STEEL REACTION FRAME

Kyle Coleman, CEE Laboratory Manager
Andrew Myers, Assistant Professor
Jerome F. Hajjar, Professor and Chair, Director, STReSS Laboratory

Department of Civil and Environmental Engineering, Northeastern University, Boston, Massachusetts

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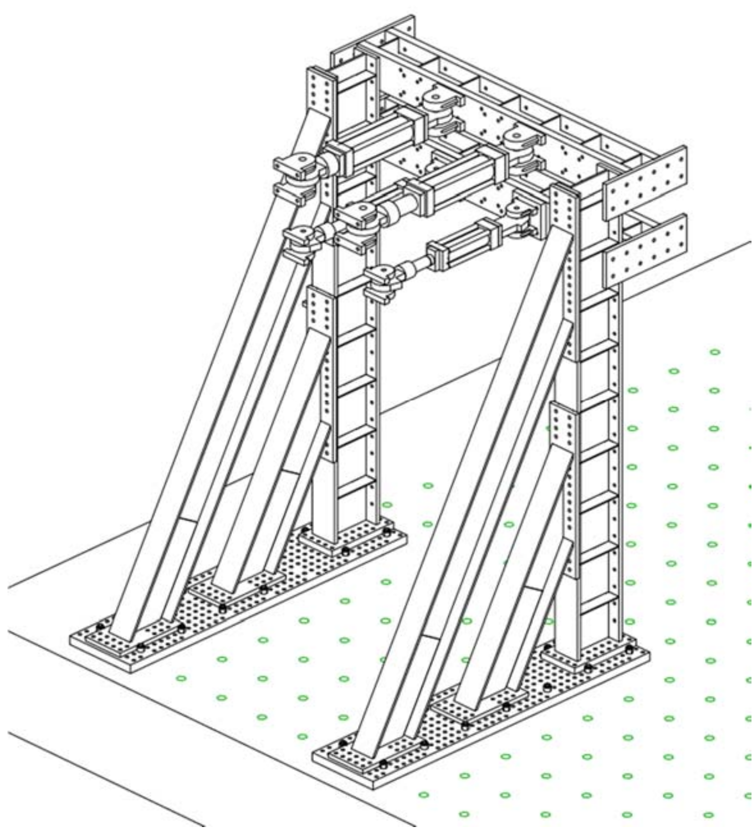


OVERVIEW

Approach

The first significant steel frame required for the facility is a steel reaction frame that will be a main feature of the STReSS laboratory. It will be a primary interface between the custom-designed concrete strong floor and our laboratory experiments. The function of the reaction frame is to transfer the forces into the floor that are applied to test specimens by hydraulic actuators. The reaction frame or portions of the frame will be utilized for most major experiments in the laboratory. Because the geometries of future experiments are unknown, the reaction frame has been designed to be reconfigurable. The frame can apply forces either horizontally at varying heights or vertically from above. The reaction frame has been designed to withstand cyclic loading involving the full capacities of the STReSS Laboratory hydraulic actuators, which total up to almost 1 million pounds of force. The design allows for several actuators to apply load simultaneously. Having two reaction frames will enable the application of force from different directions. The compact design and adaptability of the steel reaction frame to a wide range of experimental configurations is a fundamental asset for the future of the STReSS Laboratory.

The frame will be primarily designed for stiffness and fatigue for several worst-case loading scenarios. Finite Element Analysis was used to check stresses in the members for different loading cases. In addition, the AISC and RCSC specifications were used for checking strength of members and connections, including fatigue.

Design Priorities

- Strength
- Redundancy
- Stiffness
- Ease of Fabrication
- Ease of installation and easily movable
- Modular, flexible and reconfigurable
- Similar to other labs (not unconventional)
- Simple and symmetrical geometry
- Even dimensions
- Wrench clearance for bolts

Brace Angle

- 60 degrees is the maximum recommended angle for a brace frame of this type.
- We want to maximize the angle to save length and space on the strong floor.

Column and Brace Sizes

The members in the frame are designed for stiffness and fatigue. The maximum stress target for the frame is 20 ksi. All bending stress, or combined bending and axial stress, should be below this value.

