontal beam between reaction frames: W8x24
- Reaction frame columns: W14x61
- Main diagonal braces in reaction frames: (2)C9x13.4
- Smaller diagonal braces in reaction frames: (2)C4x4.5
- Reaction frame floor beam: W12x36

2.3 Connections

Bolts were measured to the nearest 1/16-in using a measuring tape. 7/8-in diameter bolts are typically used in the frame. 1-1/4-in diameter bolts are used to anchor the steel reaction frame into the strong floor. 1-1/2-in diameter bolts are used for the connection between the actuator and sandwich plate. Due to the lack of existing details, it was assumed that a minimum of 1/4-in fillet welds were used for each welded connection.

2.4 Crane

The existing crane is a Detroit Hoist with a capacity of 3 ton, which is equivalent to 6,000 pounds.

2.5 Actuator

The actuator with the greatest capacity currently available in the High Bay Laboratory is the Enerpac RR5013. It has compression capacity of 110 kips and a tension capacity of 23.6 kips. Two special plates have been fabricated to connect the actuator to the horizontal beam (Figure 7), which are referred to as the “sandwich plates” throughout this report. The sandwich plate which is used in this analysis is option A as noted in Figure 8. Note that sandwich plate - Option B consists of larger plates, more welded connections, and has a larger capacity, which is not analyzed in this report.

2.6 Strong Floor

A cross-section of the existing strong floor, shown in Figure 9, is 4-ft deep and was constructed with steel reinforcement mats of both No. 6 @ 6-in o.c. and No. 6 @ 4-in o.c. at the top and bottom of the floor cross-section, respectively. Figure 9 shows a steel tube is embedded at the surface of the strong floorand allows bolts to be anchored 4-in. No. 11 rebar is attached to the bottom of the sleeve using a full penetration weld and is hooked at the bottom of the strong floor. Figure 10 shows original 1974 hand-drafted plans of the existing strong floor in the High Bay Laboratory, which was acquired from the Cal Poly Facilities archive.

2.7 As-Built Drawings

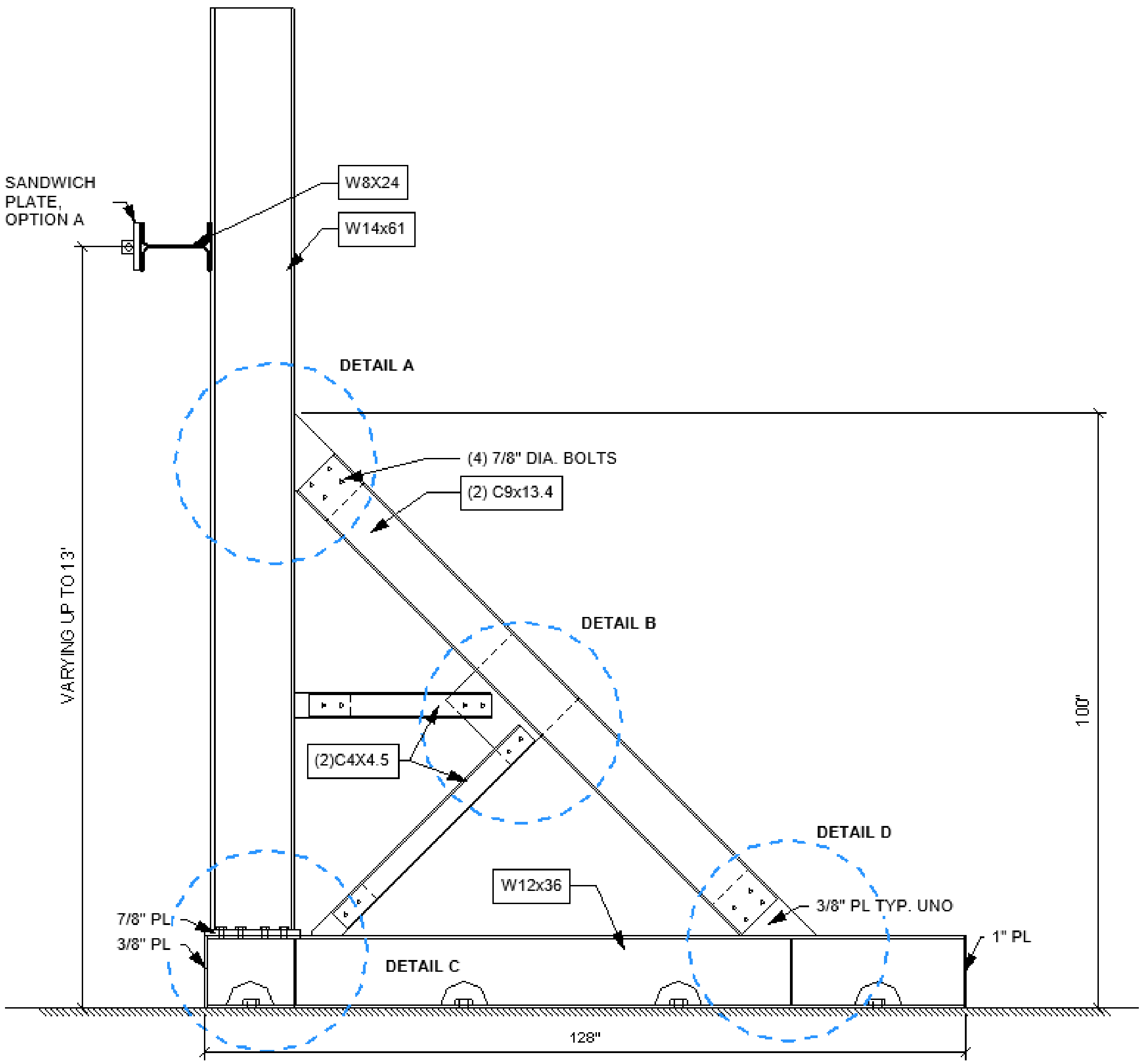


Figure 1 : Side Elevation of Reaction Frame. (For Front Elevation, see Figure 6. Details in Figures 2-5)

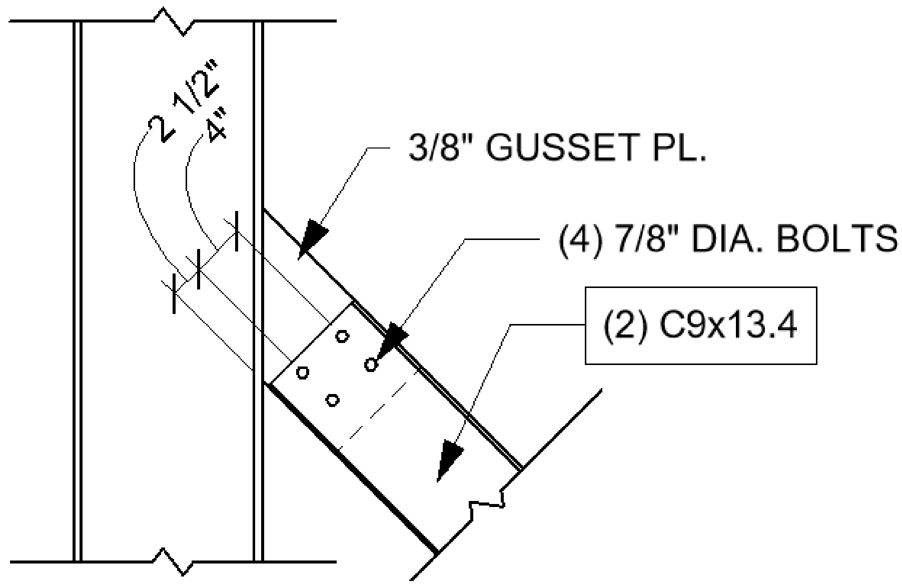


Figure 2 : Detail A - Column to Main Brace Connection

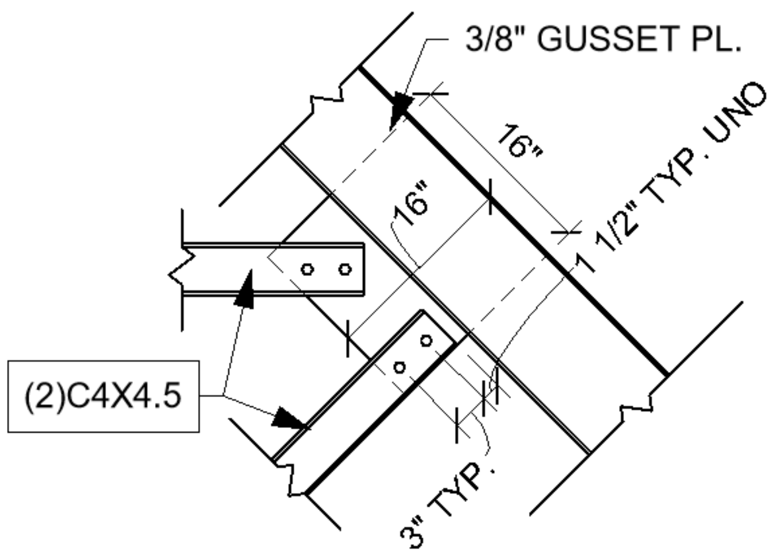


Figure 3 : Detail B – Intermediate Bracing Members to Main Brace Connection

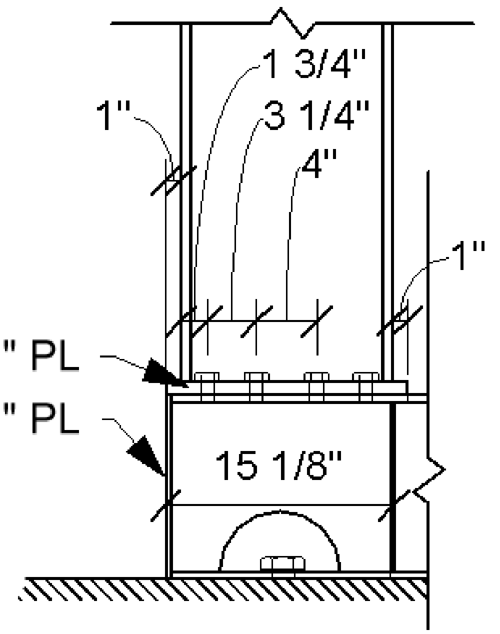


Figure 4 : Detail C - Column to Floor Beam Connection

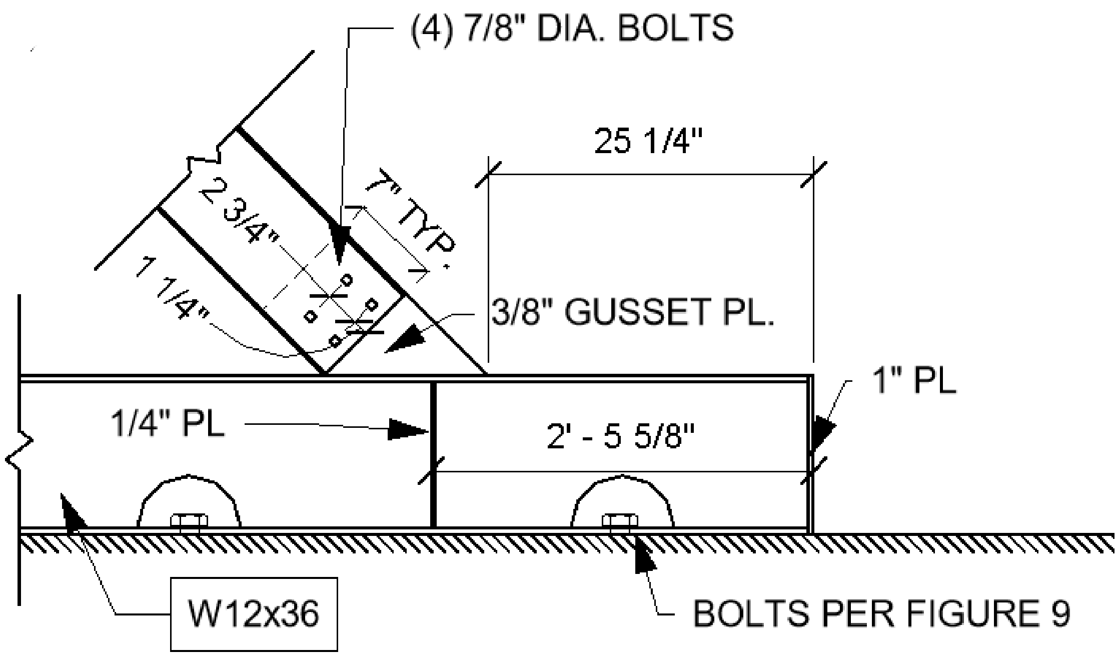


Figure 5 : Detail D - Main Brace to Floor Beam Connection

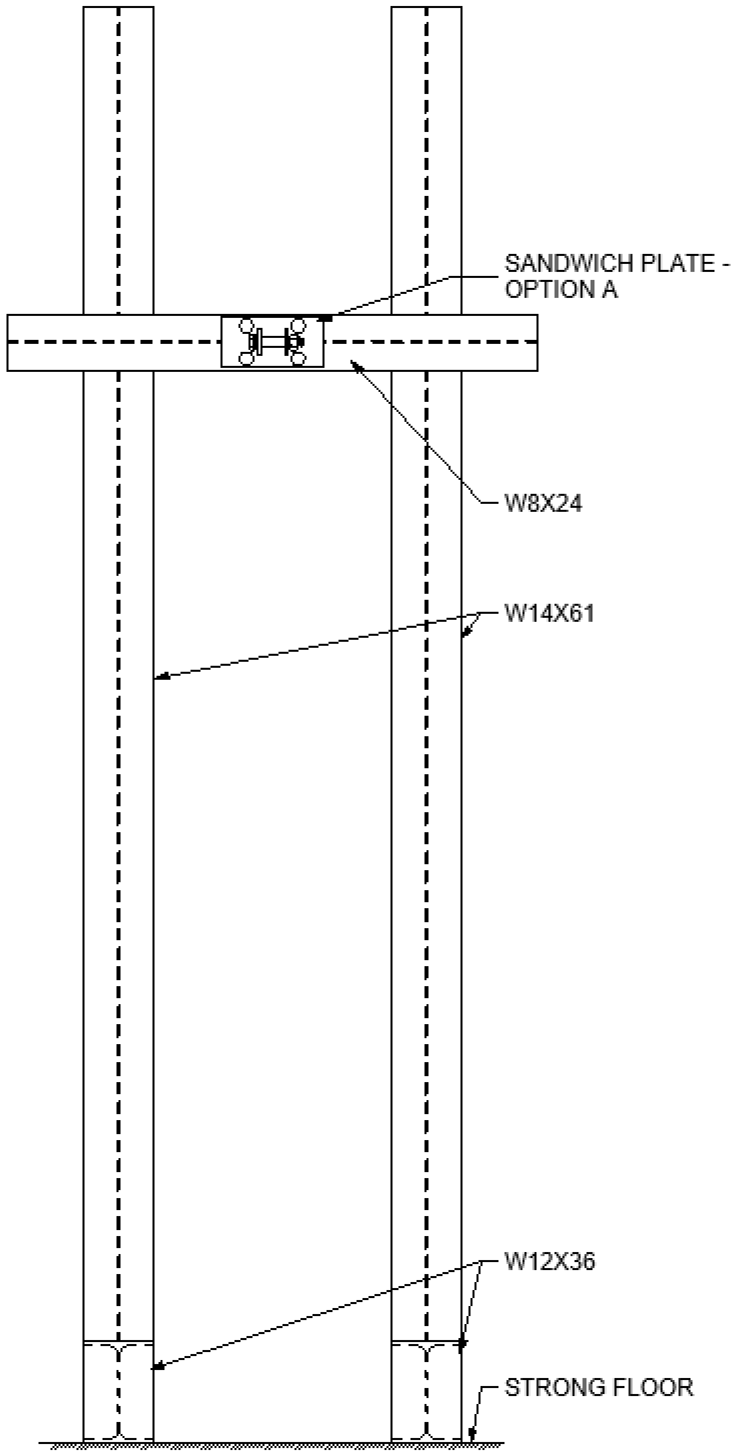


Figure 6 : Front Elevation of Reaction Frame. (See Figure 7 for Horizontal Beam.)

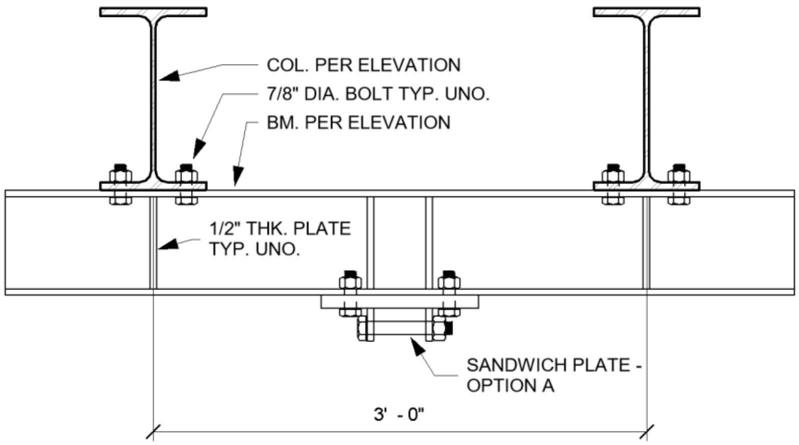
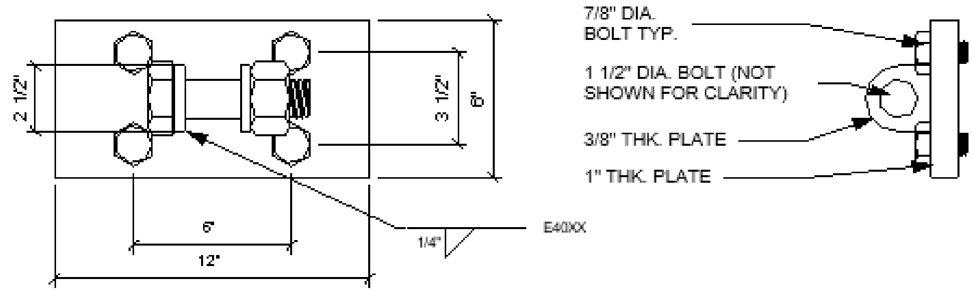
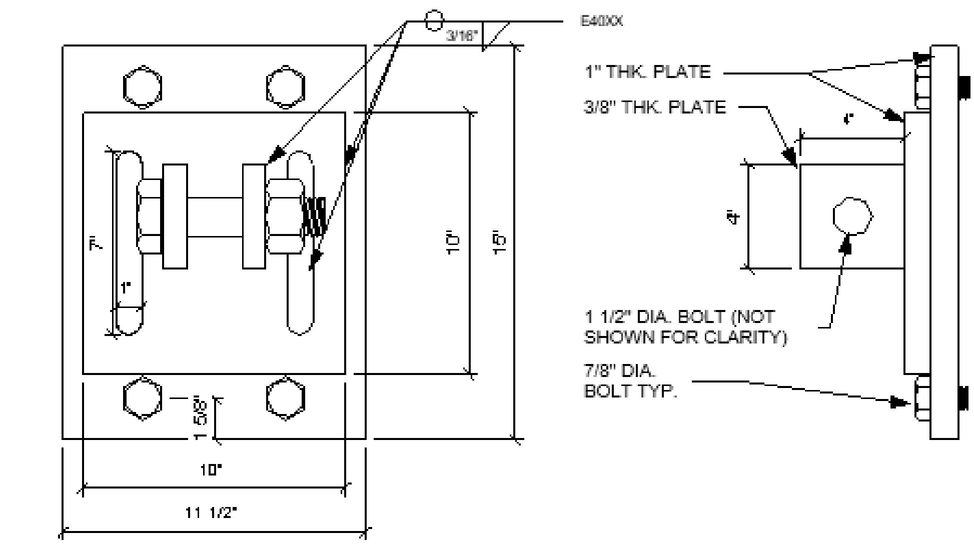