Finite Element Model Dimensions

The following dimensions were used for the SAP2000 Finite Element model:

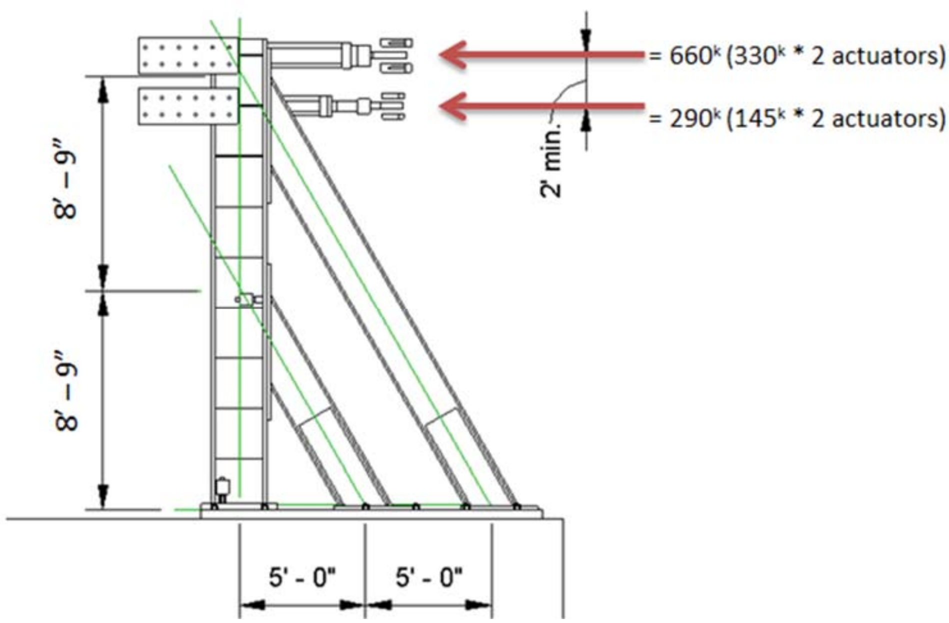


Figure 1

Steel Materials List

The following figure show the steel required for **one complete reactions frame**.

part	qty	section	length (in)	length (in)	width (in)	thickness (in)	weight/ft (#/ft)	total steel (#)	Steel Grade Min. (ksi)
Main Anchor Plate	2			165	37	4		13,871	36
Cross Beam - Short	2	w24x335	113.75				335	6,351	50
End Plates	4			39	17	2		1,506	36
stiffener plates	12			22.5	6	0.5		230	36
Brace - Long	2	w21x182	236.00				182	7,159	50
Base Plate	2			21	37	2		883	36
Connection Plate	2			87	13	2		1,285	36
Brace - short	2	w21x182	116.00				182	3,519	50
base plate	2			21	37	2		883	36
connection plate	2			89	13	2		1,314	36
column	2	w24x250	222.50				250	9,271	50
base plate	2			36	20	3		1,227	36
stiffener plates	16			22.5	6	0.5		307	36
vertical adapters	2	w12x87	30.00				87	435	36
stiffener plate	4			10.875	5	0.5		31	36
actuator adapter medium	4			21	13.5	4		1,288	36
actuator adapter small	2			21	13.5	4		644	36
							Grand Total	50,202	lbs

Figure 2 - Materials Required for One Complete Reaction Frame

LIMIT STATES

Several checks were done for both the sections involved, and for the loading configurations.

Section Checks

The section checks were done in accordance with the AISC Steel Manual. All sections were subjected to these checks.

Compressive Strength Check:

1. Local Buckling: FB, FLB, WLB (AISC Chapter E)
2. Compressive Strength for Flexural Buckling (AISC E3): $F_{MAX} < 20$ ksi

Flexural Strength Check:

1. Local Buckling: LTB, FLB, WLB (AISC Chapter F)
2. Lateral-Torsional Buckling (AISC F2): $M_n/S_x < 20$ ksi

Worst-Case Loading

This reaction frame has been designed so that the crossbeams can be located at many different heights along the column. This was done to add flexibility to the frame for future experiments, and it also produces many potential worst-case loading scenarios on the column and the braces. These loading cases have been systematically identified and checked for maximum stress. See the column section for loading cases.

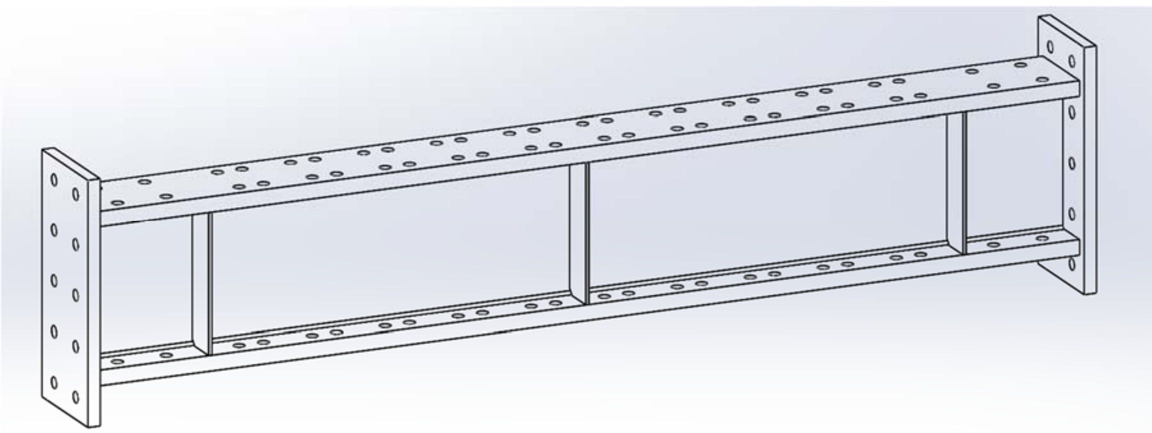
Fatigue

Fatigue is a limit state for many components of the connections as well as the controlling factor for the base metal stress range.

Stiffness

Deflection for the midpoint of the 12' span under worst-case loading conditions is 0.67". This is 0.67"/30" stroke = 2.20% of the total stroke of the large actuator.

CROSSBEAM



General

While the other components of the frame will typically remain in a set position, the crossbeam can be moved up or down the column and swapped out for different lengths as needed. The crossbeams can also be paired together to handle larger loads. Both 6' and 12' lengths are considered here. To be conservative, the cross beam will be designed as a pin-pin connected element.

Worst Case Loading

Out of the many different possibilities, it was decided that a reasonable loading worst-case for the crossbeams at 12' span is point loads of 330 kips at third-points along the length.

For the 6' span, the worst case loading is 330 kips at mid-span.

Considerations

Criteria for consideration of the crossbeam design include:

- Section weight
- Section depth
- Section stiffness
- Constructability

Design

12' span crossbeams

If a ***single beam*** is used for the above stated worst case:

M_{max} = +/- 15,840 kip*in

F_a = +/- 20 ksi

Required $S_x = M_{max}/F_a$ = 792 in^3

Allowable Sections	S_x
W14x500	838
W14x550	931
W24x335	864
W27x281	814
W33x241	831
W36x232	809

If a ***pair of crossbeams*** is used for the above stated worst case, then the single worst case can be considered as one large actuator in the center of the 12' span.

M_{max} = 330 kip * 12' * 12 / 4 = +/- 11,880 kip-in

F_a = +/- 20 ksi

Required $S_x = M_{max}/F_a$ = 594 in^3

Allowable Sections	S_x
W14x370	607
W18x311	624
W27x217	627
W33x201	686
W36x182	623

Approximately 25% of the steel weight can be saved if the paired approach is used. However, it is at the expense of approximately 25% of the bending stiffness as well as some flexibility of the setup (the pair uses all available beams at a specific height). Other drawbacks include a limit on actuator height when using the pair (approx. 16' rather than 18'), as well as adapter plates required to bridge between the pair.