Figure 7 : Plan View of Horizontal Beam



SANDWICH PLATE - OPTION A



SANDWICH PLATE - OPTION B

Figure 8 : Sandwich Plate Options

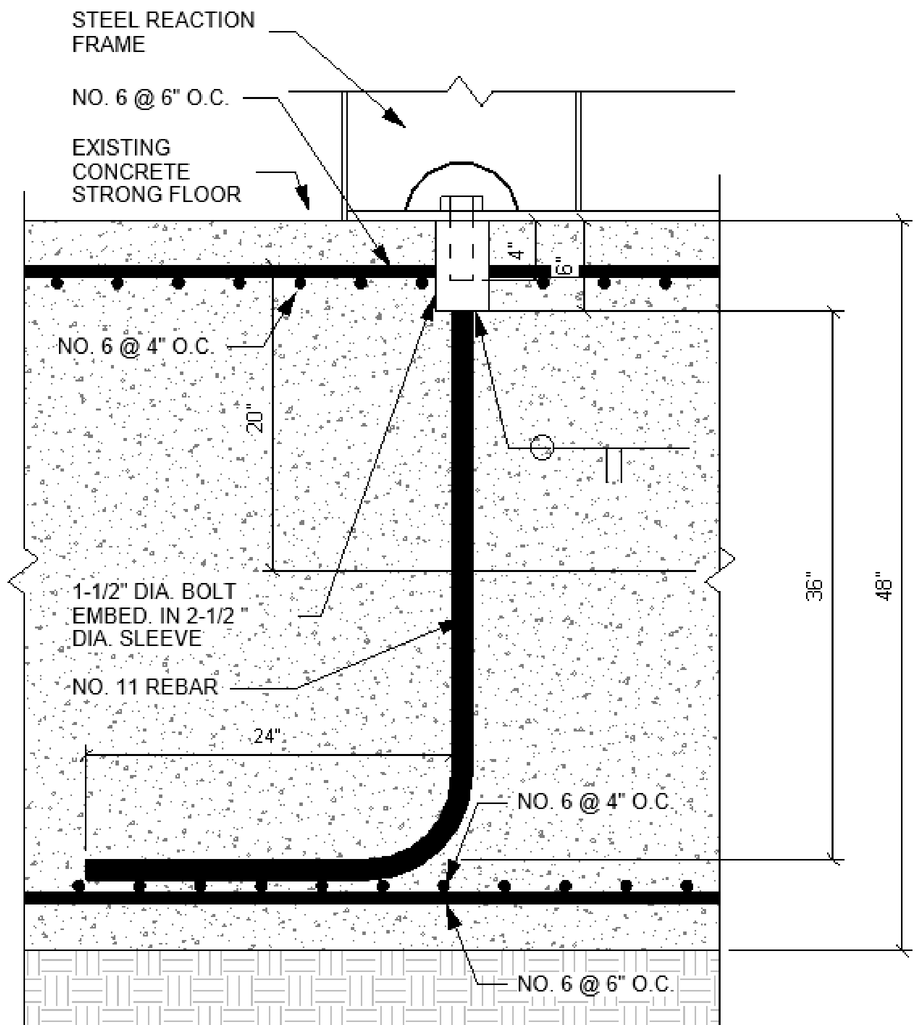


Figure 9 : Detail of Steel Reaction Frame to Strong Floor Connection

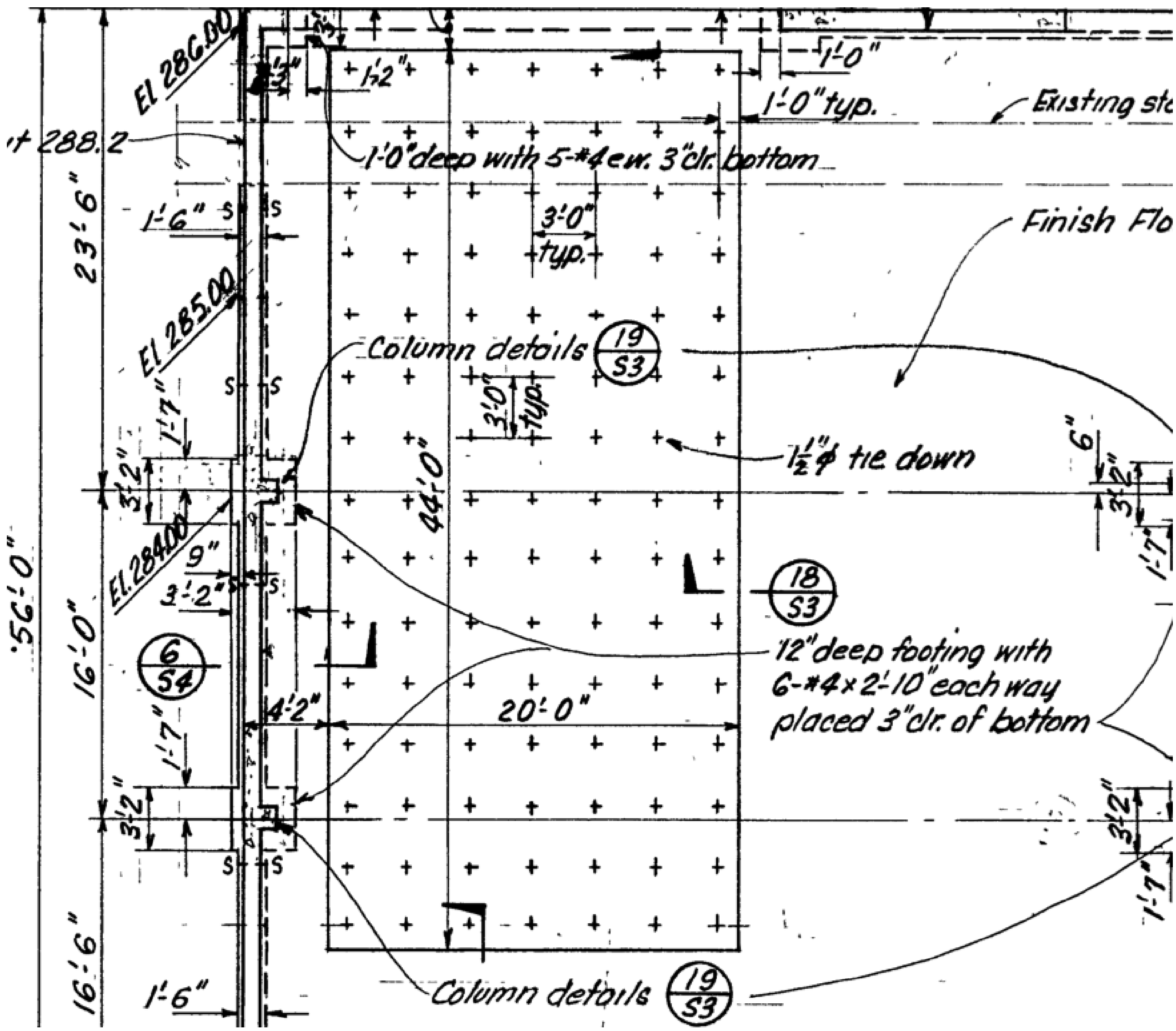


Figure 10 : Strong Floor Plan (J. Shelton, 1974)

3.0 Structural Analysis

3.1 Material Assumptions

Nominal values, based on prescribed material strengths per American Society of Civil Engineers (ASCE) 41-17, American Institute of Steel Construction (AISC) 360-73 or as indicated on existing plans, were used in this evaluation. The AISC 360-73 utilized ASD load combinations, therefore nominal capacities were determined using information from the Uniform Building Code UBC -73. The only values extracted from Chapter 4 in AISC 360-73 were for bolt shear and bolt tension.

3.1.1 Reaction Frame Material Assumptions

- Modern A36 Grade Structural Steel
    - Steel Yield Strength, F_y = 36 ksi
    - Steel Ultimate Strength, F_u = 62 ksi
- E40XX Electrode Weld Material
    - Weld Filler Strength, F_{EXX}= 40 ksi
- A325 Grade Bolts
    - Bolt Tensile Strength, F_{nt} = 66.7 ksi
    - Bolt Shear Strength, F_{nv} = 37.5 ksi

3.1.2 Strong Floor Material Assumptions

- Concrete
    - Concrete Compressive Strength, f_c' = 3000 psi
- Rebar
    - Rebar Yield Strength, F_y = 40 ksi

3.2 Loading Assumptions

- For a description of the load flow, reference Appendix A.2.1.
- Loading from the actuator was applied in the plane of the steel reaction frames and was distributed evenly between the two reaction frames.
- Frame self-weight was neglected in the reaction frame calculations because it is insignificant when compared to axial force subjected to the column from the actuator.
- Frame self-weight was considered in the strong floor anchor bolt calculations because friction due to self-weight and bolt clamping contribute to shear resistance.
- A 23.6 kip actuator force was applied cyclically at a height of 13-ft from the ground.

3.3 Analysis Assumptions

- An ultimate strength limit state approach was used to analyze the frame and all components in the reaction frames.

- A strength reduction factor of $\phi=1.0$ was used to calculate the nominal capacities of the existing members and connections.
- RISA 3D was used to verify hand calculations for axial, shear, and moment demands on the reaction frame. RISA 3D and hand calculations have a difference of less than 5% due to rounding of member lengths in RISA 3D. Thus, hand calculations were used in this analysis for demand and capacity calculations of all members and their respective connections.
- Actual member sizes were input into RISA 3D for deflection check.
- The smaller diagonal members, (2) C4x4.5, were included in the frame to reduce the unbraced length of the (2) C9x13.4 and were anticipated not to transfer loads. Thus, the (2) C4x4.5 were not included in the analysis.

3.4 RISA 3D Analysis

RISA 3D was used to model the demands on the reaction frame system; the model and outputs are summarized in Appendix A.1. Only two reaction frames, as noted in Section 2.1, were used in the model. The model was created using member sizes determined in Section 2.2 and a point load of 23.6 kips was applied to the mid-span of the horizontal beam positioned 13-ft above the ground. Only pinned connections were used in the model. The RISA 3D results were compared to hand calculations, yielding a percent difference of less than five percent. It should be noted that this analysis assumed no fixity in the connections for simplicity in analysis, when in reality there are some fixities.

3.5 Capacity Analysis per Code Provisions

Limit states of the reaction frame were determined using modern or vintage code provisions. As noted in Section 3.3, a strength reduction factor of $\phi=1.0$ is used in this analysis. An ultimate strength limit state was used to determine all capacities in the frames and will be reduced according to researcher defined criteria based on their structural test specimen and experimental objectives of their project.

The steel reaction frame members and connections were analyzed using AISC 360-10. For limit states not mentioned in AISC 360-10, Segui's *Steel Design, 5th Edition* was used to determine capacities (Segui 2013). However, AISC 360-73 and UBC-73 were used to determine anchor bolt capacities: AISC 360-73 provided ASD capacities for bolts and UBC-73 was required to determine load combinations and bolt nominal capacities. American Concrete Institute (ACI) 318-14 was used to determine anchor bolt to concrete connection capacities, including concrete breakout and development length.

Deflection for the frame system was determined based on the cracked section analysis, per ACI 318-14, of the proposed concrete shear wall specimens noted in Section 1.4. The cracking limit state of these walls was considered because the frame must be sufficiently stiff to perform well under the elastic energy that will be released from the frame into the wall due to the wall cracking. An additional analysis accounting for inelastic deflection of the wall is necessary once the final concrete wall specimen design has been completed.

3.6 Standards of Practice

- American Concrete Institute’s *Building Code Requirements for Structural Concrete* (ACI 318-14)
- American Institute of Steel Construction’s *Steel Construction Manual* (ANSI/AISC 360-10, 14th Edition)
- American Institute of Steel Construction’s *Steel Construction Manual* (ANSI/AISC 360-73, 7th Edition)
- American Society of Civil Engineers’ *Seismic Evaluation and Retrofit of Existing Buildings* (ASCE 41-17)
- Universal Building Code (UBC-73)

3.7 Limit States and D/C Ratios for Steel Reaction Frames and Strong Floor

The capacities calculated in this table are in accordance with modern code provisions, as noted in Section 3.6. The demands were determined using hand calculations, which are within 5% of RISA 3D values as mentioned in Section 3.5. Comments on deflection values are discussed in Section 4.6. Some limit states were deemed non-critical, non-probable failure mode and not calculated in this report as indicated by the N/A designation in Table 1.

Table 1 : Summary of Limit States for Reaction Frame

High Bay Steel Reaction Frame and Strong Floor Limit States (for Vu =23.6 kips, h=13-ft)					
Member/ Connection	Limit State	Capacity	Demand	D/C	Code Reference
Horizontal Beam between Reaction Frames (Sec. A.2.2)	Shear (k)	32.4	11.8	0.364	AISC 360-10 Eqn. G2-1
	Flexure (k-ft)	69.3	17.7	0.255	AISC 360-10 Eqn. F2-1
	Deflection (in) **	0.138	0.00959	0.069	ACI 318-14 T.6.6.3.1.1(a)
Sandwich Plate (Option A) between Horizontal Beam and Actuator (Sec. A.2.2)	Weld in Tension (k)	36.6	11.8	0.322	AISC 360-10 Eqn. J2-4
	Bolt Shear (ksi)	54	13.4	0.248	AISC 360-73 Chapter 4
	Bolt Bearing (ksi)	36	20.1	0.558	
	Bolt Tear Out (k)	11.8	11.8	1.000	
	Bolt Bending (ksi)	54	0.04	0.001	
	Plate Yielding (k)	33.8	11.8	0.349	AISC 360-10 Eqn. D2-1
	Plate Rupturing (k)	21.8	11.8	0.541	AISC 360-10 Eqn. D2-2
	Bolt Tension in Sandwich Plate(ksi)	40	9.81	0.245	AISC 360-73 Chapter 4
	Prying Action in Sandwich Plate (ksi)	40	13.1	0.328	
	Plate Bending (ksi)	36	17.7	0.492	
	Plate Shear (ksi)	N/A	N/A	N/A	
Horizontal Beam and Reaction Frame Connection (Sec. A.2.2)	Stiffener Buckling (ksi)	N/A	N/A	N/A	
	Stiffener Yielding (ksi)	36	3.9	0.108	AISC 360-10 Eqn. D2-1
	Bolt Tension in Column to Beam Connection (ksi)	66.7	4.91	0.123	AISC 360-73 Chapter 4
	Prying Action in Column to Beam Connection (ksi)	40	6.45	0.161	
Reaction Frame Column (Sec. A.2.3)	Flexure (k-ft)	273.6	61.4	0.224	AISC 360-10 Eqn. F2-1
	Shear (k)	75.9	11.8	0.155	AISC 360-10 Eqn. G2-1
	Deflection (in) **	0.135	0.308	2.281	ACI 318-14 T.6.6.3.1.1(a)
	Yielding (k)	644	20.8	0.032	AISC 360-10 Eqn. D2-1
	Rupture (k)	960.3	20.8	0.022	AISC 360-10 Eqn. D2-2
	Compression (k)	639	20.8	0.033	AISC 360-10 Eqn. E3-1

Table 2 Cont'd : Summary of Limit States for Reaction Frame

High Bay Steel Reaction Frame and Strong Floor Limit States (for Vu =23.6 kips, h=13-ft)					
Member/ Connection	Limit State	Capacity	Demand	D/C	Code Reference
Reaction Frame Column to Floor Beam (Sec. A.2.3)	Bolt Tension (k)	321	20.8	0.065	AISC 360-73 Chapter 4
	Bolt Shear (k)	180.4	9	0.050	AISC 360-73 Chapter 4
	Weld in Shear (k)	286.5	9	0.031	AISC 360-10 Eqn. J2-3
	Weld in Tension (k)	286.5	20.8	0.073	AISC 360-10 Eqn. J2-4
	Shear of Base Metal (k)	386.78	9	0.023	AISC 360-10 Eqn. J2-5
Main Brace (Sec. A.2.4)	Plate Yielding (k)	121.5	29.4	0.242	AISC 360-10 Eqn. D2-1
	Plate Rupturing (k)	165.5	29.4	0.178	AISC 360-10 Eqn. D2-2
	Block Shear (k)	103.6	29.4	0.284	AISC 360-10 Eqn. J4-5
	Bolt Shear (ksi)	37.5	12.2	0.320	AISC 360-73 Chapter 4
	Bolt Bearing (ksi)	36	10.5	0.292	
	Bolt Tear Out (k)	11	3.68	0.335	
	Member Yielding (k)	283.7	14.7	0.052	AISC 360-10 Eqn. D2-1
	Member Rupturing (k)	38.4	14.7	0.383	AISC 360-10 Eqn. D2-2
	Member Compression (k)	N/A	N/A	N/A	
	Weld in Shear and Tension (k)	137.7	29.4	0.214	AISC 360-10 Eqn. J2-4
	Axial Deflection **	0.138	0.0218	0.158	ACI 318-14 T.6.6.3.1.1(a)
Floor Beam to Strong Floor (Sec. A.2.5)	Anchor Bolt Shear	66.3	2.95	0.044	AISC 360-73 Chapter 4
	Anchor Bolt Yielding	117.8	10.4	0.088	AISC 360-73 Chapter 4
	Clamping Force + Friction	71.75	23.6	0.329	
Strong Floor (Sec. A.2.5)	Break Out	98.6	10.4	0.105	ACI 318-14 25.4.3.1
	Rebar Yielding	62.4	10.4	0.167	
Overall Reaction Frame (Sec. A.2.6) *	Deflection of Wall Specimen (in) **	0.135	0.339	2.511	ACI 318-14 T.6.6.3.1.1(a)
	Deflection at 3.0% Drift (in)	N/A	4.68	N/A	
*** This analysis uses a cracked moment of inertia. Reference Section 3.5 for more detail. ** Only accounts for elastic deflection of the wall specimen. Demand should be modified based on researcher's anticipated drift capacity (including inelastic response) of their test specimen.**					

4.0 Discussion

4.1 Reduction Factors

An ultimate strength limit state approach was used in the analysis of the frame and associated components. This was done to find the nominal capacity of the reaction frame and strong floor. After determining the nominal capacity, a reasonable safety factor should be applied by the researcher to the reaction frame members and respective connections based on the loading they anticipate in their experimental test, to avoid yielding in the reaction frames and safety of researchers in the laboratory.

4.2 Verification of Demands

It should be noted that hand calculations are used in the determination of demands and discussion of results. RISA-3D was utilized to develop a computational model and compared to hand calculations to verify accuracy of member forces and deflection of the frame under a lateral force of 23.6 kips. Forces and deflections obtained from RISA 3D were within 5% of values from hand calculations. RISA 3D calculated deflection of 0.321-in was compared to a deflection 0.308-in using hand calculations at the point of the applied load 13-ft above the ground.

4.3 Critical Limit States

The determined governing limit states for the current setup are deflection in the frame and shear in the horizontal beam. Additionally, the sandwich plate – option A was the most critical element in the system. It is recommended that the sandwich plate – option B is used instead. The capacity of option B was not calculated, but option B has larger plates and bolts thus it can be safely assumed to have a larger capacity.

Beam shear in the horizontal beam will be a concern if a larger capacity actuator is used. The horizontal beam can be easily replaced with a beam of greater shear capacity if a larger capacity actuator is used. Considerations also must be made for deflection in the reaction frame system as this can affect the accuracy of test results; therefore, a frame stiffening plan is described in Section 4.5.

4.4 Impact of Actuator Location on Frame Response

The height of the horizontal beam that connects the two frames and supports the actuator can be adjusted on the columns of the frame. If the actuator is repositioned below the main diagonal braces (refer to Figure 1), there will be increased column base shear demand. This will require certain limit states to be reassessed, including: shear in the column to plate weld and bolt shear between the 7/8" plate and floor beam. At an actuator height below the main diagonal brace, the column moment and deflection demands would decrease. However, if the actuator was moved higher on the frame, the moment and deflection would increase in the column, requiring a stiffer and stronger frame.

4.5 Proposed Upgrades

4.5.1 Brace Upgrade

The main concern revealed from the frame analysis in this report was that the deflection of the reaction frame during cyclic loading from the actuator exceeded the tolerances that would allow researchers to accurately apply load to the top of the proposed concrete wall specimens.

In order to mitigate this issue, a retrofit approach has been developed where another brace is added to the frame system to increase the frame stiffness. This is shown in Figure 11 and described below:

- Gusset plates will be fillet welded onto bolted plates.
- Bolted plates will be attached to the column and the floor beam extension.
- The new bracing member will be bolted into gusset plates. There are several suggested options:
    - Option A: (2) C15x50, where the reaction frame system is ten times (10x) stiffer than proposed concrete wall specimen mentioned in Section 1.4.
    - Option B: HSS 12x8x5/8, where the frame is 5x stiffer than the wall specimen.
    - Option C: HSS 14x0.625, where the frame is 5x stiffer than the wall specimen.
- A floor beam extension will be added to existing floor beam to attach the proposed brace.
- Stiffener plates will be installed at locations where the proposed brace attaches to floor beam and column in order to prevent local web buckling of these members.
- Note: The proposed braces do not require additional intermediate bracing as buckling should not be a concern. Axial force applied to the braces will be small compared to the axial capacity.

The stiffening options consist of channels or rectangular/circular HSS tubes. The brace constructed from two channel sections is an attractive option as it does not require additional cuts to attach the gusset plates and will provide twice the stiffness, yet it may buckle in the weak axis. The HSS tubes are less likely to buckle in the weak axis, but require a cut slot to fit the gusset plates.

4.5.2 Bolt Upgrade

Another suggestion to improve the performance of the frame for future uses is upgrading the bolts. The bolts were analyzed using material properties in accordance with AISC 360-73. The bolt shear and tension values prescribed in ASIC 360-73 are significantly lower than contemporary values as found in AISC 360-10, but were adjusted using load combinations found in UBC-73. These values are similar to contemporary values, but may still be upgraded for increased capacity.

However, yielding in the bolts first may be desirable, due to the fact that they’re inexpensive and easy to replace. This may prevent yielding in other connections and members.

4.5.3 Actuator Upgrade

A larger capacity actuator can be utilized if the reaction frame is sufficiently stiff such that deflection is not an issue under the maximum actuator load. Deflection criteria will have to be determined upon the individual researcher's requirements and/or tolerance. Additionally, the force may be applied closer to the column-to-brace connection, which decreases deflection in the reaction frame, but increases shear demand on the members and connections. Based off the beam's shear demand to capacity ratio being closest to 1.0 (not including sandwich plate or deflection), the maximum capacity of the frame is upwards of 60 kips if the reaction frames are stiffened and the installation of a sandwich plate with a larger capacity (Figure 8, Option B).

4.5.4 Considerations

As a result of the additional braces, loads will be redistributed. Since there will be two load paths, the stiffer brace member will experience larger forces. The redistribution of forces is dependent on the distance and stiffness of each brace is from the applied load.

Additionally, with the new brace the frame system behavior changes and different limit states become a concern. To finalize the design of the reaction frame upgrade it would be necessary complete a new analysis of the system with similar limit states of the existing brace. These demand and capacity calculations (Appendix A.2) can be utilized to determine demand and capacity values for the upgraded system. New members should be designed considering these critical limit states. Member rupture and bolt tear out were concerns with the existing brace member. Member rupture will not be a concern for any of these suggested brace upgrade options due to the increase in cross-sectional area and bolt tear out will be accommodated by using larger gusset plates. The new critical limit state will likely be prying action in the bolted connections to the column and floor beam.

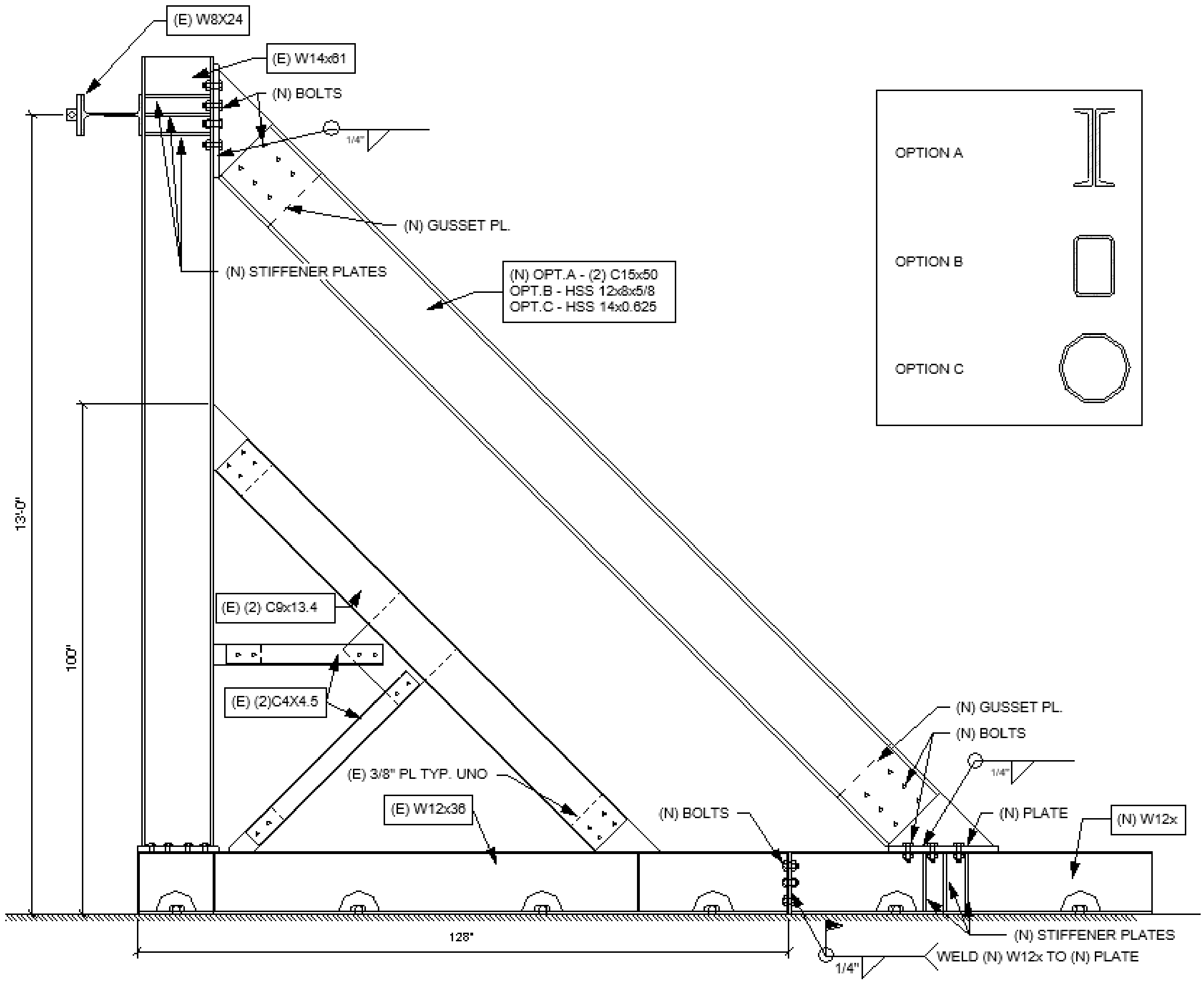


Figure 11 : Elevation of Proposed Stiffening Retrofit

4.6 Deflection and Drift

The upper bound deflection for the frame should correspond to the ultimate deflection of the concrete wall. However, the inelastic deformation response of the concrete wall specimen has not yet been calculated since the design of these specimens has not been finalized. Therefore, the values in Table 1 are representative of the cracked limit state.