It is recommended to use the larger ***single beam***, and include two beams to allow 4 actuators to be used. A W24x335 section was selected.

6' Span Crossbeams

Two crossbeams should be available for the alternative configuration of a 6' span. It is recommended that these beams have the same section as the 12' span for simplicity. However, the savings in steel would be significant if a lighter section were used.

We are considering the 6' span worst case to be a single large actuator at mid-span.

M_{max} = 330 kip * 6' * 12 / 4 = +/- 5,940 kip*in

F_a = +/- 20 ksi

Required $S_x = M_{max}/F_a = 297\ in^3$

Allowable Sections	S_x
W14x193	310
W18x158	310
W24x131	329
W27x114	299

Conclusion:

The recommended choice for the crossbeams:

(2) 14' W24x335 for 12' span
(2) 8' W24x131 for 6' span, or W24x335 for simplicity

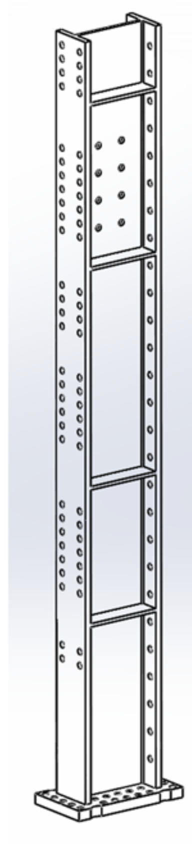
However, deeper beams, such as W33x241, are feasible if vital. A W14 section was originally considered, but in the *vertical loading configuration*, the section does not line up with our 8" hole pattern. A W24 section was required to fit the spacing.

The 12' available unbraced length is allowable by section AISC F2.2a:

L_b = 144 in < L_p = 188 in, therefore limit state of lateral torsional buckling does not apply.

See calculations for shear strength check.

COLUMN



General

The column is the vertical element of the frame, and it will stand at a height of 19’ from the floor. The column will be supported horizontally at two points along the height by the braces at 8.75’ and at 17.5’. The floor connection and the column connection of the braces are considered to be moment connections.

Worst-Case Loading

Since the crossbeams can be moved to any point along the height of the columns, the Worst-Case loading is not immediately apparent. Ten cases with forces in the same direction were identified to be checked. The cases are shown in the following figures with the results from SAP2000 in the corresponding tables. The analysis check procedure is as follows:

1. Check interaction ratio and note if using H1-1a or H1-1b (AISC)
2. Check if $P/A + M/S < 20$ ksi (target design stress)
3. Repeat for next case

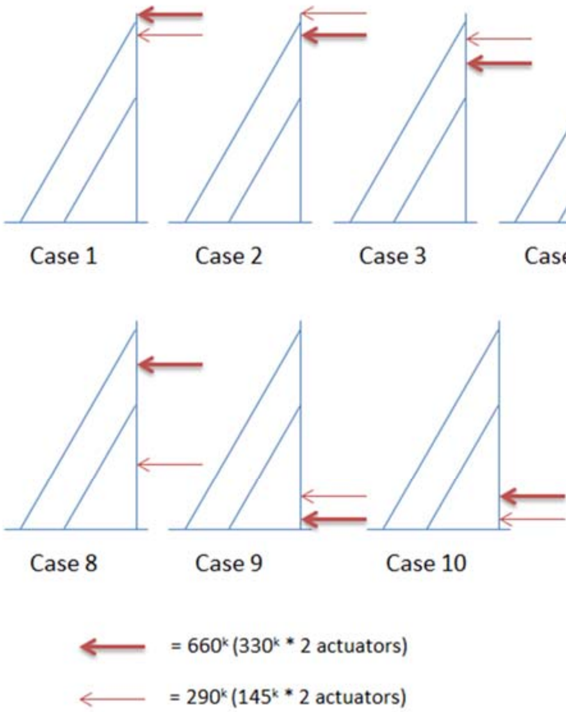


Figure 3 : Worst Case Loading Scenarios - Forces in the Same Direction

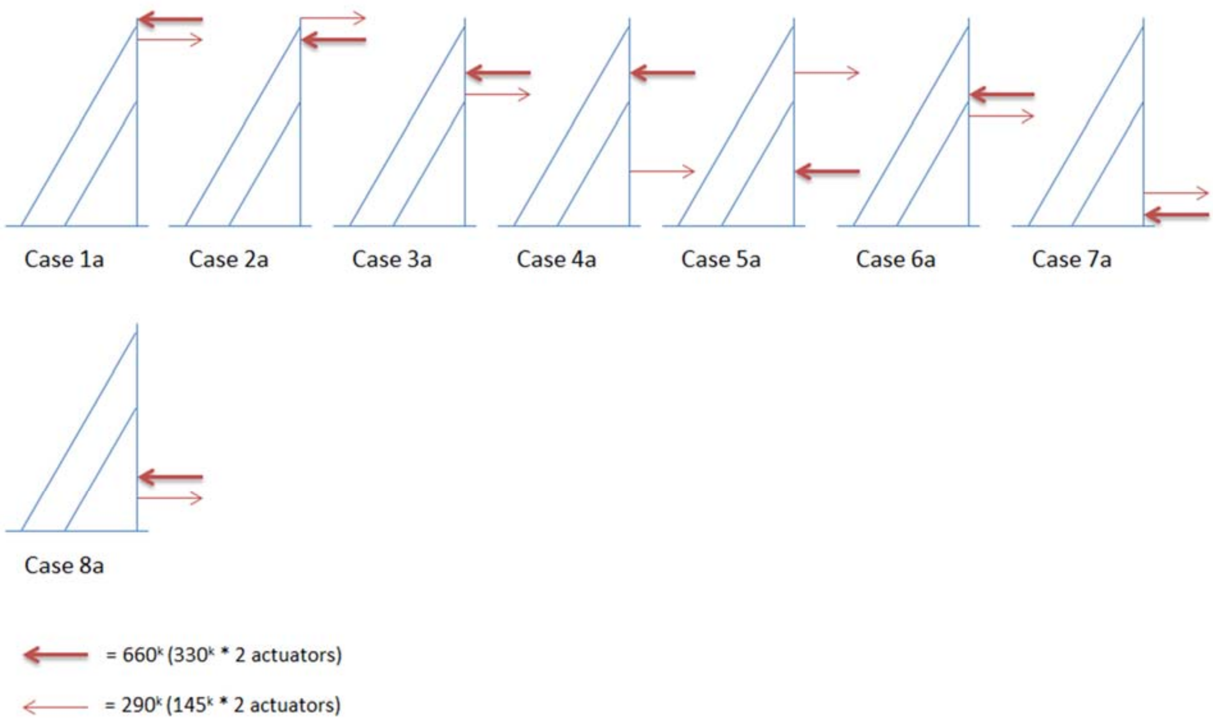


Figure 4: Worst Case Loading Scenarios - Forces in opposite Directions

Considerations

The column should have wide flanges since it will be connecting to the crossbeam as well as the braces. A wider flange, such as the W14, W24 or W27 will make connections simpler.

The column will need holes along one flange to accommodate the adjustable heights of the crossbeam. On the other flange, holes are needed for the brace connections, which will always be at the same point.

Design

Tension Case

In tension, we want the combined bending and axial stress to not exceed the target stress of 20 ksi. Using SAP2000, it was found that the highest stress of 20.88 ksi occurs in Case 1 for a W24x250 column (See Table 1). In addition, tensile strength for fracture over net area and tension and flexure interaction were checked for the worst cases.

Compression Case

In compression, we want the section to have strength to resist buckling in combined compression and flexure. By AISC equation H1-1b, the W24x279 column has a maximum stress ratio in case 1 of 0.561, and is therefore acceptable in compression (Table 1).