The calculated drift in this report, with respect to the cracked limit state of the concrete wall, is 0.0865%, but does not represent the anticipated ultimate drift capacity of the wall. Birely (2011) examines the ultimate drift capacity of 70+ planar concrete walls with various design parameters. Specifically the walls that similar to those in the proposed research from Section 1.4 with a low boundary element reinforcing ratio (average drift of 3.1%), low cross-sectional aspect ratio (CSAR) around 10 (average drift is 1.5%), and low vertical reinforcing ratio (average drift is 3.0%). Based on these results, the expected drift ratio for the proposed concrete wall specimens will be around 2.5 to 3.0% because reinforcing steel in the boundary element and web are believed to greatly affect the deflection. To be conservative, the deflection associated with a 3.0% drift (4.68-in) should be considered the upper bound deflection for the frame.

4.7 Stress Fatigue

The cyclic loads applied to this structure are quasi-static and consists of a low number of cycles. high testing tress sfatigue in materials usually experience a minimum of 10,000 cycles. This was not a concern due to the low number of testing cycles that frame is expected to have experienced and will experience over the years.

5.0 Conclusion

5.1 Critical Limit States

As mentioned in Section 4.3, the critical limit states determined in this analysis of the High Bay Laboratory steel reaction frame are shear in the horizontal beam and deflection in the overall system. To resolve the first concern Sandwich plate – option A should be replaced with option B for increased capacity. To mitigate effects of deflection, a stiffening schematic was provided in Section 4.5.

5.2 Proposed Upgrades

As described in Section 4.5, the goal with these adding new braces at a height of 13-ft from the ground is to ensure that the reaction frame will be considerably stiffer than test specimens. There is no specific criteria for deflection limits, therefore the stiffness of the frame was compared to the proposed concrete wall specimens. Since this is a proposed solution, capacities of the new braces have not been calculated. If the proposed solution is to be designed, capacities and demands in these members and connections will be determined accordingly.

The system may further be strengthened by using modern grade bolts, as mentioned in Section 4.5.2. However, this is not a critical issue and may be addressed if a larger capacity actuator is purchased.

5.3 Maximum Capacity

As discussed in Section 4.5.3, a larger capacity actuator or larger force may be applied to specimen if the stiffness is increased and sandwich plate are replaced. The frame's capacity is expected to increase to more than 60 kips if the proposed upgrades are made.

6.0 Acknowledgements

The authors of this report would like to thank Drs. Anahid Behrouzi and Peter Laursen of California State Polytechnic University – San Luis Obispo for their guidance on the analysis required to determine the capacity of the High Bay steel reaction frame and strong floor. The authors would also like to thank Professor Jill Nelson for her guidance in determination and analysis of limit states in the reaction frames. The authors would like to thank Vincent Pauschek and Professor Emeritus Sat Rihal for their assistance with the project.

7.0 References

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A. Appendix

A.1 RISA 3D Output

Member Primary Data

	Label	I Joint	J Joint	K Joint	Rotate(deg)	Section/Shape	Type	Design List	Material	Design Rules
1	M1	N1	N2			W14x61	Column	Wide Flange	A36 Gr.36	Typical
2	M2	N3	N4			2C9x13.4	VBrace	Channel	A36 Gr.36	Typical
3	M3	N1	N5			W12x40	Beam	Wide Flange	A36 Gr.36	Typical
4	M4	A1	A5			W12x40	Beam	Wide Flange	A36 Gr.36	Typical
5	M5	A4	A3			2C9x13.4	VBrace	Channel	A36 Gr.36	Typical
6	M6	A1	A2			W14x61	Column	Wide Flange	A36 Gr.36	Typical
7	M7	A8	N8			W8x24	Beam	Wide Flange	A36 Gr.36	Typical

Joint Boundary Conditions

	Joint Label	X [k/in]	Y [k/in]	Z [k/in]	X Rot.[k-ft/rad]	Y Rot.[k-ft/rad]	Z Rot.[k-ft/rad]
1	N1	Reaction	Reaction	Reaction			
2	N5	Reaction	Reaction	Reaction			
3	N6	Reaction	Reaction	Reaction			
4	N7	Reaction	Reaction	Reaction			
5	A1	Reaction	Reaction	Reaction			
6	A6	Reaction	Reaction	Reaction			
7	A7	Reaction	Reaction	Reaction			
8	A5	Reaction	Reaction	Reaction			

Joint Loads and Enforced Displacements (BLC 1 : Actuator)

	Joint Label	L,D,M	Direction	Magnitude[(k,k-ft), (in,rad), (k*s^2/f...
1	N17	L	X	23.6

Basic Load Cases

	BLC Description	Category	X Gravity	Y Gravity	Z Gravity	Joint	Point	Distributed	Area(Me...	Surface(P...
1	Actuator	None				1				

Load Combinations

	Description	Sol..	PD..	SR..	BLC	Fact..	BLC	Fact..	BLC	Fact..	BLC	Fact..	BLC	Fact..	BLC	Fact..	BLC	Fact..	BLC	Fact..	BLC	Fact..	BLC	Fact...
1	Actuator L...	Yes			1	1																		

Joint Reactions (By Combination)

	LC	Joint Label	X [k]	Y [k]	Z [k]	MX [k-ft]	MY [k-ft]	MZ [k-ft]
1	1	N1	9.932	-22.653	-.069	0	0	0
2	1	N5	0	-1.644	.005	0	0	0
3	1	N6	-6.209	6.675	.088	0	0	0
4	1	N7	-15.523	17.622	-.025	0	0	0
5	1	A1	9.932	-22.653	.069	0	0	0
6	1	A6	-6.209	6.675	-.088	0	0	0
7	1	A7	-15.523	17.622	.025	0	0	0
8	1	A5	0	-1.644	-.005	0	0	0
9	1	Totals:	-23.6	0	0			
10	1	COG (ft):	NC	NC	NC			

Maximum Member Section Forces

	LC	Member Lab...		Axial[k]	Loc[ft]	y Shear[k]	Loc[ft]	z Shear[k]	Loc[ft]	Torque[k-...	Loc[ft]	y-y Moment[...	Loc[ft]	z-z Moment[...	Loc[ft]
1	1	M1	m...	0	6.563	11.8	6.563	0	12.104	0	12.104	.005	0	64.162	6.563
2			min	-21.732	0	-9.932	0	0	0	-.222	6.563	-.001	11.958	0	12.104
3	1	M2	m...	30.734	0	0	0	0	0	-.007	0	0	0	0	0
4			min	30.734	0	0	0	0	0	-.007	0	0	0	0	0
5	1	M3	m...	15.523	6.563	5.754	4.047	.02	4.047	.005	0	.217	0	4.855	7.547
6			min	-6.209	4.047	-15.978	6.563	-.069	0	0	6.563	-.057	4.047	-10.776	6.453
7	1	M4	m...	15.523	6.563	5.754	4.047	.069	0	0	6.563	.057	4.047	4.855	7.547
8			min	-6.209	4.047	-15.978	6.563	-.02	4.047	-.005	0	-.217	0	-10.776	6.453
9	1	M5	m...	30.734	0	0	0	0	0	.007	0	0	0	0	0
10			min	30.734	0	0	0	0	0	.007	0	0	0	0	0
11	1	M6	m...	0	6.563	11.8	6.563	0	0	.222	6.563	.001	11.958	64.162	6.563
12			min	-21.732	0	-9.932	0	0	12.104	0	12.104	-.005	0	0	12.104

Maximum Member Section Forces (Continued)

	LC	Member Lab...		Axial[k]	Loc[ft]	y Shear[k]	Loc[ft]	z Shear[k]	Loc[ft]	Torque[k-...	Loc[ft]	y-y Moment[...	Loc[ft]	z-z Moment[...	Loc[ft]
13	1	M7	m...	0	0	0	0	11.8	1.5	0	0	.222	0	-.001	0
14			min	0	0	0	0	-11.8	0	0	0	-17.478	1.5	-.001	0

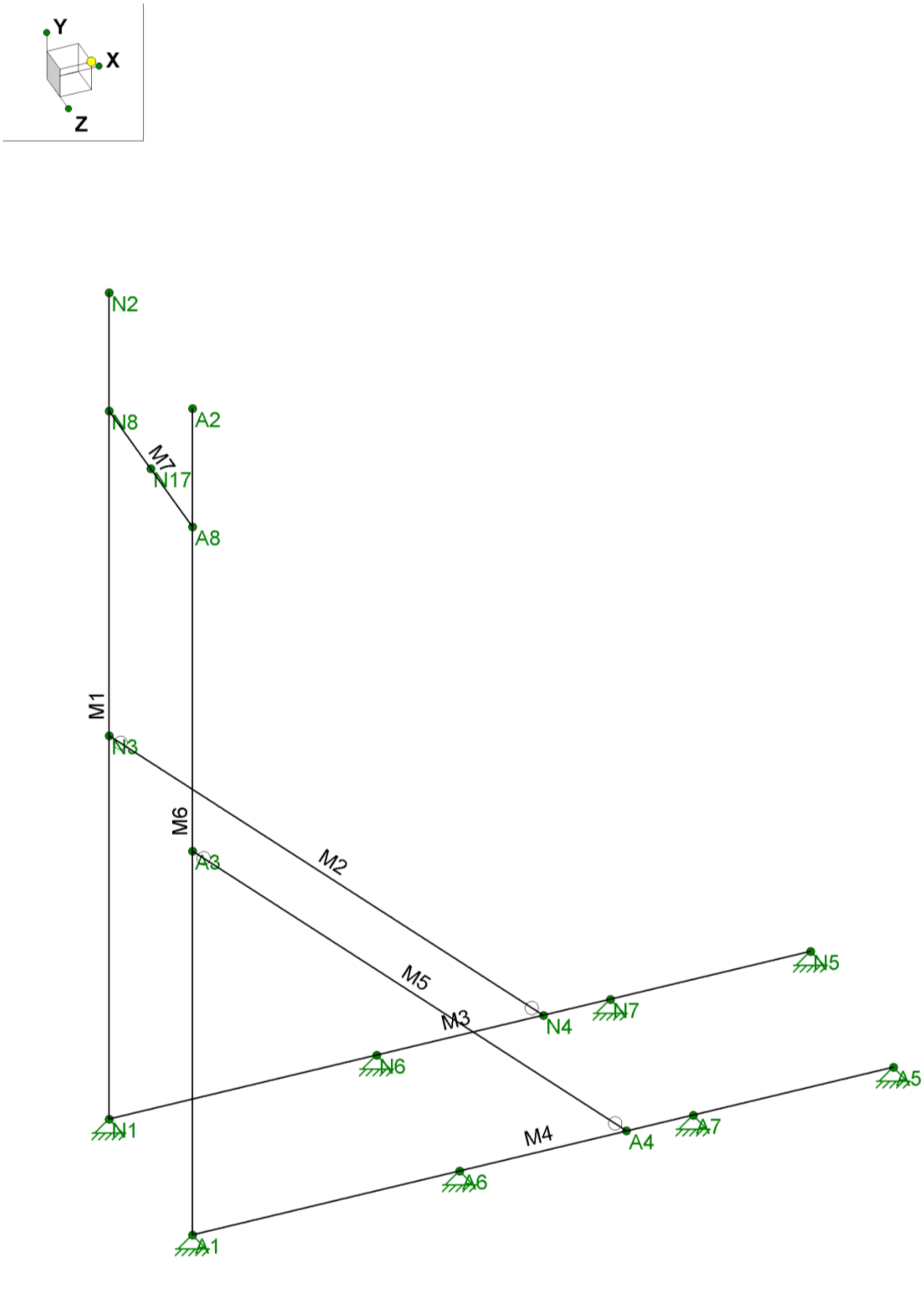
Member Section Forces

	LC	Member Label	Sec	Axial[k]	y Shear[k]	z Shear[k]	Torque[k-ft]	y-y Moment[k-ft]	z-z Moment[k-ft]
1	1	M1	1	-21.732	-9.932	0	-.217	.005	.342
2			2	-21.732	-9.932	0	-.217	.005	35.104
3			3	0	11.8	0	-.222	0	59
4			4	0	11.8	0	-.222	-.001	17.7
5			5	0	0	0	0	0	0
6	1	M2	1	30.734	0	0	-.007	0	0
7			2	30.734	0	0	-.007	0	0
8			3	30.734	0	0	-.007	0	0
9			4	30.734	0	0	-.007	0	0
10			5	30.734	0	0	-.007	0	0
11	1	M3	1	0	-.921	-.069	.005	.217	-.342
12			2	0	-.921	-.069	.005	.037	2.075
13			3	-6.209	5.754	.02	.005	-.034	-3.852
14			4	0	1.644	-.005	0	.014	4.316
15			5	0	1.644	-.005	0	0	0
16	1	M4	1	0	-.921	.069	-.005	-.217	-.342
17			2	0	-.921	.069	-.005	-.037	2.075
18			3	-6.209	5.754	-.02	-.005	.034	-3.852
19			4	0	1.644	.005	0	-.014	4.316
20			5	0	1.644	.005	0	0	0
21	1	M5	1	30.734	0	0	.007	0	0
22			2	30.734	0	0	.007	0	0
23			3	30.734	0	0	.007	0	0
24			4	30.734	0	0	.007	0	0
25			5	30.734	0	0	.007	0	0
26	1	M6	1	-21.732	-9.932	0	.217	-.005	.342
27			2	-21.732	-9.932	0	.217	-.005	35.104
28			3	0	11.8	0	.222	0	59
29			4	0	11.8	0	.222	.001	17.7
30			5	0	0	0	0	0	0
31	1	M7	1	0	0	-11.8	0	.222	-.001
32			2	0	0	-11.8	0	-8.628	-.001
33			3	0	0	11.8	0	-17.478	-.001
34			4	0	0	11.8	0	-8.628	-.001
35			5	0	0	11.8	0	.222	-.001

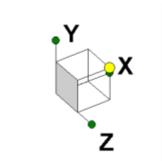
Joint Deflections (By Combination)

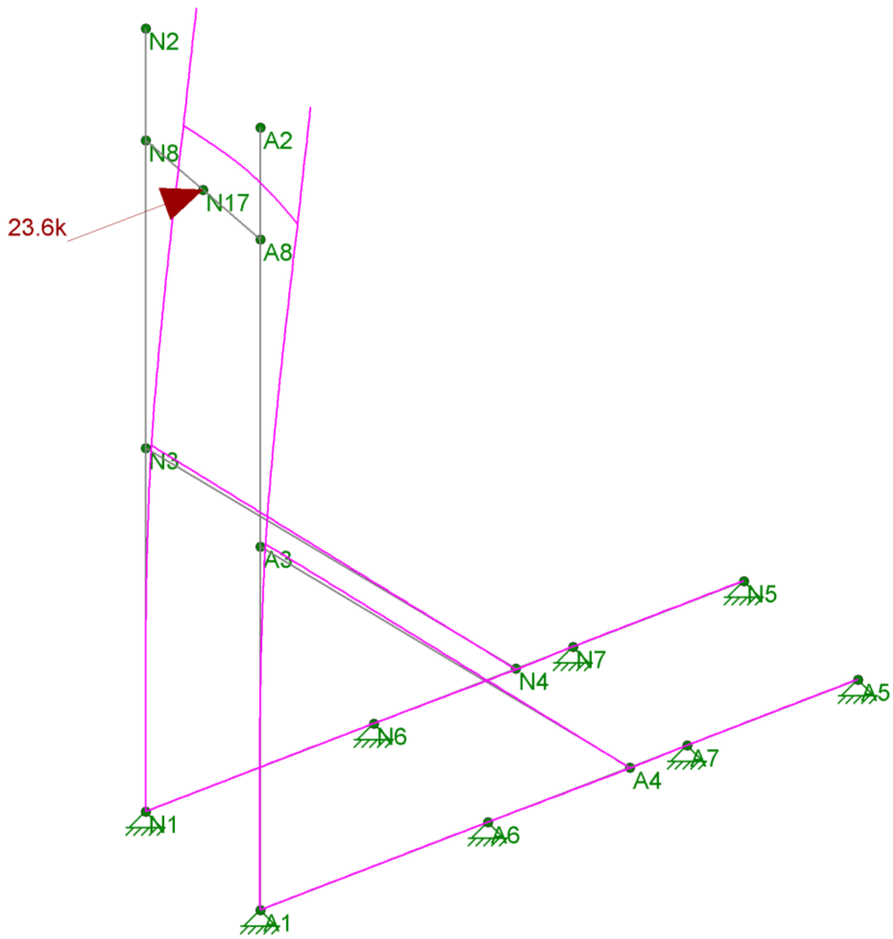
	LC	Joint Label	X [in]	Y [in]	Z [in]	X Rotation [rad]	Y Rotation [rad]	Z Rotation [rad]
1	1	N1	0	0	0	-1.195e-06	3.612e-05	1.021e-05
2	1	N2	.355	.004	0	1.652e-07	4.389e-03	-3.777e-03
3	1	N3	.038	.004	0	5.262e-07	2.37e-03	-2.046e-03
4	1	N4	0	-.007	0	-1.076e-04	3.026e-06	8.366e-05
5	1	N5	0	0	0	-1.076e-04	-1.031e-06	-4.127e-06
6	1	N6	0	0	0	-6.669e-05	-8.774e-06	-1.111e-04
7	1	N7	0	0	0	-1.076e-04	2.229e-06	1.455e-04
8	1	N8	.264	.004	0	1.652e-07	4.389e-03	-3.777e-03
9	1	A1	0	0	0	1.195e-06	-3.612e-05	1.021e-05
10	1	A2	.355	.004	0	-1.652e-07	-4.389e-03	-3.777e-03
11	1	A3	.038	.004	0	-5.262e-07	-2.37e-03	-2.046e-03
12	1	A4	0	-.007	0	1.076e-04	-3.026e-06	8.366e-05
13	1	A5	0	0	0	1.076e-04	1.031e-06	-4.127e-06
14	1	A6	0	0	0	6.669e-05	8.774e-06	-1.111e-04
15	1	A7	0	0	0	1.076e-04	-2.229e-06	1.455e-04
16	1	A8	.264	.004	0	-1.652e-07	-4.389e-03	-3.777e-03
17	1	N17	.321	.004	0	0	0	-3.777e-03

A.1.1 Member and Joint Labels

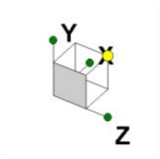


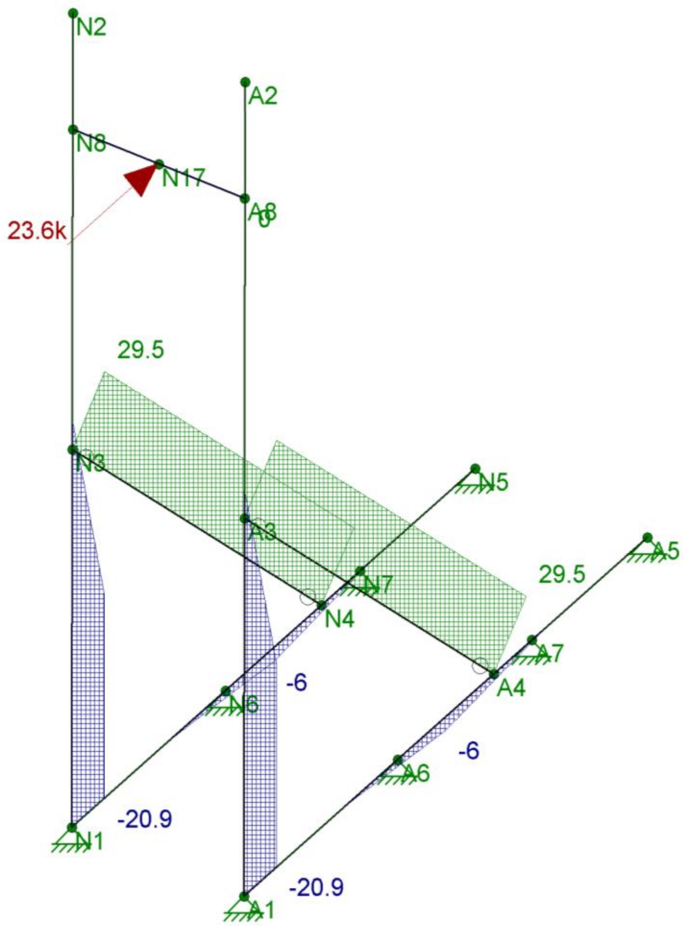
A.1.2 Deflected Shape



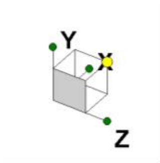


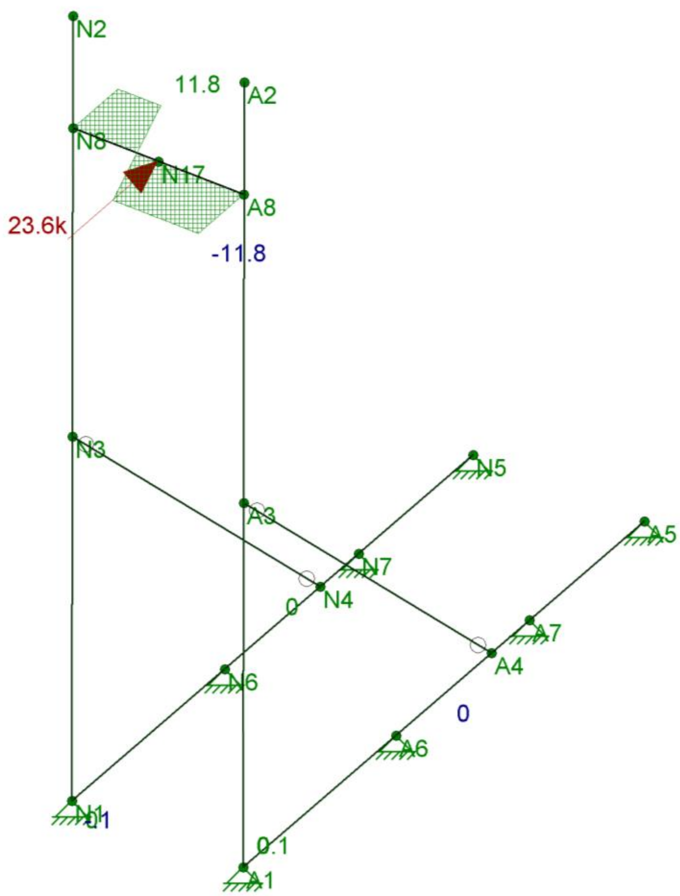
A.1.3 Axial Force Diagram



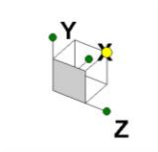


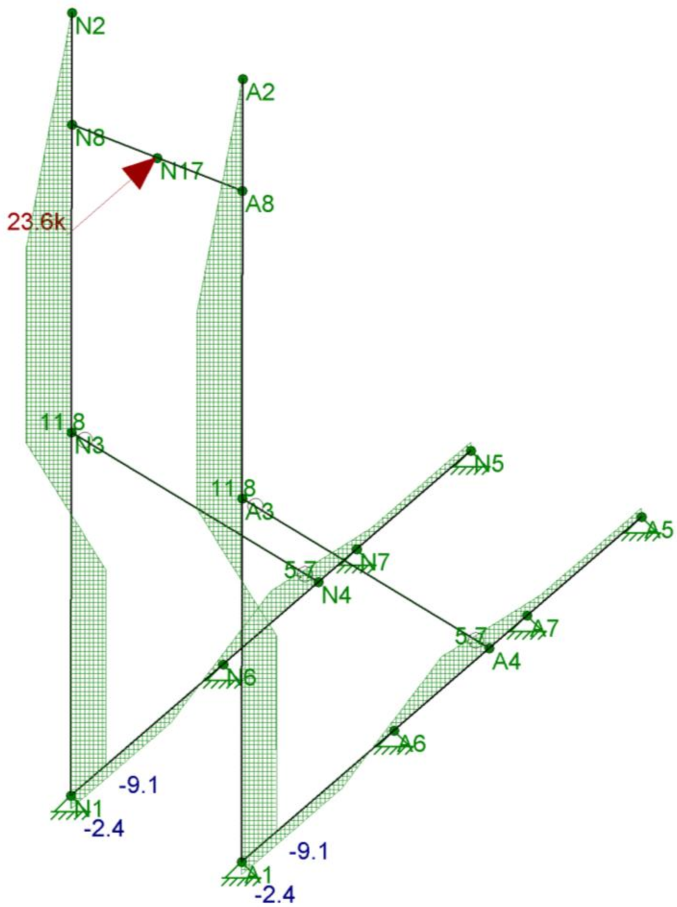
A.1.4 Shear Force Diagram (Along Z-Axis)