Case	interaction eqn	interaction value	P_u (kip)	M_u (k-in)	P/A + M/Sx	Large Force Location (inches from base)	Small Force Location (inches from base)
1	H1-1a	0.543	816	4311	17.78	216	192
2	H1-1a	0.547	790	5869	19.84	192	216
3	H1-1a	0.501	728	7084	20.88	156	192
4	H1-1a	0.560	724	5662	18.62	156	216
5	H1-1b	0.444	124	9260	16.03	48	24
6	H1-1b	0.502	199	10162	18.45	48	72
7	H1-1b	0.478	214	8996	16.85	48	156
8	H1-1a	0.537	514	7524	18.65	156	48
9	H1-1b	0.265	40	5720	9.40	12	36
10	H1-1b	0.358	73	7637	12.82	36	12

Table 1 : Analysis Results for Worst-Cases with Forces in Same Direction

Case	interaction eqn	interaction value	P_u (kip)	M_u (k-in)	P/A + M/Sx	Large Force Location (inches from base)	Small Force Location (inches from base)
1a	H1-1a	0.206	323	2942	13.08	216	192
2a	H1-1b	0.236	174	4377	13.74	192	216
3a	H1-1b	0.257	207	4678	15.08	156	132
4a	H1-1a	0.359	239	6795	20.75	156	48
5a	H1-1b	0.276	109	5581	15.42	48	156
6a	H1-1b	0.208	76	4240	11.59	117	93
7a	H1-1b	0.108	30	2261	5.98	12	36
8a	H1-1b	0.213	129	4094	12.22	64	40

Table 2: Analysis Results for Worst-Cases with Forces in Opposite Directions

Geometry

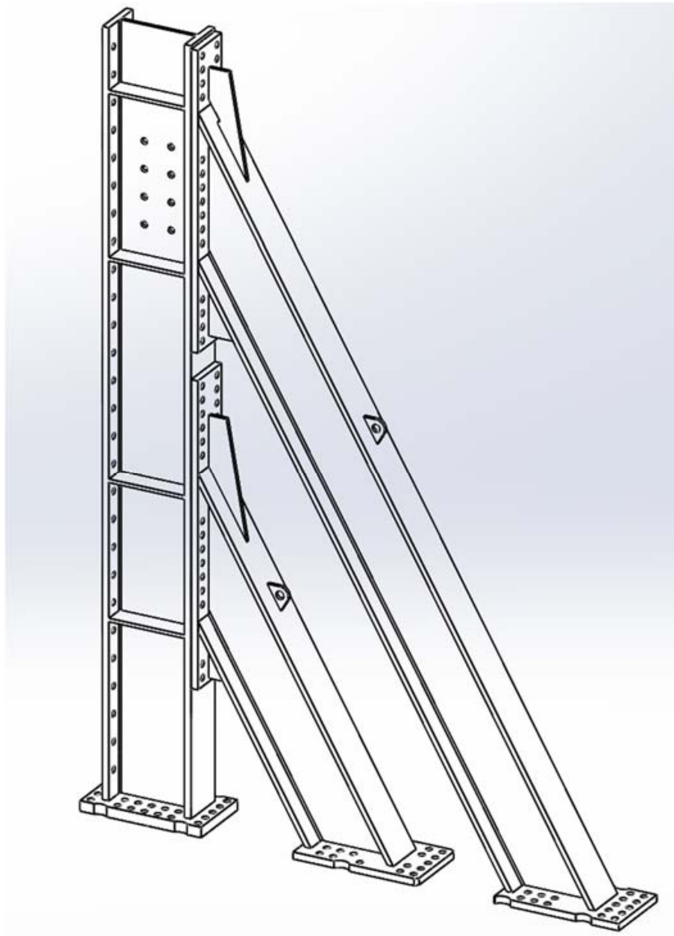
It is important that the flanges intersect the baseplate near the centerline of the floor anchors so that the maximum number of bolts can be located on the base plate. It is also important that the hole pattern can be rotated 90 degrees with the column when the vertical configuration of the reaction frame is needed. It was found that a W24x250 section satisfies these considerations.

Conclusion

The recommended choice for the columns is:

(2) 19' W24x250

BRACE



General

The brace is designed as a beam-column element. The longer brace is designed, and the smaller brace is sized to be the same for simplicity. Connections at both ends are considered as moment resisting connections.