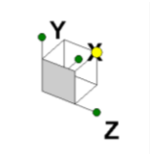


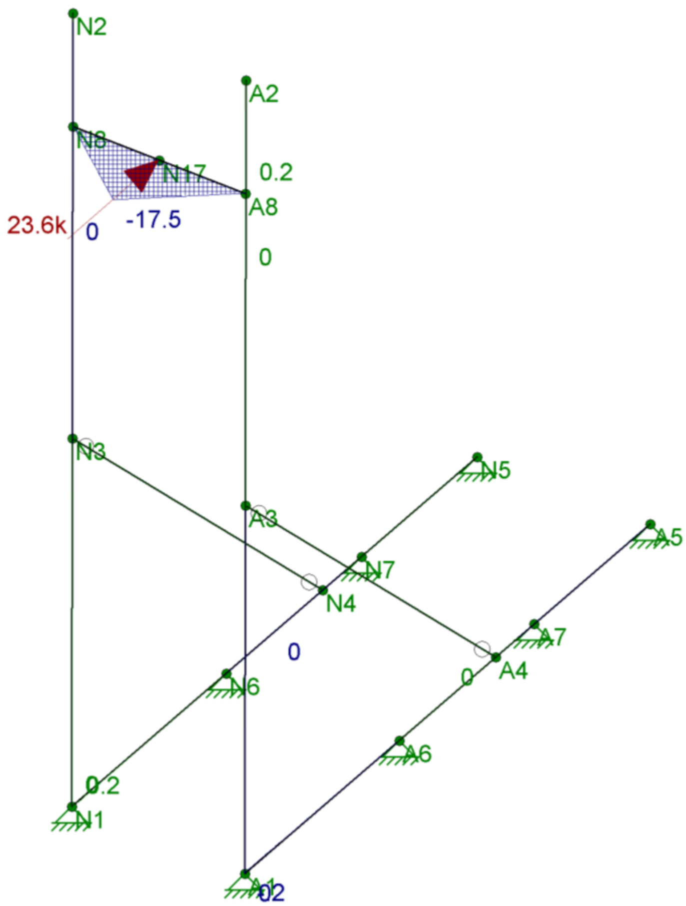
A.1.5 Shear Force Diagram (Along Y-Axis)



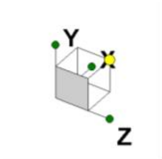


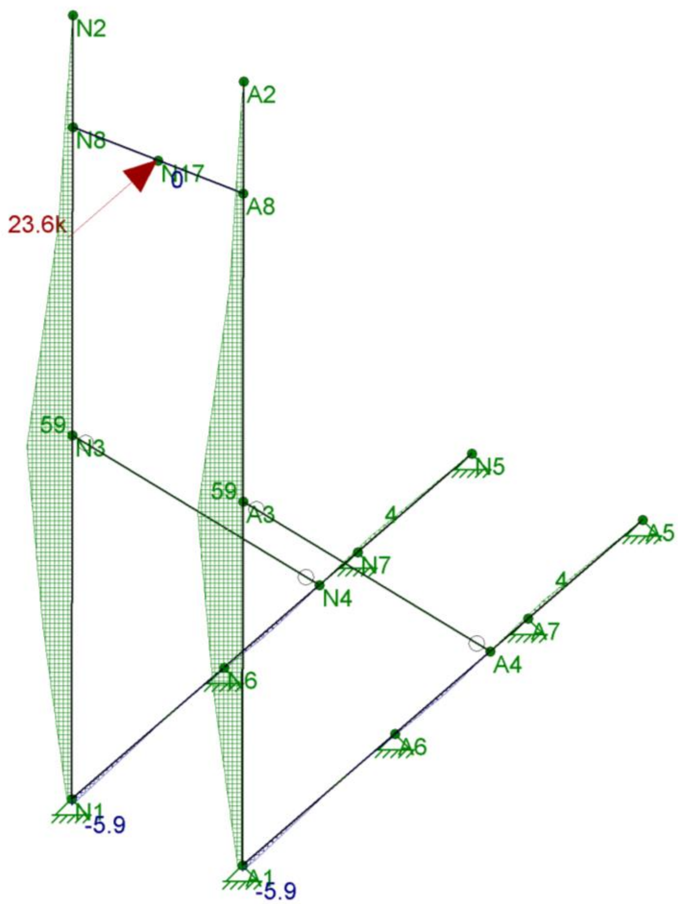
A.1.6 Moment Diagram (Y-Y Axis)





A.1.7 Moment Diagram (Z-Z Axis)

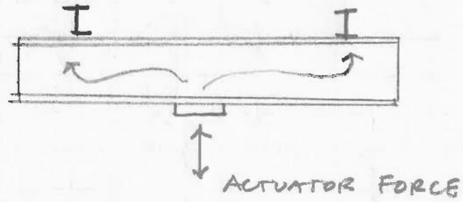




A.2 Calculations

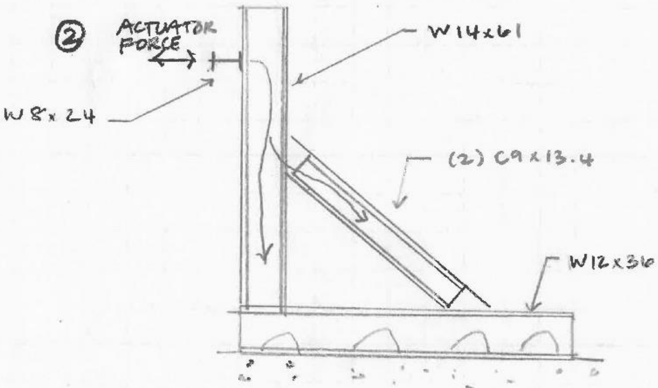
A.2.1 Load Flow

1



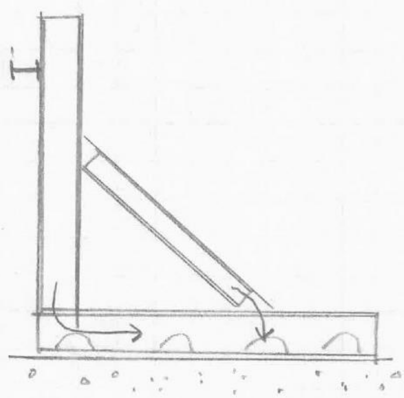
ACTUATOR FORCE INTO BEAM, WHICH TRANSFERS TO COLUMNS THROUGH SHEAR AND MOMENT.

2



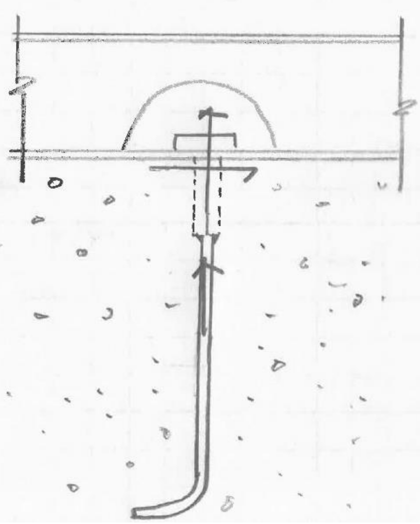
SHEAR, MOMENT, AND AXIAL FORCES IN THE COLUMN. AXIAL FORCES ONLY IN BRACE.

3



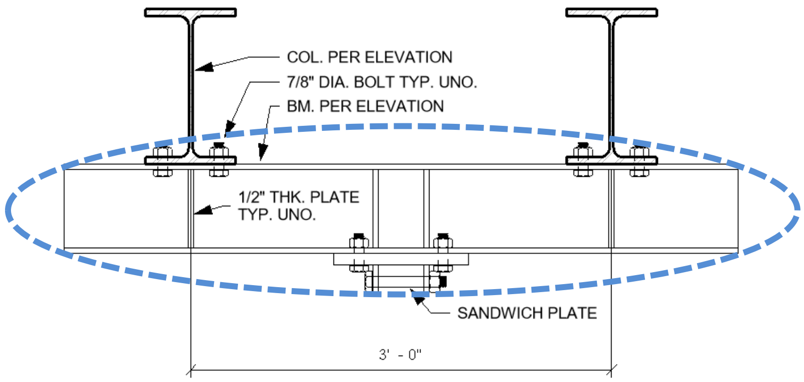
AXIAL AND SHEAR FORCES IN FLOOR BEAM.

4



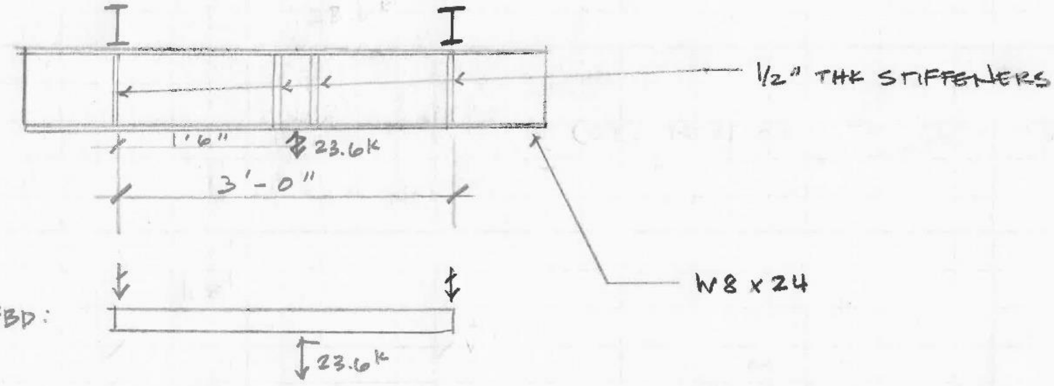
SHEAR AND AXIAL FORCES IN FLOOR BOLT. AXIAL FORCES IN HOOKED REBAR, WHICH IS DEVELOPED INTO THE STRONG FLOOR.

A.2.2 Horizontal Beam and Connections



HORIZONTAL BEAM BETWEEN REACTION FRAMES

FBD:



$V_u = P/2 = 11.8^k$

$M_u = \frac{P\ell}{4} = 17.7^{k \cdot FT}$

FLEXURE: PER AISC EQN. F2-1

THE CROSS-SECTION IS COMPACT. LATERAL TORSIONAL BUCKLING NOT CHECKED BECAUSE 3' SPAN.

$M_n = M_p = F_y Z_x$
$= 36ksi * 23.1 in^3$
$= 69.3 k \cdot FT$

$\boxed{M_n = 69.3 k \cdot FT > M_u = 17.7 k \cdot FT}$ ok

SHEAR: PER AISC EQN. E2-1

$V_n = 0.6 F_y A_w C_v$
$= 0.6 (36ksi) * (6\frac{1}{8}\text{" DEEP} * 0.245\text{" THK}) * 1.0$

$h/t_w = 25.9 \leq 2.24\sqrt{29000/36} = 63.6$ ✓
$\therefore C_v = 1.0$

$\boxed{V_n = 32.4^k > V_u = 11.8^k}$ ok

DEFLECTION: PER AISC T. 3-23

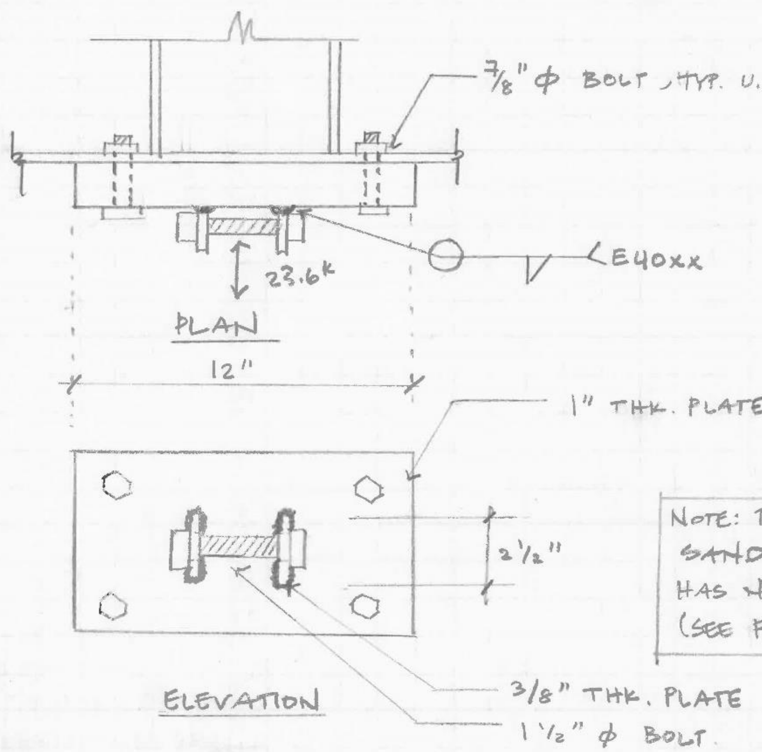
$\Delta_{ELASTIC} = \frac{P\ell^3}{48EI} = \frac{23.6^k (3' * 12''/1')^3}{48(29000 ksi)(82.5 in^4)} = 0.00959''$

$\boxed{\Delta_{beam} = 0.00959''}$

K_{beam} = 11.8k/0.00959 in
= 1230.5 k/in

K_{wall}= 174.7 k/in (Ref. Sec. A.2.7) < K_{beam} =1230.5 k/in **OK**

SANDWICH PLATE BETWEEN HORIZONTAL BEAM AND ACTUATOR



NOTE: THERE IS ANOTHER SANDWICH PLATE, WHICH HAS NOT BEEN EVALUATED. (SEE FIG. 4)

WELD IN TENSION

PER AISC EQN J2-4

$$R_n = 0.60\, F_{EXX} \left(\sqrt{2}/2\right)\left(D/16\right) l$$
$$= 0.60\,(40\text{ ksi})\left(\sqrt{2}/2\right)\left(4''/16\right)\left(2\tfrac{1}{2}'' + 3/8''\right) * 2 \text{ SIDES}$$
$$= 24.4^k$$

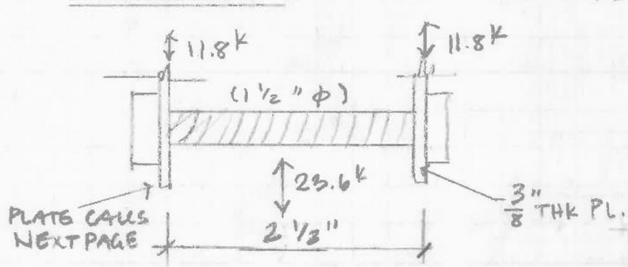
R_n ALLOWED TO BE MULTIPLIED BY $(1.0 + 0.50 \sin^{1.5}\theta)$ IF LOADED AT AN ANGLE.

$$R_n = 24.4^k\left(1.0 + 0.50 \sin^{1.5}(90)\right)$$

$\boxed{R_n = 36.6^k > R_u = 23.6^k/2 \text{ WELDS} = 11.8^k}$ OK

BOLT SHEAR

PER AISC T. J3.2

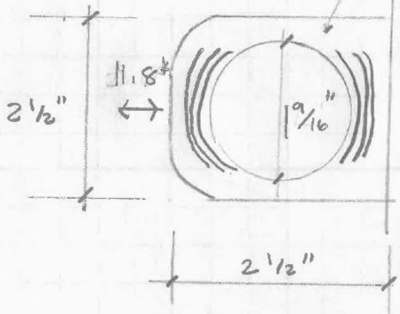


$F_{nv} =$

$$F_{uv} = \frac{V}{A} = \frac{23.6^k}{\frac{\pi (1\tfrac{1}{2}'')^2}{4}} = 13.4 \text{ ksi}$$

$\boxed{F_{nv}\ 54 \text{ ksi} > F_{uv} = 13.4 \text{ ksi}}$ OK

BOLT BEARING

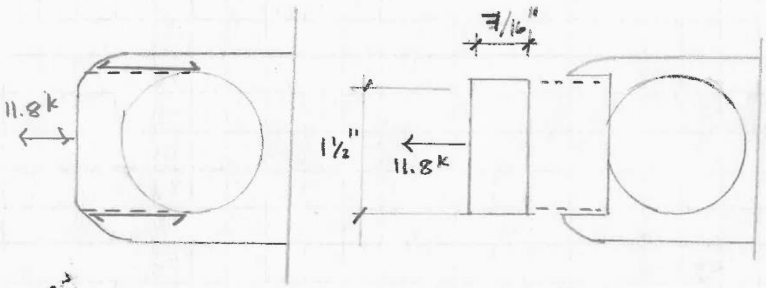


$$F_B = \frac{P}{t\,d_{HOLE}} = \frac{11.8^k}{(3/8")(1\,1/2" + 1/16")} = 20.1\text{ ksi}$$

$$F_{ALL.} = 36\text{ksi}$$

$$\boxed{F_{ALL} = 36\text{ ksi} > F_B = 20.1\text{ksi}}\ \underline{\text{ok}}$$

BOLT TEAR OUT



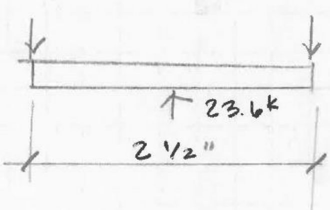
APPROXIMATED
TEAR OUT

$$V_{TO} = 11.8^k$$

$$V_{ALL} = 2\text{ SIDES } (3/8" \times 7/16") * 36\text{ksi}$$
$$= 41.8^k$$

$$\boxed{V_{ALL} = 11.8^k \geq V_{T.O.} = 11.8^k}$$ NOT GOOD

BOLT BENDING

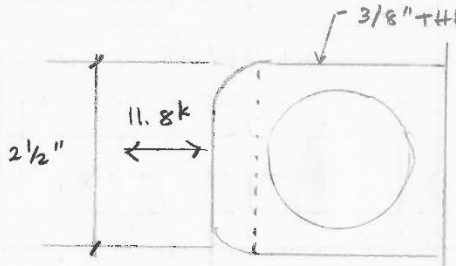


$$M_u = \frac{P\ell}{4} = \frac{23.6^k(2\,1/2")}{4} = 14.8^{k"}$$

$$f_B = \frac{M}{S} = \frac{14.8^{k"}}{\frac{\pi(1\,1/2"\phi)^3}{32}} = 0.04\text{ ksi} = 43.6\text{ psi}$$

$$\boxed{f_{B,ALL} = 54\text{ ksi} > f_B = 0.04\text{ksi}}\ \underline{\text{ok}}$$

PLATE YIELDING

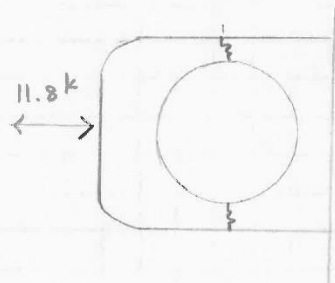


$P_{ALL} = A_g * F_y = 2\frac{1}{2}"(3/8") * 36ksi$

$P_{ALL} = 33.8k$

$\boxed{P_{ALL} = 33.8^k > P_u = 11.8^k}$ ok

PLATE RUPTURING

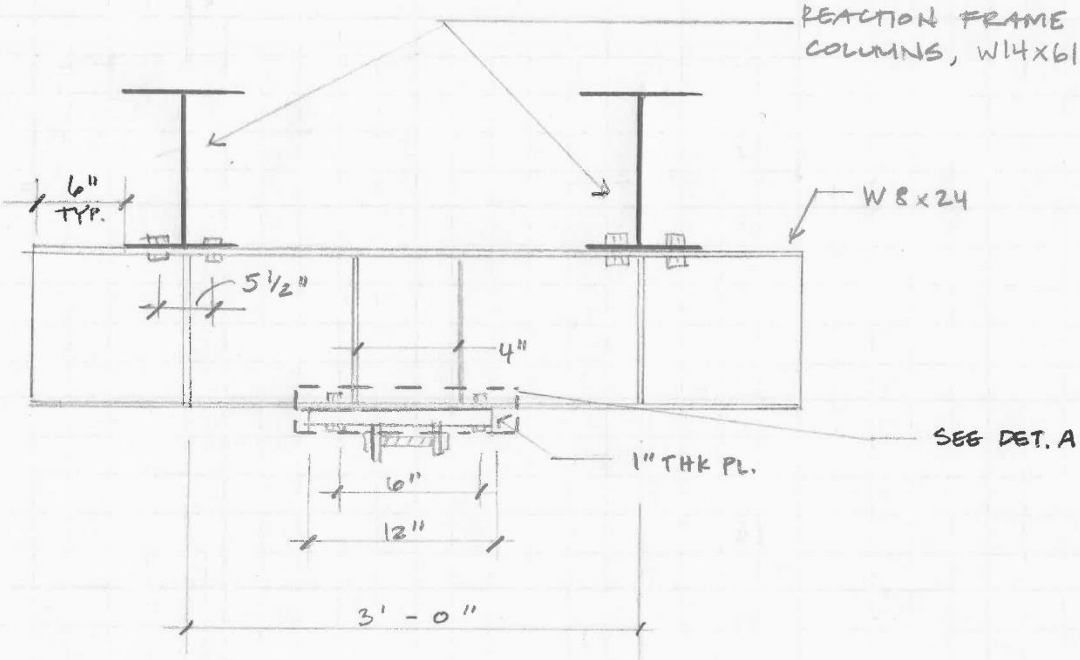


$P_{ALL} = A_{net} * F_u = (2\frac{1}{2}" - 1\frac{1}{2}" - 1/16")(3/8")(62ksi)$

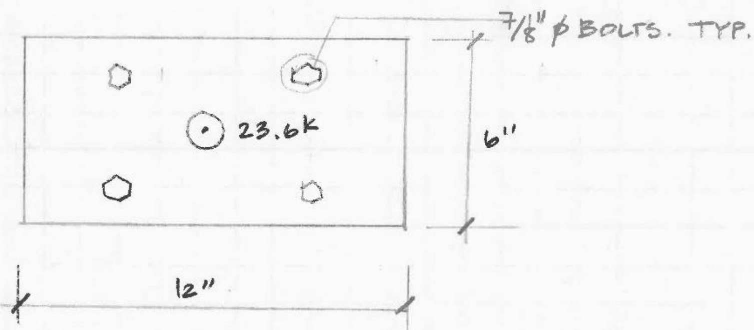
$= 21.8k$

$\boxed{P_{ALL} = 21.8^k > P_u = 11.8^k}$ ok

HORIZONTAL BEAM AND REACTION FRAME CONNECTION



PLAN OF HORIZ. BEAM



DET. A ELEVATION

DET. A SECTION

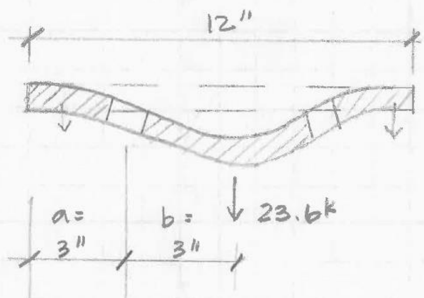
BOLT TENSION: PER AISC 7TH EDITION

$F_{nt} = 40\,ksi$

$$F_{ut} = \frac{23.6^k}{4 \text{ BOLTS}} \Big/ \frac{\pi(7/8)^2}{4} = 9.81\,ksi$$

$\boxed{F_{nt} = 40ksi > F_{ut} = 9.81ksi}$ OK

PRYING ACTION:



REF. EX 7.9 SEGUI
FIFTH EDITION STEEL DESIGN

$$b' = b - d/2 = 3" - \frac{7/8"}{2} = 2.56"$$

$$a' = a + d/2 = 3.48"$$

$$d' = d + 1/8 = 1"$$

p = trib width of bolt.

$$\delta = 1 - \frac{d'}{p} = 1 - 1/6 = 5/6$$

ASSUME $\alpha = 1.0$ (WORST CASE)

$$B_c = \text{BOLT FORCE} = T\left[1 + \frac{\delta\alpha * b'}{(1+\delta\alpha)\,a'}\right]$$

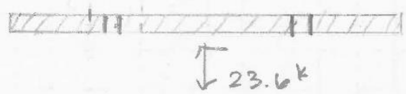
$$= \frac{23.6^k}{4}\left[1 + \frac{5/6(1.0) * 2.56"}{1 + 5/6(1.0) * 3.48"}\right]$$

$$B_c = 7.87^k$$

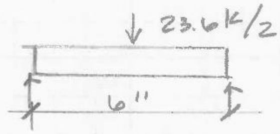
$$F_{vp} = 7.87^k \Big/ \frac{\pi(7/8")^2}{4} = 13.1\,ksi$$

$\boxed{F_{nt} = 40ksi > F_{up} = 13.1ksi}$ OK

PLATE BENDING:



FBD:



THERE ARE 4 BOLTS. CONSIDERING TRIBUTARY AREA OF 2 BOLTS

$$M_u = \frac{P\ell}{4} = \frac{11.8^k(6")}{4} = 17.7\ k"$$

$$f_b = M/S = 17.7k" / \left(6" \times 1"^2/6\right) = 17.7\ ksi$$

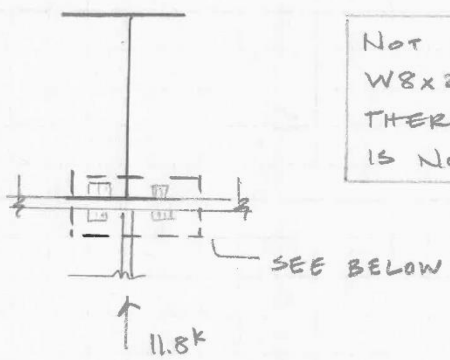
$F_b = 36 ksi$ (NORMAL STRESS)

$$\boxed{F_b = 36 ksi > f_b = 17.7 ksi}$$ ok

PLATE SHEAR:

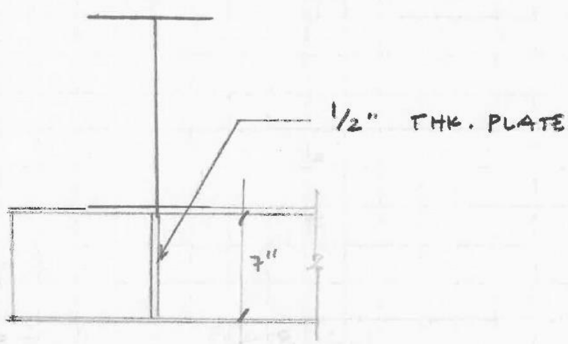
$\boxed{\text{NOT CONCERNED. SLENDER ELEMENT. SHEAR WON'T GOVERN}\quad h/t_w = 6/1 = 6}$

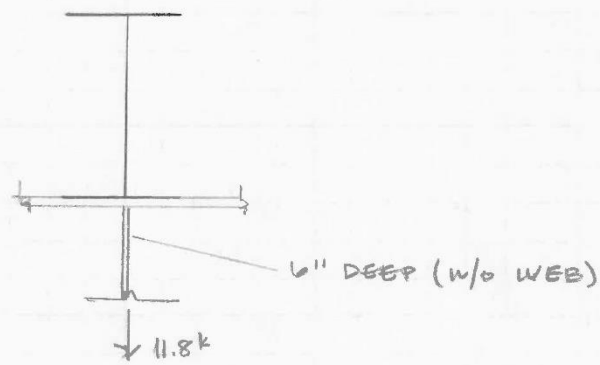
STIFFENER BUCKLING:



NOT CONCERNED ABOUT BUCKLING. W8x24 IS A COMPACT SECTION, THEREFORE LOCAL WEB BUCKLING IS NOT LIKELY TO OCCUR.

STIFFENER YIELDING:



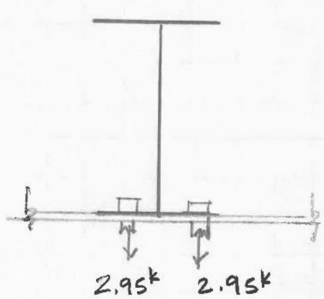


$F_y = 36\text{ksi}$

$F_u = 11.8^k / (6'' \times 1/2'') = 3.9\text{ksi}$

$\boxed{F_y = 36\text{ksi} > F_u = 3.9\text{ksi}}$ OK

BOLT TENSION

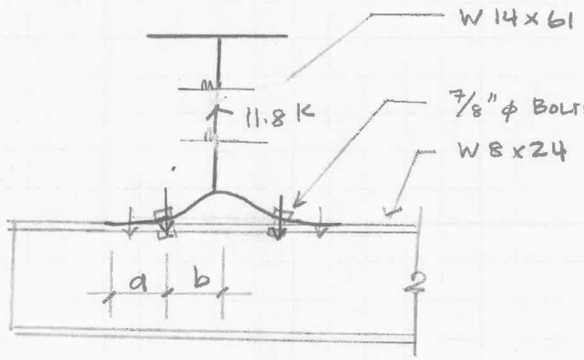


$P_u = 11.8^k / 4 \text{ BOLTS} = 2.95^k$

$F_u = 2.95^k \Big/ \left(\dfrac{\pi (7/8)^2}{4} \right) = 4.91\text{ksi}$

$\boxed{F_{nt} = 90\text{ksi} > F_u = 4.91\text{ksi}}$ OK

PRYING ACTION



PER SEGUI EX. 7.9
FIFTH EDITION STEEL DESIGN

$a = 2\ 7/16"$ $\quad a' = a + d/2 = 2\ 7/16 + 7/8/2 = 2.88"$

$b = 2\ 1/2"$ $\quad b' = b - d/2 = 2.06"$

$d' = d + 1/8 = 1"$

p = TRIB WIDTH OF BOLT = 4.94"

$\delta = 1 - d'/p = 1 - 1/4.94" = 0.797"$

$$Bc = T\left[1 + \frac{\delta\alpha}{(1+\delta\alpha)} * \frac{b'}{a'}\right]$$

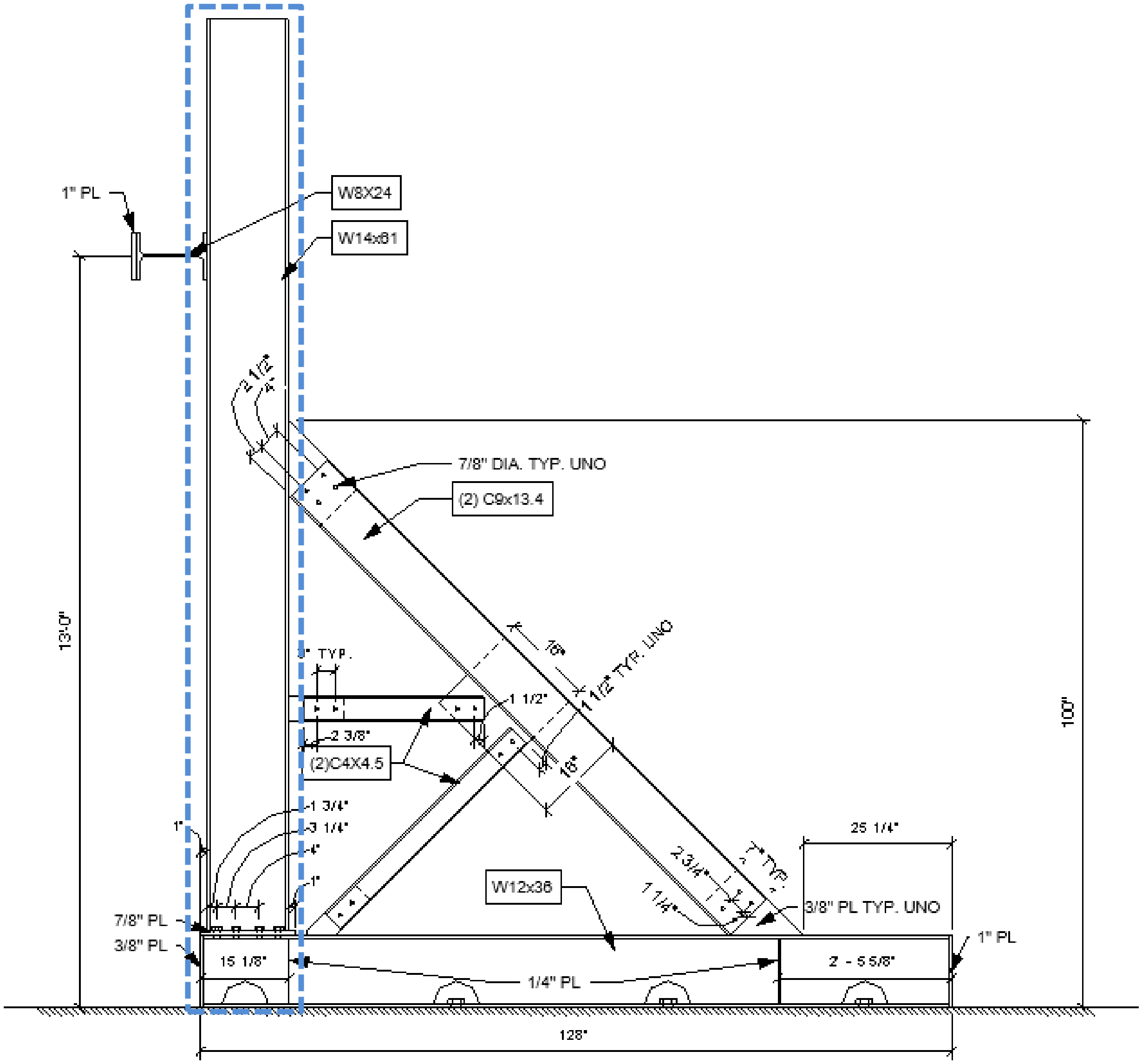
$$= 11.8^k/4\text{ BOLTS}\left[1 + \frac{0.797}{1+0.797} * \frac{2.06"}{2.88"}\right] = 3.88^k$$

$$F_{up} = 3.88^k \Big/ \frac{\pi(7/8)^2}{4} = 6.45\text{ ksi}$$

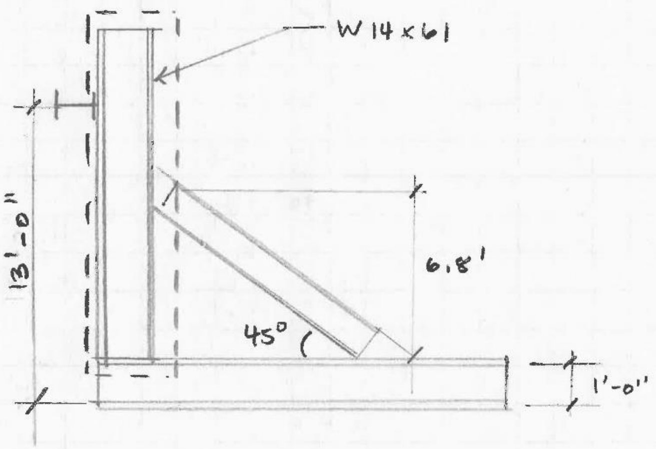
$\boxed{F_{nt} = 40\text{ksi} > F_{up} = 6.45\text{ksi}}$ OK

NOTE: BOLTS IN FRAME ARE ANALYZED W/ AISC. 7TH EDITION. BOLT STRENGTHS

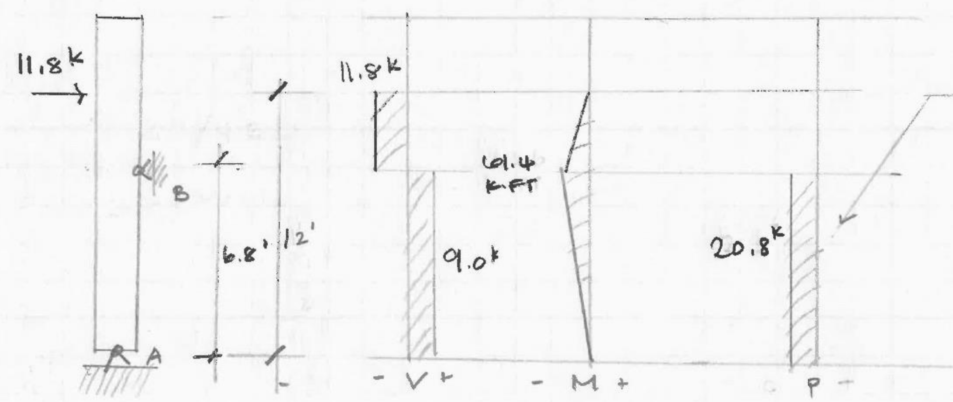
A.2.3 Reaction Frame Column and Connections



REACTION FRAME COLUMN



ELEVATION OF FRAME



SELF WT. IS NEGLECTED B/C MINIMAL WHEN COMPARED TO REACTION

STATICS:

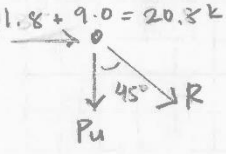
$$\circlearrowright \Sigma M_A = R_B(6.8') - 11.8^k(12') = 0$$

$$R_B = 20.8\,k\ (c)$$

$$R_A = 11.8^k - 20.8^k = -9.0^k\ (T)$$

$$M_{max} = 11.8^k(12' - 6.8') = 61.4\,k\cdot FT$$

$11.8 + 9.0 = 20.8^k$



$P_u = 20.8^k$ (45° ANGLE)

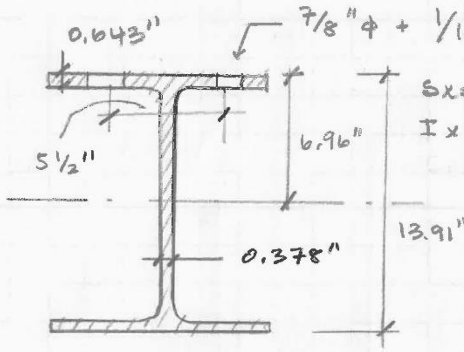
$R = 29.4\,k$

$V_{max} = 11.8^k$

$M_{max} = 61.4\,k\cdot FT$

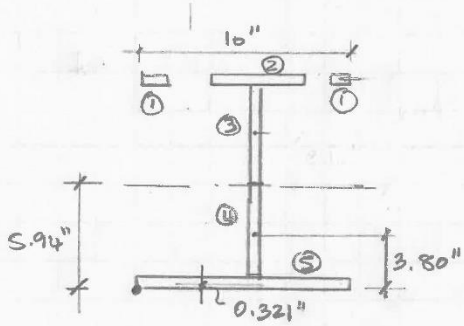
$P_{max} =$ 20.8 k

NET SECTION FLEXURE:



$S_x = 92.2 in^3$
$I_x = 641 in^4$

W14x61



① $A = 1.81" * 0.643" = 1.16 in^2$
$Ay = 1.16 in^2 (6.64) + 6.96) = 15.78 in^3$

② $A = 4.56" * 0.643" = 2.93 in^2$
$Ay = 2.93 in^2 (6.64" + 6.96") = 39.85 in^3$

③ $A = 0.378" * 6.32" = 2.39 in^2$
$Ay = 24.16 in^3$

④ $A = 2.39 in^2$
$Ay = 9.07 in^3$

⑤ $A = 10" * 0.643" = 6.43 in^2$
$Ay = 2.06 in^3$

$$N.A. = \frac{\Sigma yA}{\Sigma A} = \frac{(15.78 + 39.85 + 24.16 + 9.07 + 2.06)}{1.16 + 2.93 + 2.39 + 2.39 + 6.43} = 5.94"$$

$$I_e = \frac{(0.378")(12.62")^3}{12} + ((1.16 in^2) * 2 + (2.39 in^2))(7.65")^2 + 6.43" * (5.62")^2$$

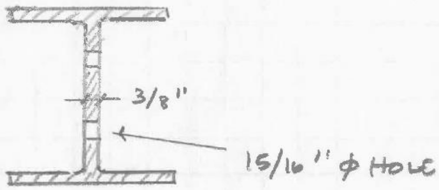
$I_e = 542 in^4$

$$S_e = \frac{542 in^4}{5.94} = 91.2 in^3$$

$M_n = F_y S_x = 36 ksi * 91.2 in^3 = 273.6 k.ft$

$\boxed{M_n = 273.6 k.ft > 61.46 k.ft}$ ok

NET SECTION SHEAR:



$V_n = 0.6\ F_y\ A_w\ C_v$

$= 0.6\ (36\,\text{ksi})\ (11\tfrac{1}{4}" - 2\,(15/16"))\ \tfrac{3}{8}"\ \text{THK} \quad * 1.0$

$\boxed{V_n = 75.9^k > V_{max} = 11.8^k}$ o.k.

FLEXURE: PER AISC EQN. F2-1

NOTE: THIS IS GROSS SECTION

$M_n = M_p = F_y Z_x$
$= 36\text{ksi}\,(102\text{ in}^3)$
$= 306\text{ k.FT}$

$\boxed{M_n = 306\text{ k.FT} > M_{max} = 61.4\text{ k.FT}}$ OK

SHEAR: PER AISC EQN E2-1

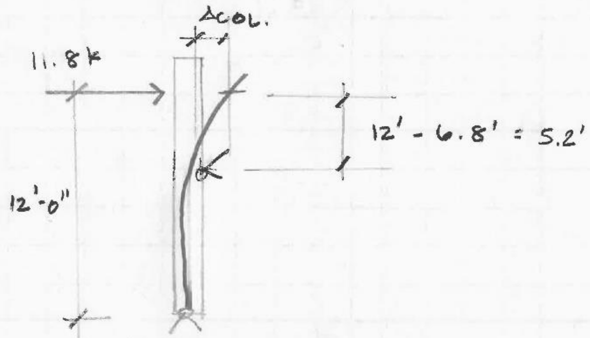
$V_n = 0.6\ F_y\ A_w\ C_v$
$= 0.6\,(36\text{ksi})\,(11\tfrac{1}{4}''\text{ DEEP} * 3/8''\text{ THK}) * 1.0$

$h/t_w = 30.4 \le 2.24\sqrt{29000/36} = 63.6$ ✓
$\therefore C_v = 1.0$

$\boxed{V_n = 79.0\text{k} > V_{max} = 11.8\text{ k}}$

DEFLECTION:

$$k_{COL} = \frac{3EI}{L^3} = \frac{3\,(29000\text{ ksi})\,(107\text{ in}^4)}{(5.2' * 12''/1')^3} = 38.3\text{ k/in}$$

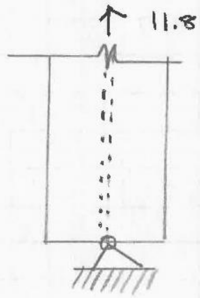


DEFLECTED SHAPE

$$\Delta_{COL.} = \frac{F}{k_{COL.}} = \frac{11.8\text{k}}{38.3\text{k/in}} = 0.308''$$

$\boxed{k_{COL} = 38.3\text{ k/in} < K_{wall} = 174.7\text{ k/in (Ref. Sec. A.2.7)}}$ NO GOOD

YIELDING: PER AISC EQN D2-1



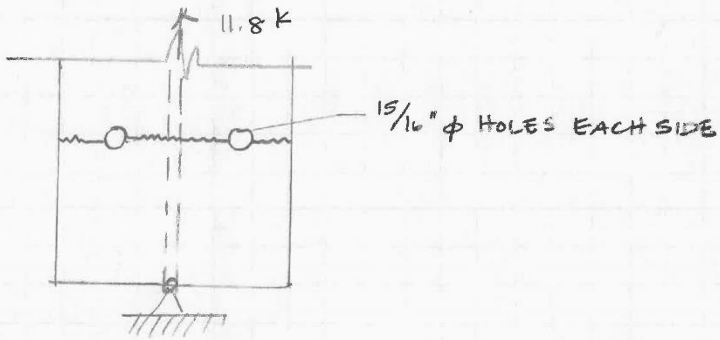
$P_n = F_y A_g$

$= (36\,ksi)(17.9\,in^2)$

$= 644\,k$

$\boxed{P_n = 644k > P_{max} = 20.8^k}$ ok

RUPTURE: PER AISC EQN. D2-2



$P_n = F_u A_e$

$= 62ksi\ (17.9\,in^2 - 4(0.643''\,THK * 15/16''\,\phi))$

$= 960.3^k$

$\boxed{P_n = 960.3^k > P_{max} = 20.8^k}$ ok

COMPRESSION: PER AISC EQN E3-1 /E3-2 /E3-4

$\frac{kL}{r} = \frac{1.0\,(6.8' * 12'')}{5.98''} = 13.6$

$4.71\sqrt{E/F_y} = 4.71\left(29000\,ksi/36\,ksi\right)^{1/2} = 133.6$

$13.6 < 133.6 \quad \therefore \quad F_{cr} = \left[0.658^{F_y/F_e}\right] F_y$

$F_e = \frac{\pi^2 E}{\left[kL/r\right]^2} = \frac{\pi^2\,(29000\,ksi)}{(13.6)^2} = 1547\,ksi$

$F_{cr} = 0.658^{(36/1547)} * 36 = 35.7\,ksi$

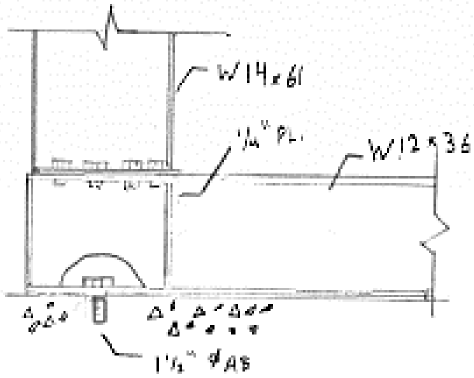
$P_n = F_{cr} A_g$

$= 35.7\,ksi\,(17.9\,in^2)$

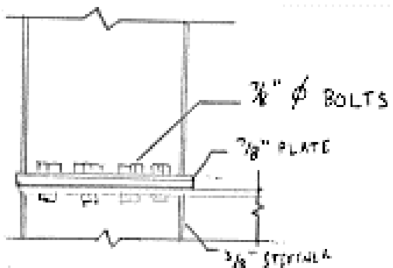
$= 639\,k$

$\boxed{P_n = 639^k > P_{max} = 20.8^k}$ ok

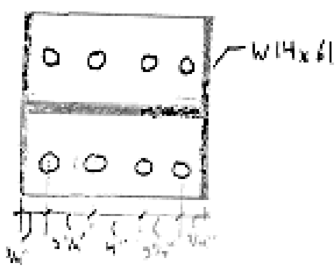
BOLTED CONNECTIONS ON COLUMN



1) BOLTS CONNECTING PLATE TO BEAM



ELEVATION VIEW

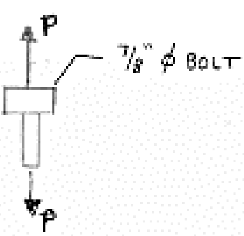


PLAN VIEW

FAILURE MODES: ① BOLT YIELDING (TENSION)
② BOLT SHEAR

* DIVIDE CAPACITY OF TENSILE CAPACITY BY 0.6 + SHEAR CAPACITY BY 0.4 TO FIND NOMINAL CAPACITY. *

1) YIELDING CALCULATION (TENSILE STRENGTH CAPACITY) — GRADE A325 BOLTS



7/8" ϕ BOLT : f_t (ALLOWABLE TENSILE STRESS) = 40 KSI
(PER AISC 360-7 ; Pg.4-3)

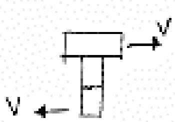
TENSILE CAPACITY (PER BOLT): 24.05 KIPS (TABLE ON Pg. 4-3; AISC 360-7)

CONNECTION CAPACITY = $(24.05^K \times 8 \text{ BOLTS}) / 0.6 = 320.67 \text{ KIPS} = P_{n_{BOLT}}$

$P_{u_{BOLT}} = 20.8^K$ (FROM REACTION FRAME (COLUMN CALCS Page)

$\boxed{P_{u_{BOLT}} = 20.8^K \leq P_{n_{BOLT}} = 321 \text{ KIPS}}$ D/C = .065

2) BOLT SHEAR (PER AISC 360-7)



7/8" ϕ BOLT : f_v = 15 KSI (TABLE ON Pg. 4-6; AISC 360-7)

SHEAR CAPACITY (PER BOLT) = 9.02 KIPS (TABLE ON Pg. 4-6; AISC 360-7)

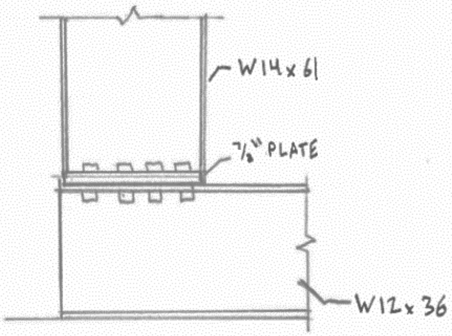
CONNECTION SHEAR CAPACITY $= (9.02 \text{ K} \times 8 \text{ BOLTS}) / 0.4$

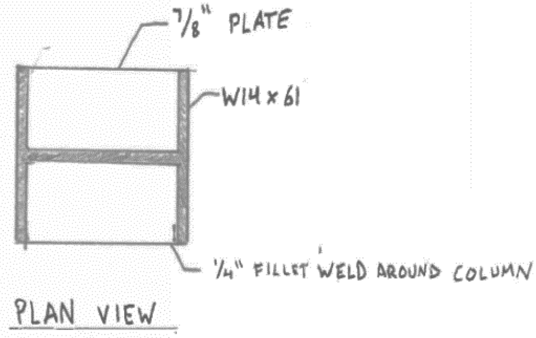
$V_{n_{BOLT}} = 180.4$ KIPS

$V_u = 9.0^K$ (FROM REACTION FRAME CALC PAGE)

$\boxed{V_u = 9.0^K < V_n = 180.4K \checkmark}$ D/C = 0.050

2) COLUMN WELDED TO PLATE





ASSUMPTIONS :
- WELD MATERIAL STRENGTH IS ASSUMED TO BE 40 KSI CONSERVATIVELY. (F_{EXX})
- THE TYPE OF WELD IS A 1/4" FILLET WELD
- PLATE MATERIAL STRENGTH 36 KSI . (A36 KSI)
- WELDS ARE PARTIAL JOINT PENETRATION WELDS.

FAILURE MODES : 1) SHEAR (WELD)
2) TENSION (WELD)
3) SHEAR OF BASE METAL

EQUATION : $R_n = F_{nw} A_{we}$ OR $R_n = F_{nBM} A_{BM}$ (J 2-3 ; AISC 360-10)

R_n = DESIGN STRENGTH
F_{nw} = NOMINAL STRESS OF THE WELD MATERIAL (KSI)
F_{nBM} = NOMINAL STRESS OF THE BASE MATERIAL (KSI)
A_{we} = EFFECTIVE AREA OF THE WELD (in²)
A_{BM} = CROSS-SECTIONAL AREA OF BASE METAL (in²)

1) SHEAR STRENGTH (WELD)

TABLE J2.5 : $F_{nw} = 0.6\, F_{exx} = 0.6\,(40 \text{ KSI}) = 24 \text{ KSI}$

A_{we} = effective length of weld x effective throat

= PERIMETER OF W14x61 (PER AISC 360-7 Pg. 1-16) x 1/4"

$= (2d + 2b_f) \times 1/4"$

$= (2 \times 13\text{-}7/8" + 2 \times 10") \times 1/4"$

$A_{we} = 11.9375 \text{ in}^2$

$R_{nv} = 24 \text{ KSI} \times 11.9375 \text{ in}^2$ $\qquad R_{nv} = 286.5$ KIPS

$R_{uv} = 9.0^k$ (FROM REACTION FRAME CALCS)

$$\boxed{R_{uv} = 9.0^k \le R_{nv} = 286.5^k}$$

D/C = .031

2) TENSION STRENGTH (WELD)

TABLE J2.5 : $F_{nw} = 0.6 F_{EXX} = 24$ KSI

A_{we} = SAME AS SHEAR STRENGTH

$A_{we} = 11.9375 \text{ in}^2$

$R_{np} = 24 \text{ KSI} \times 11.9375 \text{ in}^2$

$R_{np} = 286.5$ KIPS

$R_{up} = 20.8^K$ (AXIAL FORCE FROM REACTION FRAME COLUMN CALCS)

$\boxed{R_{up} = 20.8^K \leq R_{np} = 286.5^K}$

D/C = .0726

3) SHEAR OF BASE METAL (7/8" PLATE TO W14x61)

TABLE J2.5 : $F_{nBM} = F_u = 62$ KSI (ASCE-41)

A_{BM} = SEE AISC 360-10 : J4

J4 : SHEAR YIELDING : $R_n = 0.6 F_y t$ (J4-3)

where : A_{gv} = gross AREA SUBJECT TO SHEAR (in^2)

SHEAR RUPTURE : $R_n = 0.6 F_u t$ (J4-4)

Where : A_{nv} = NET AREA SUBJECT TO SHEAR (in^2)

SHEAR YIELDING : $R_n = 0.6 F_y t$ (J4-3)

$F_y = 36$ KSI

$t = 3/8"$ (CONSERVATIVE, TOOK SMALLEST THICKNESS FROM WEB OF COLUMN)

$R_n = 0.6(36 \text{ KSI})(3/8") = 8.1 \frac{K}{in}$

SHEAR RUPTURE : $R_n = 0.6 F_u t$ (J4-4)

$F_u = 62$ KSI

$t = 3/8"$ (SEE ABOVE)

$R_n = 0.6(62 \text{ KSI})(3/8") = 13.95 \frac{K}{in}$

SHEAR YIELDING = 8.1 K/in < RUPTURE = 13.95 K/in ∴ USE 8.1 K/in.

STRENGTH OF BASE MATERIAL CONNECTION : $R_n = 8.1 \text{ K/in} \times L_w$

L_w (length of weld) = 47.75 in

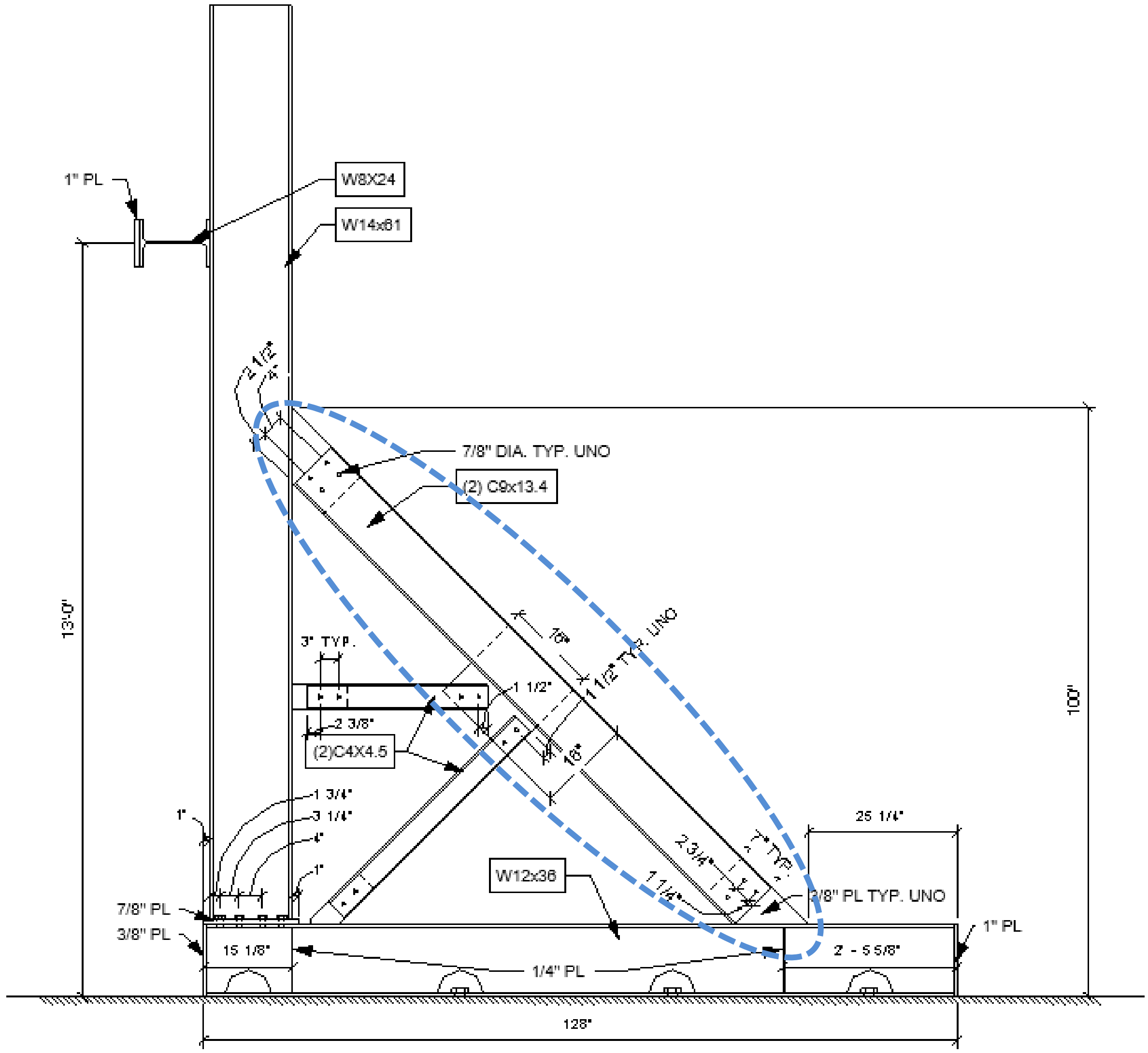
$R_n = 47.75 \text{ in} \times 8.1 \text{ K/in} = 386.78^K$

$R_{uv} = 9.0^K$

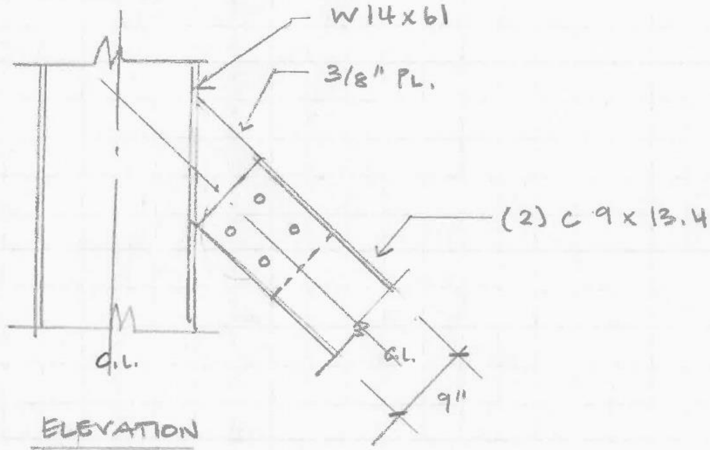
$\boxed{R_{uv} = 9.0^K < R_n = 386.78 \text{ KIPS}}$

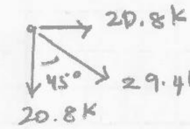
D/C = 0.023

A.2.4 Main Brace and Connections

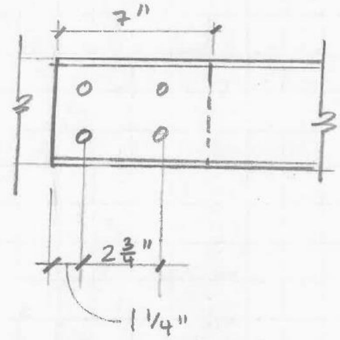


MAIN BRACE





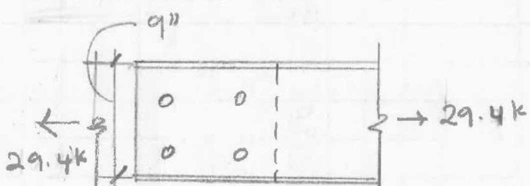
ELEVATION



NOTE:
DETAIL DRAWN FLAT

DETAIL

PLATE YIELDING: PER AISC EQN. D2-1



FORCE FROM REACTION FRAME COLUMN CALCS.

$P_n = F_y A_g$
$= 36\text{ksi}\,(3/8" * 9")$
$= 121.5^k$

$\boxed{P_n = 121.5^k > P_u = 29.4^k}$ OK

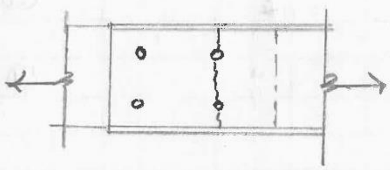
PLATE RUPTURE: PER AISC EQN. D2-2

$P_n = F_u A_e$

$= 62\text{ksi} * A_e$

$A_e = A_n U$ BOLTS NOT STAGGERED

$A_n = A_g - \Sigma A_{BOLTS} + \cancel{\frac{s^2}{4g}(t)} \rightarrow 0$



$A_n = 9''(3/8'') - 2(3/8'')(7/8'' + 1/16'') = 2.67\text{in}^2$

$A_e = 2.67\text{in}^2(1.0)$

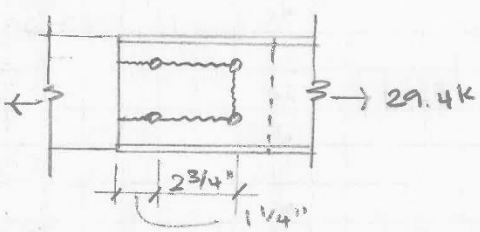
$P_n = 62\text{ksi} * 2.67\text{in}^2$

$\boxed{P_n = 165.5^k > P_u = 29.4^k}$ ok

BLOCK SHEAR:

$R_n = \overset{①}{0.6 F_u A_{nv} + U_{bs} F_u A_{nt}}$

$\leq \underset{②}{0.6 F_y A_{gv} + U_{bs} F_u A_{nt}}$



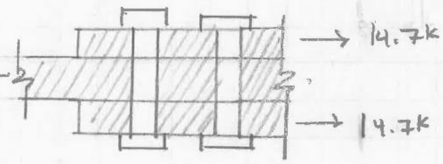
① $0.6(62\text{ksi})(4'' - 1.5\text{ BOLTS}(7/8'' + 1/16''))\,3/8''$
$+ 1.0(62\text{ksi})(4'' - 1\text{ BOLT}(7/8'' + 1/16''))\,3/8''$
$= 107.4^k$

② $0.6(36\text{ksi})(4'' * 3/8'') + 1.0(62\text{ksi}(4'' - 1\text{BOLT}(7/8'' + 1/16''))\,3/8''$
$= 103.6^k$ GOVERNS

$\boxed{R_n = 103.6^k > R_u = 29.4^k}$ ok

BOLT SHEAR:

PER AISC 7TH EDITION

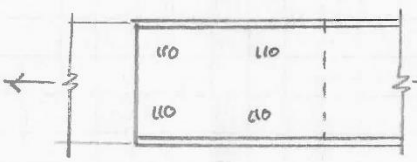


$F_{nv} = 15\text{ ksi}$

$$F_{uv} = \frac{29.4^k / 4\text{ BOLTS}}{\frac{\pi (7/8)^2}{4}} = 12.2\text{ ksi}$$

$\boxed{F_{nv} = 37.5\text{ ksi} > F_{uv} = 12.2\text{ ksi}}$ OK

BOLT BEARING:



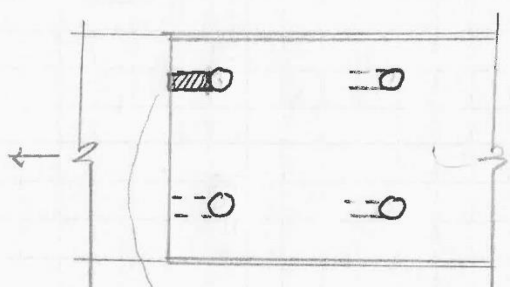
$\frac{29.4^k}{2\text{ CHAN.}} = 14.7^k$

$$F_B = \frac{P}{t\, d_{HOLE}} = \frac{14.7^k / 4\text{ BOLTS}}{(3/8")(7/8" + 1/16")} = 10.5\text{ ksi}$$

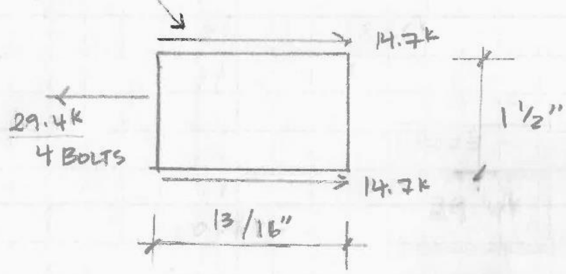
$F_{ALL} = 36\text{ ksi}$

$\boxed{F_{ALL} = 36\text{ ksi} > F_b = 10.5\text{ ksi}}$ OK

BOLT TEAROUT:



$\frac{29.4^k}{2\text{ CHAN.}} = 14.7^k$



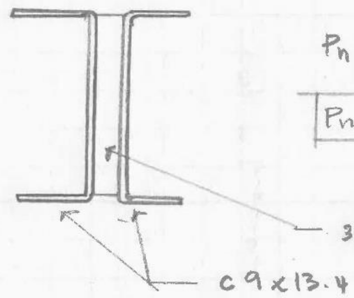
APPROXIMATED TEAROUT

$V_{T.O.} = 14.7^k / 4\text{ BOLTS} = 3.68^k$

$V_{ALL} = (13/16")(3/8"\text{ PL}) * 2\text{ SIDES} * 36\text{ ksi} = 11.0^k$

$\boxed{V_{ALL} = 11.0^k > V_{T.O.} = 3.68^k}$ OK

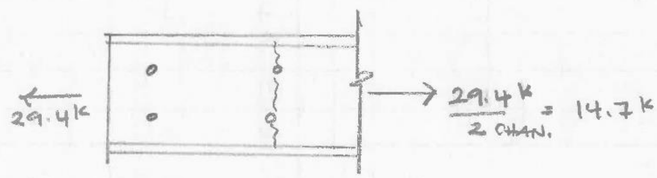
MEMBER YIELDING PER AISC EQN D2-1



$P_n = F_y A_g = 36\text{ksi} * (3.94\text{in}^2) * 2$ CHANNELS

$\boxed{P_n = 283.7^k > P_u = 14.7^k}$ OK

MEMBER RUPTURE PER AISC EQN D2-2



$P_n = F_u A_e$

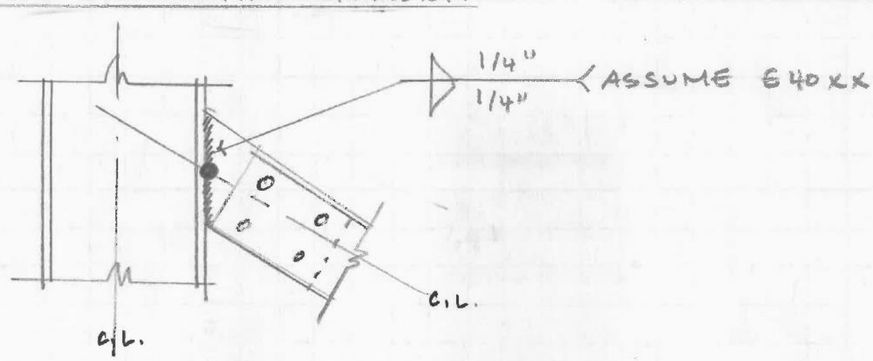
$= 62\,\text{ksi}\,(3.94\,\text{in}^2 - 2 \text{ BOLTS } (0.233''\text{ THK}) * (7\,1/8''\text{ DEEP}))$

$\boxed{P_n = 38.4^k > P_u = 14.7^k}$ OK

MEMBER COMPRESSION

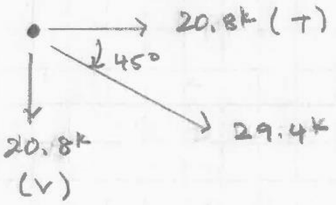
NOT CONCERNED ABOUT BUCKLING. UNBRACED LENGTH, $l_b < 5'$. THEREFORE, COMPRESSION STRENGTH ABOUT EQUAL TO YIELDING STRENGTH.

WELD IN SHEAR + TENSION PER AISC EQN J2-4



WELD LENGTH = 12 1/2"

FBD:



$$R_n = 0.60\,F_{exx}\,(\sqrt{2}/2)\,(D/16)\,l$$
$$= 0.60\,(40\text{ ksi})\,(\sqrt{2}/2)\,(4/16)\,(12\tfrac{1}{2}'') * 2\text{ SIDES}$$

$R_n = 106.1^k$

SINCE LOADED @ 45°

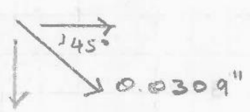
$$R_n = 106.1^k * (1 + 0.5\sin^{1.5}(45°)) = 137.7^k$$

$\boxed{R_n = 137.7^k > R_u = 29.4^k}$ OK

AXIAL DEFLECTION

$P = 29.4^k$

$$\Delta_{AXIAL} = \frac{PL}{EA} = \frac{29.4^k\,(10' * 12''/1')}{29000\text{ ksi} * 3.94\text{ in}^2} = 0.0309''$$

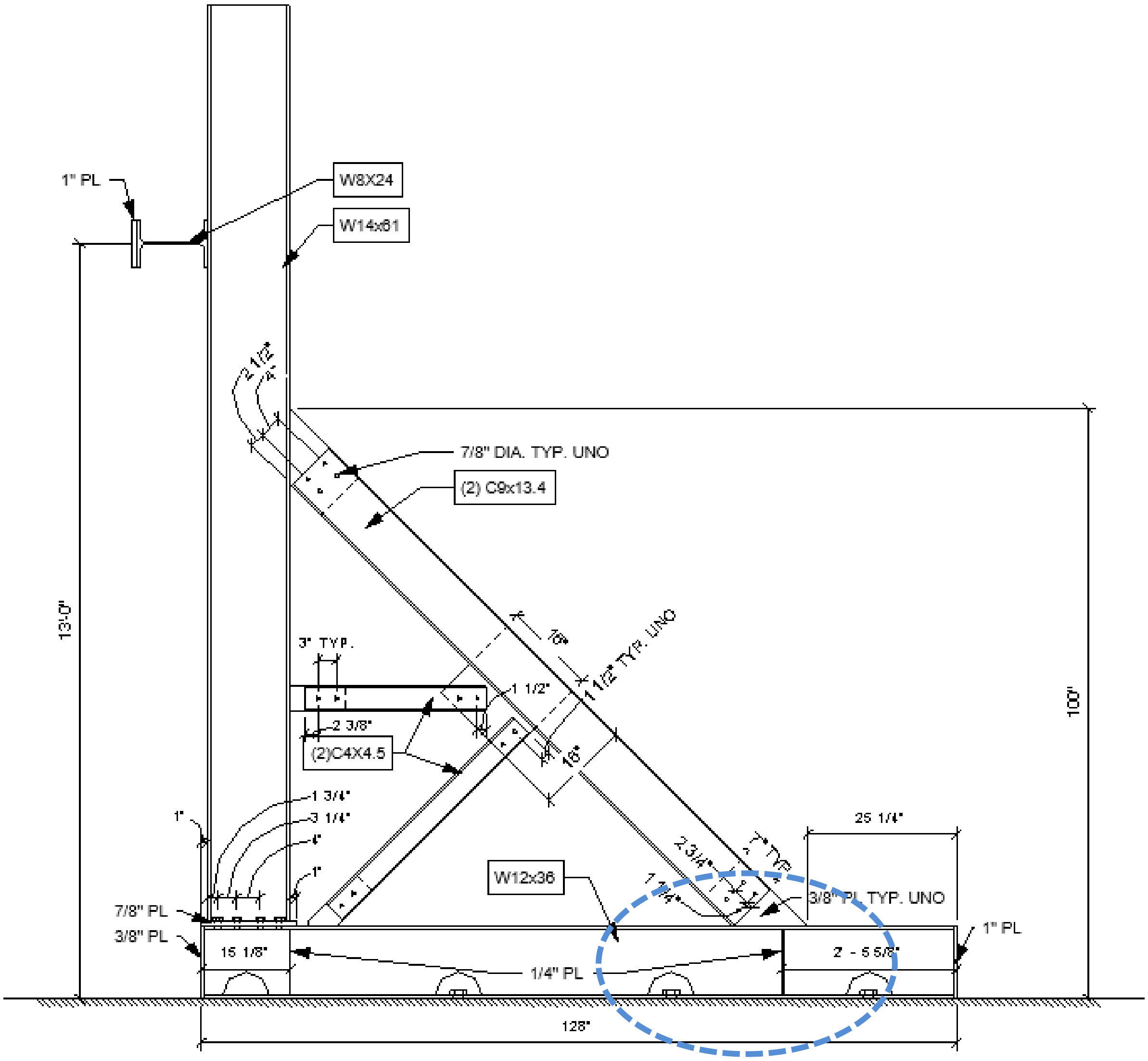


$$\sin 45 = \frac{x}{0.0309''}$$

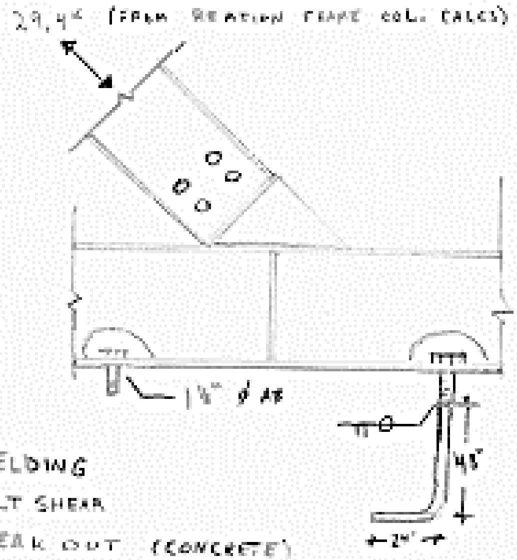
$x = 0.0218''$

$\boxed{\Delta_{BRACE} = 0.0218'' \text{ IN DIR. OF FORCE APPLIED}}$

A.2.5 Reaction Frame to Strong Floor Connections



FLOOR BEAM TO STRONG FLOOR CONNECTION



FAILURE MODES

ANCHOR BOLT: 1) YIELDING

2) BOLT SHEAR

3) BREAK OUT (CONCRETE)

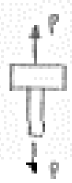
4) REBAR YIELDING

$F_u = 29.4^K \sin(45°) = 20.79^K$

STIFFNER: 5) PLATE BUCKLING

6) PLATE YIELDING

1) ANCHOR BOLT (YIELDING) A325 - 1 1/2" ϕ BOLT



$F_{nt} = \frac{40}{.6}$ KSI (4-3; AISC 360-70) = $F_{nt} = 66.7$ KSI

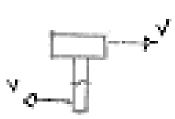
TENSION CAPACITY (PER BOLT) = 117.8 KIPS = P_n (TABLE ON Pg. 4-3; AISC 360-7)

* ASSUME BOLTS SPLIT DEMAND LOAD FROM DIAGONAL.

DEMAND: $20.79^K / 2$ bolts $= 10.4^K = P_u$

$P_u = 10.4$ KIPS $< P_n = 117.8$ KIPS ✓ D/C = .088

2) ANCHOR BOLT (BOLT SHEAR) $F_v = \frac{15}{.4}$ KSI = 37.5 KSI



(FROM TABLE ON Pg. 4-10 IN AISC 360-70)

BOLT SHEAR CAPACITY (PER BOLT) = 66.3 KIPS = V_n

ASSUMPTION: • SHEAR ON FLOOR BEAM SPREADED EQUALLY TO ANCHOR BOLTS

DEMAND: HALF ACTUATOR LOAD (11.8^K) SPREAD AMONGST 4 ANCHOR BOLTS

$V_u = 11.8^K / 4 = 2.95^K$

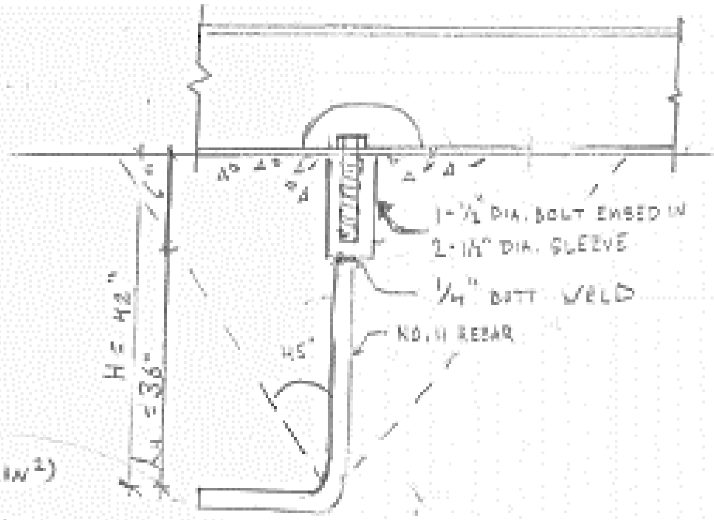
$V_u = 2.95^K < V_n = 66.3$ KIPS ✓

D/C = 0.044

3) PULL OUT (CONCRETE)

ASSUMPTIONS:

- CONCRETE WILL CRACK @ 45° ANGLE
- $f'_c = 3{,}000$ PSI
- $f_r = 7.5\lambda\sqrt{f'_c}$
- MODIFICATION FACTOR (λ) = 1
  - NORMAL WEIGHT CONCRETE



PULL OUT (KIPS) = f_r (KSI) × SA_{CONE} (IN²)

where: SA_{CONE} = SURFACE AREA OF CONE

$$SA_{CONE} = \pi \times 1.4142 \times 54'' = 239.9 \text{ in}^2$$

$$f_r = \frac{7.5\lambda\sqrt{f'_c}}{1000\ \frac{lb}{kip}} = .411 \text{ KSI}$$

PULLOUT = .411 KSI × 239.9 in² = $\underline{98.60 \text{ KIPS}} = P_{nc}$

DEMAND: $P_{uc} = P_n$ (FROM ANCHOR BOLT YIELDING) = 10.4^K

$P_{uc} = 10.4^K < P_{nc} = 98.60^K$ ✓

D/C = .105

4) REBAR YIELDING

ASSUMPTIONS: • ¼" BUTT WELD WAS DESIGNED TO WITHSTAND REBAR YIELDING

• GRADE 40 # 11 BAR $f_y = 40$ KSI

P_{nR} 40 KSI × 1.56 in² (ACI 318-14)

$\underline{P_{nR} = 62.4 \text{ KIPS}}$

DEMAND: SAME AS P_{uc} IN CALCULATION ABOVE: $P_{uc} = 10.4^K = P_{uR}$

$P_{uR} = 10.4^K < P_{nR} = 62.4^K$

D/C = .167

DEVELOPMENT LENGTH OF HOOK (TENSION)

ACI 318-14: 25.4.3.1

$$l_{dh} \geq \max \begin{cases} \left(\frac{f_y \psi_e \psi_c \psi_r}{50\lambda\sqrt{f'_c}}\right) d_b = 14.42'' \\ 8 d_b = 8(1.41'') = 11.28'' \\ 6'' \end{cases}$$

$f_y = 40{,}000$ PSI

$f'_c = 3{,}000$ PSI

$\psi_e = 1.0$ (TABLE 25.4.3.2)

$\psi_c = 0.7$ (TABLE 25.4.3.2)

$\psi_r = 1.0$ (TABLE 25.4.3.2)

$\lambda = 1.0$ (TABLE 25.4.3.2)

$d_b = 1.41''$

$l_{dh} = 14.42''$

$\underline{l_{dh} = 14.24'' \leq l_H = 36''}$ ✓

FLOOR BEAM STIFFNER (FLOOR BEAM IS W12 x 36)

DETERMINE IF FLOOR BEAM IS COMPACT. $(\lambda \le \lambda_p)$ (TABLE B4.1 AISC 360-10)

λ = WIDTH THICKESS RATIO

WEB: $\lambda = \frac{T}{t_w}$

$\lambda_p = 3.76\sqrt{E/F_y}$

$\lambda = \frac{10.125\,in}{(5/16)\,in} = 32.4$

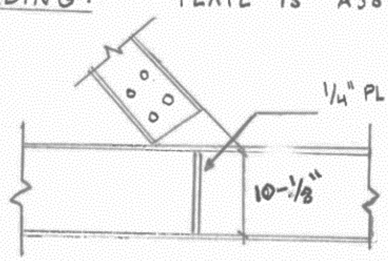
$\lambda_p = 3.76\sqrt{29{,}000\,KSI/36\,KSI} = 106.71$

$\lambda = 32.4 \le \lambda_p = 106.71$ ✓

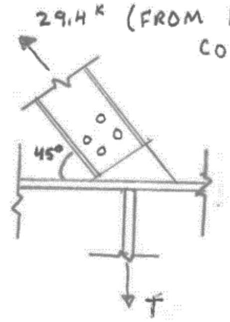
∴ NO LOCAL BUCKLING IN WEB

STIFFNER BUCKLING NOT CONSIDERED, W12 x 36 IS COMPACT IN THE WEB, THEREFORE LOCAL WEB BUCKLING IS NOT LIKELY TO OCCUR.

STIFFENER YIELDING: PLATE IS A36 (F_y = 36 KSI)



29.4 K (FROM REACTION FRAME COLUMN CALCS)



f_y = 36 KSI

$f_u = T/(10.125" \times 1/4")$

$T = 29.4^K (\sin 45°) = 20.79^K$

$f_u = 20.79^K/(2.53) = 8.22$ KSI

$\boxed{f_u = 8.22\,KSI \le f_y = 36\,KSI}$ ✓

D/C = .228

SHEAR FRICTION ON REACTION FRAME

ASSUMPTIONS:
- CLAMPING FORCE OF BOLTS CONNECTING FRAME TO STRONG FLOOR IS 40% OF TENSION CAPACITY
- FRICTION FORCE ADDED WITH CLAMPING FORCE GIVES SHEAR CAPACITY BETWEEN STRONG FLOOR AND REACTION FRAME.
- MEMBER SIZES WERE FOUND IN AISC 360-7.
- IGNORED BOLT + PLATE WEIGHTS TO BE CONSERVATIVE

WEIGHT OF REACTION FRAME: (ONE SIDE)

MEMBER	MEMBER SIZE	CALCULATION	WEIGHT
HORIZONTAL BEAM	W8 x 24	24PLF x 5'/(2 FRAMES)	60 #
FRAME COLUMN	W14 x 61	61 PLF x 14'	854 #
FLOOR BEAM	W12 x 36	36 PLF x 10.66'	384 #
DIAGONAL BRACES (LONGER)	(2) C9 x 15	15 PLF x 2 x 9.9'	297 #
DIAGONAL BRACE (SHORTER)	(2) C4 x 5.4	5.4 PLF x 2 x 4'	43 #
DIAGONAL BRACE (MID LENGTH)	(2) C4 x 5.4	5.4 PLF x 2 x 6'	65 #
		Σ =	1,703 #

(W_F) TOTAL WEIGHT OF FRAME: 1,703# x 2 SIDES

$W_F = 3,406$ #

FRICTION FORCE (F_{FR})

ASSUMPTION: • COEFFICENT OF FRICTION (μ) ASSUMED TO BE 0.30 AVERAGED BETWEEN STEEL/CAST CONCRETE (0.1) + STEEL/TROWELLED CONCRETE (0.5). (PALLET 2002)

$$F_{FR} = \mu W_F$$

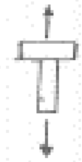
$$= 0.3\,(3,406\ \#) = 1,022\ \#$$

F_{FR} = 1.02 KIPS

CLAMPING FORCE (F_c)

ASSUMPTIONS: • CLAMPING FORCE BETWEEN STRONG FLOOR + REACTION FRAME DUE TO ANCHOR BOLTS IS 25 % OF YIELDING STRENGTH OF ANCHOR BOLTS.

* ANCHOR BOLT IS A325 WITH $F_{nt} = 66.7$ KSI



F_c = 0.25 × BOLT TENSILE CAPACITY (F_{nt}) × AREA OF BOLT

$= 0.25 \times 66.7\ \text{KSI} \times \frac{\pi(1.5'')^2}{4}$

$F_c = 29.47$ KIPS / BOLT

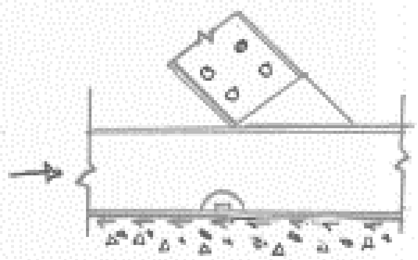
SHEAR FRICTION OF FRAME + CONCRETE

V_{FR} = (CLAMPING FORCE × μ × 8 BOLTS) + F_{FR}

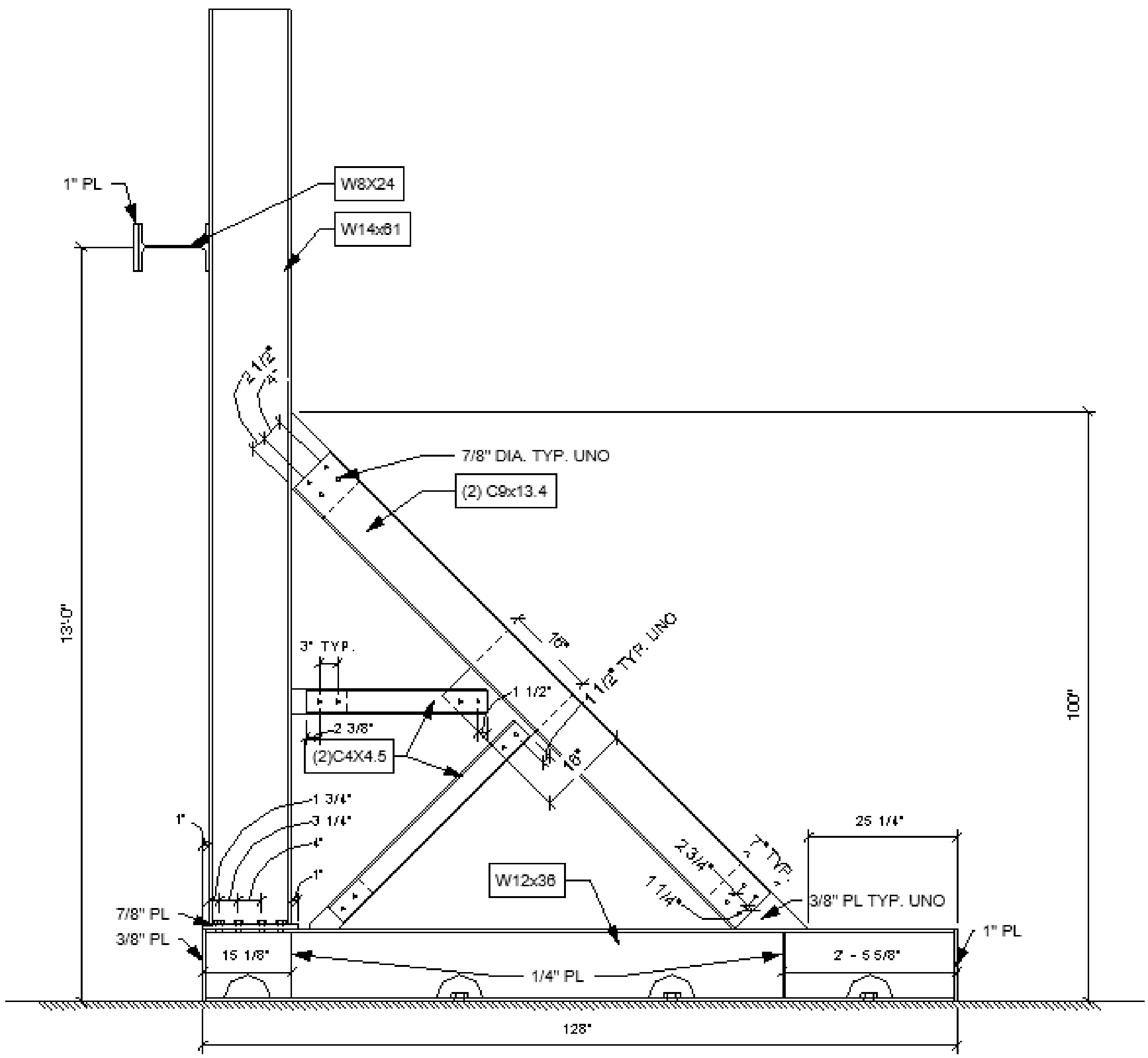
$= (29.47^k \times 0.3 \times 8) + 1.02$ KIPS

$V_{FR} = 71.75$ KIPS

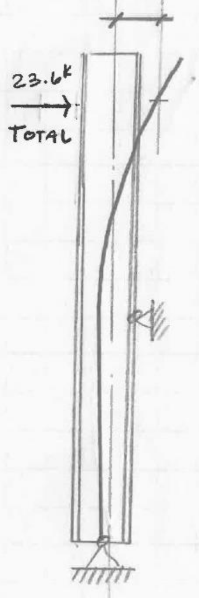
* 71.75 KIPS ARE NEEDED TO ENGAGE THE ANCHOR BOLTS IN SHEAR *



A.2.6 Overall System Deflection



DEFLECTION OF OVERALL SYSTEM



(Pg 53) (Pg 40) (Pg 63)

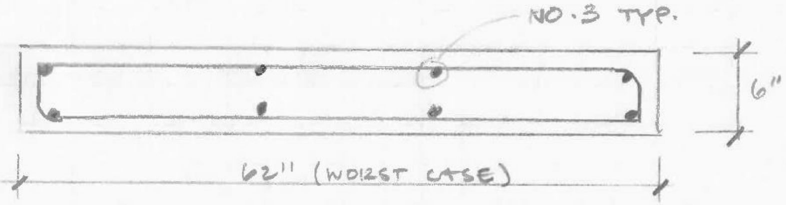
$\Delta_{CA} = 0.308'' + \Delta_{BEAM} = 0.00959'' + \Delta_{BRACE} = 0.0218''$

$\Delta_{TOTAL} = 0.308'' + 0.00959'' + 0.0218''$

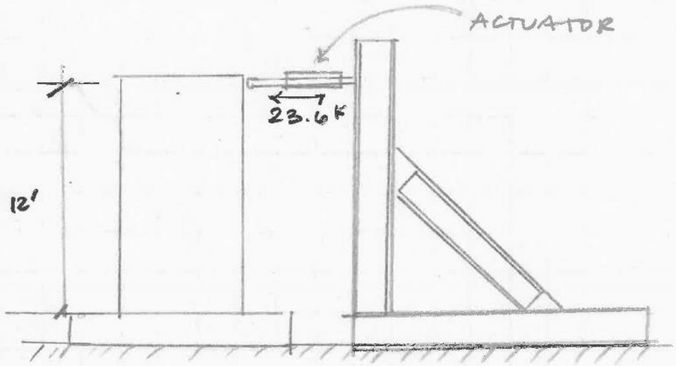
$\boxed{\Delta_{TOTAL} = 0.339''}$

A.2.7 Proposed Concrete Wall Specimen

STIFFNESS OF PROPOSED WALL SPECIMEN



TYPICAL CONCRETE WALL SECTION



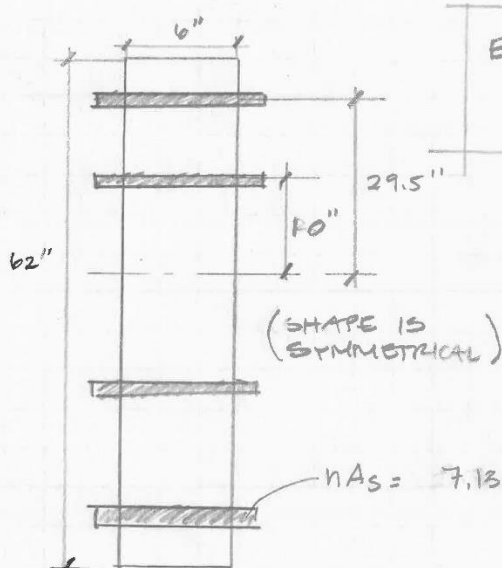
WALL ELEVATION

USING CRACKED STIFFNESS APPROACH PER ACI 318-14
TABLE 6.6.3.1.1(a) : $I_{cr} = 0.35 I_g$

FIRST TRANSFORM CROSS SECTION. CONSIDER ONLY FLEXURAL REINF.

$n = E_s / E_c = 29000 \text{ ksi} / 4066 \text{ ksi} = 7.13$

$E_c = W_c^{1.5}\, 33 \sqrt{f_c'}$
$= (150\text{ PCF})^{1.5}\, 33 \sqrt{3000 * 1.5}$
$= 4066840 \text{ psi}$



$nA_s = 7.13 (0.11 \text{ in}^2 * 2) = 1.569 \text{ in}^2$

$$I_g = \frac{bh^3}{12} + Ad^2$$

$$= \frac{6''(62'')^3}{12} + \left(1.569\,in^2(10'')^2 + 1.569\,in^2(29.5'')^2\right) * 2$$

$$= 122209\ in^4$$

$$I_{cr} = 0.35\,I_g$$

$$= 0.35(122209\,in^4)$$

$$= 42773\,in^4$$

$$K_{WALL} = K_{CANTILEVER} = \frac{3EI_{cr}}{L^3} = \frac{3(4066\,ksi)(42773\,in^4)}{(12' \times 12''/1')^3}$$

$$\boxed{\begin{array}{l} K_{WALL} = 174.7\ k/in \\ \Delta_{WALL} = 23.6k/174.7\,k/in = 0.135'' \end{array}}$$

Note: This is based on the cracked limit state of the proposed concrete wall test specimens.

A.2.8 Proposed Upgrade

PROPOSED STIFFENING 1:

ADD ADD'L BRACE

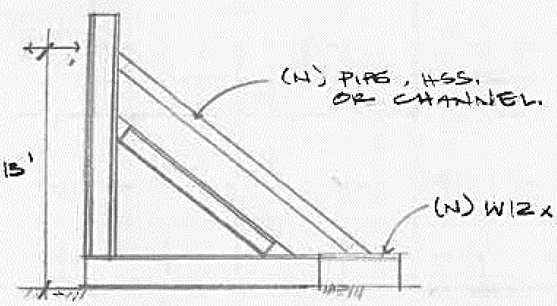
INITIAL SPRING STIFFNESS OF WALL: $\frac{3EI_g}{L^3}$

(ONLY CONSIDERING FLEX. STIFFNESS B/C GOVERNED BY FLEX.)

$$k_{wall,i} = \frac{3(4066\,ksi)(122209\,in^4)}{(12' \times 12''/1')^3} = 499.2\ k/in$$

$K_{REACTION\ FRAME}$ = 38.3 k/in (ASSUME ONLY COLUMN TO BE CONSERVATIVE)

R.F. SHOULD BE 10x STIFFER THAN WALL.



ELEVATION OF PROPOSED

NEGLECT K_{COL} B/C SMALL

DESIGN PIPE S.T. K = 4990 k/in (10 x K_{WALL})

$$k_{BRACE} = \frac{EA}{L}\cos^2\theta$$

$$4990\,k/in = \frac{29000\,ksi\,(A_{REQ'D})}{((13 \times 12)^2 + (13 \times 12)^2)^{1/2}} \cos^2(45°)$$

$A_{REQ'D}$ = 75.9 in² / 2 FRAMES = 38.0 in²

THEREFORE TRY (2) C15x50 w/ A = 40.7 in²	FOR 10x STIFF.
OR HSS 12x8x5/8 w/ A = 21.0 in²	FOR 5x STIFF
OR HSS 14x0.625 w/ A = 24.1 in²	FOR 5x STIFF.

A.3 Existing Condition Photographs

Figure 12: Front Right View of Reaction Frames

Figure 13 : Back Left View of Reaction Frames

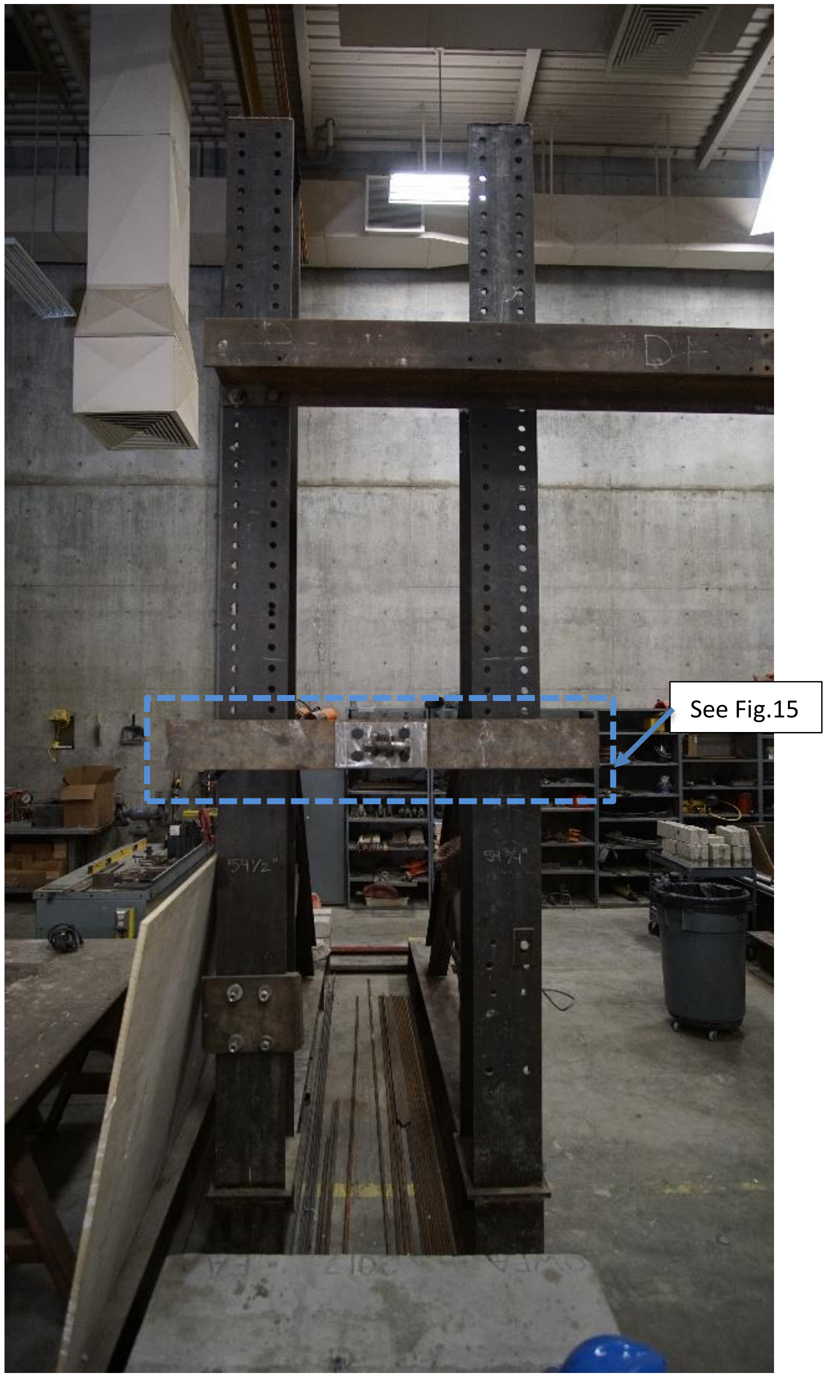


Figure 14 : Reaction Frame Front Elevation

Figure 15 : Reaction Frame Horizontal Beam

Figure 16 : Sandwich Plate on Horizontal Beam

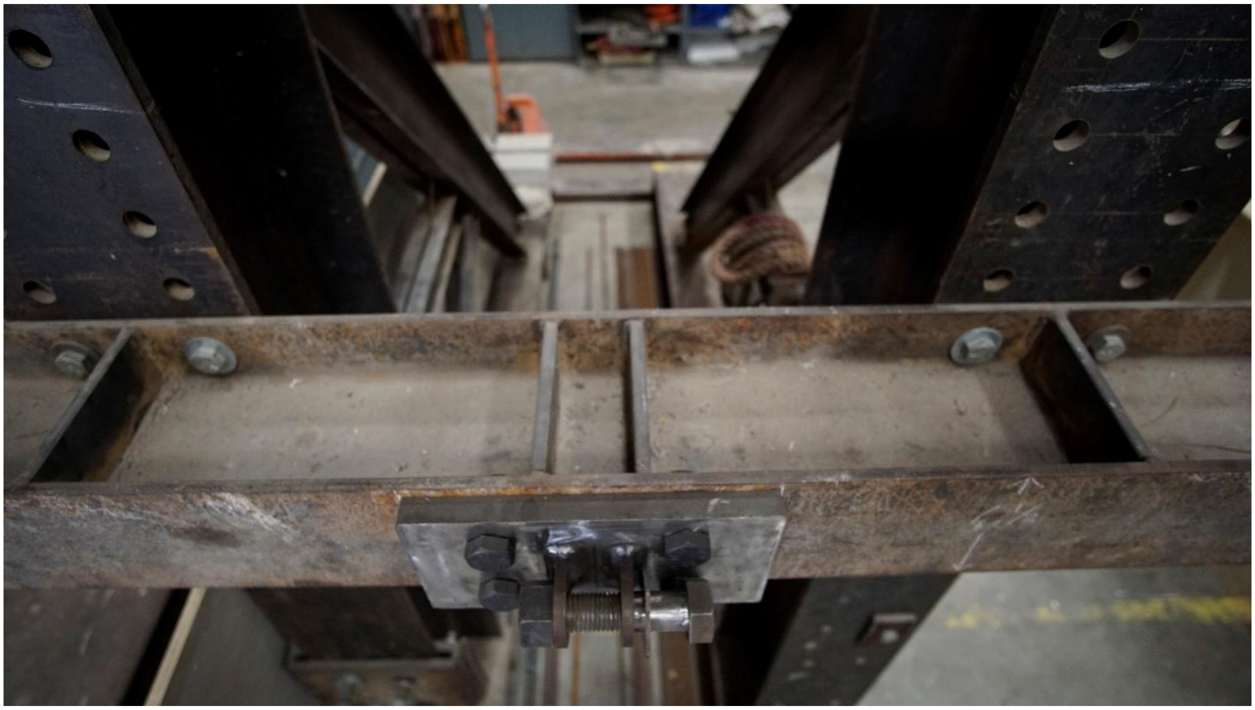


Figure 17 : Top View of Horizontal Beam

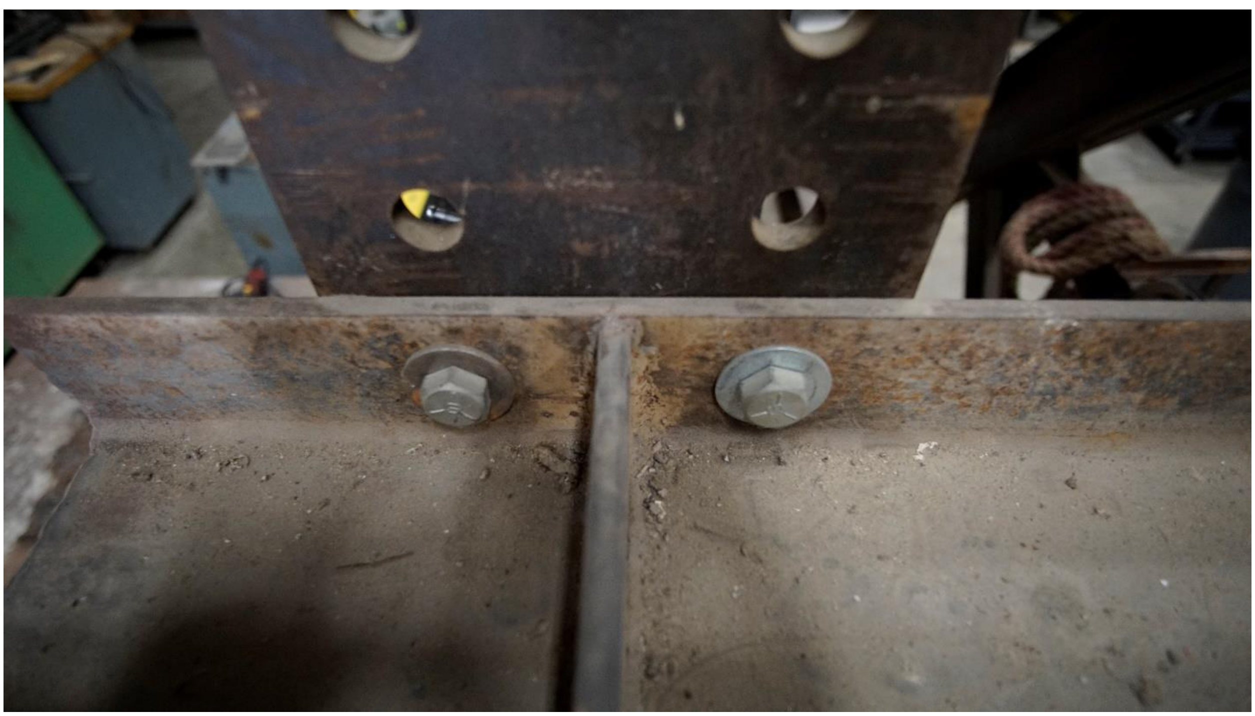


Figure 18 : Stiffener Plate in Horizontal Beam

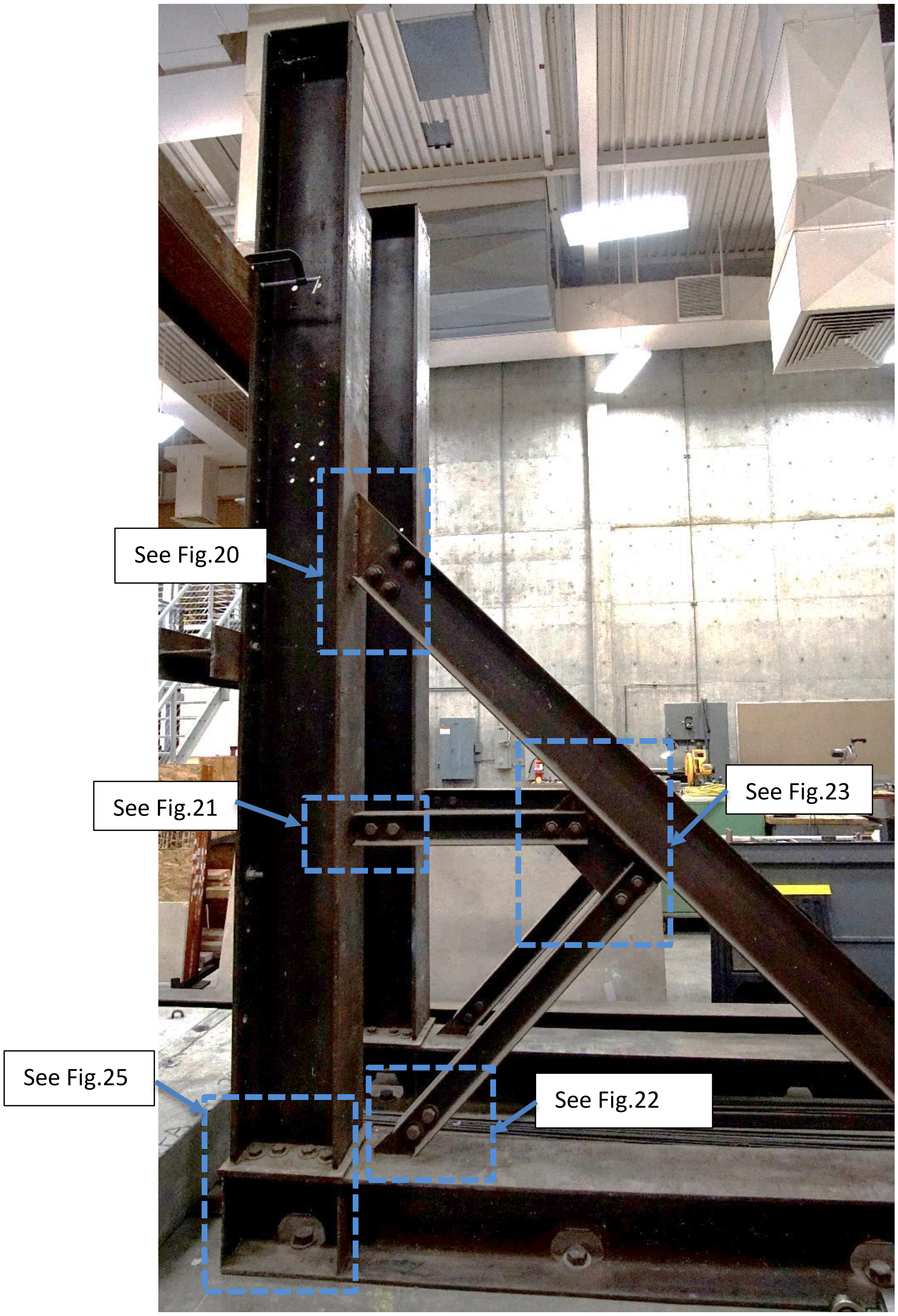


Figure 19 : Elevation of Steel Reaction Frames

Figure 20 : Brace Connected to Column with Gusset Plate

Figure 21 : Intermediate Horizontal Member for Main Brace

Figure 22 : Intermediate Diagonal Member for Main Brace

Figure 23 : Main Brace Gusset Plates Bolted to Intermediate Members

Figure 24 : Brace to Floor Beam Connection

Figure 25 : Reaction Frame Column to Floor Beam Connection

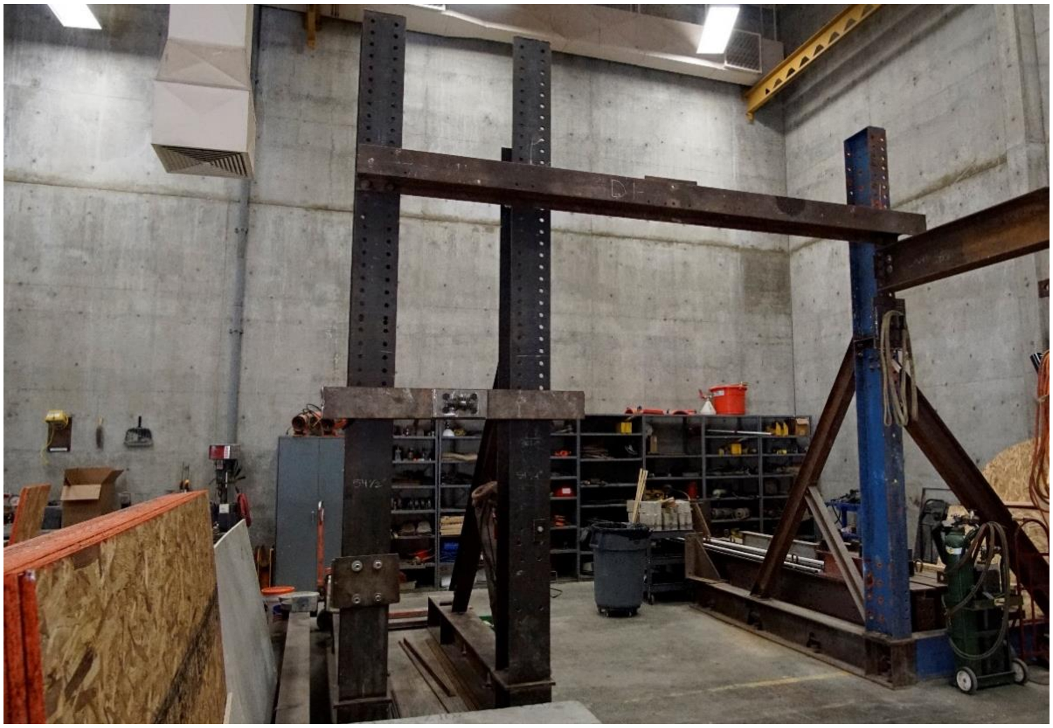


Figure 26 : Cal Poly San Luis Obispo High Bay Laboratory