Worst-Case

The worst-case, by inspection, is as follows:

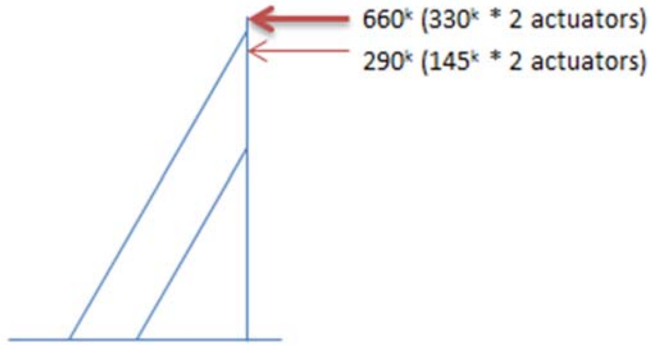


Figure 5

Design

Tension + Flexure:

Using Finite Element software (SAP2000), it is estimated that the tension loading case results in an axial force on the brace of 810 kips, along with a moment of 2111 kip*in.

Maximum Stress = $P/A + M/S_x$

810 kips / 53.6 in^2 + 2111 in-kip / 416.7 in^3 = 20 ksi <= 20 ksi → ok

Tension:

Tensile Strength for fracture over net area (AISC D2 and D3) is checked for the worst case of 810 kips. Since there are no holes in this section, $A_e = A_g$.

P_n / omega = $F_u A_e$ / omega = 65 ksi * 53.6 in^2 / 2.0 = 1,742 kips > 810 kips → ok

Compression + Flexure:

In the compression case, we want the section to have strength to resist buckling in combined compression and flexure. By AISC interaction equation H1-1a:

Demand/Capacity = $P_r/P_c + 8/9*(M_{rx}/M_{cx}+M_{ry}/M_{cy})$

0.968 = 0.811+0.132+0.025 →ok

Geometry:

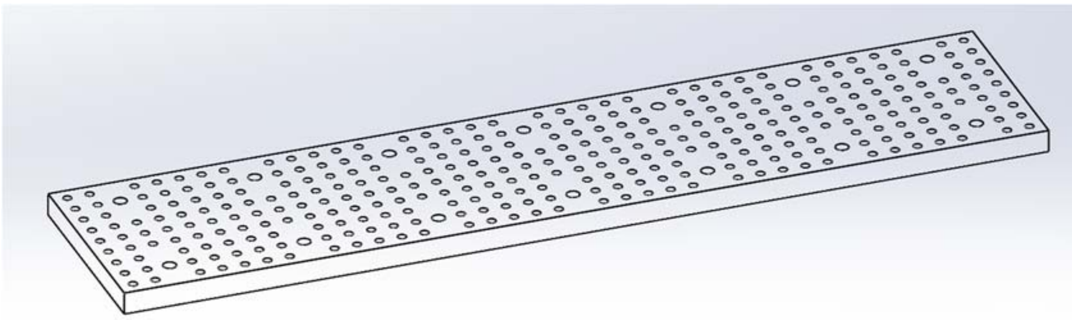
The depth of the section was driven by the location of the flanges at the intersection of the base connection plate. The intersection for a w21 allowed for the best connection locations.

Conclusion

The recommended choice for the brace is:

(2) 19’ W21x182
(2) 5’ W21x182

ANCHOR PLATE



General

The anchor plate is required to effectively distribute the forces in the reaction frame to the strong floor by way of the anchors. Local anchors are depended on for transferring tension forces, while shear forces are assumed to transfer to all anchors in the plate. It was found through analysis and discussion with other experienced laboratory managers that the simplest and most efficient way to transfer forces is with a solid plate. The plate will be 36” wide to provide 6” of edge distance for the anchors. The anchor holes will be unthreaded through-holes for 2” diameter rod, and all other holes will be threaded 1.5” diameter through-holes. Threaded anchor rod will be pre-tensioned using CY Series supernut fasteners.

Worst-Case Loading

The worst loading case is when the largest load occurs at the connection point of one brace along the column (Case 1). In that case the load is transferred almost totally to one brace, then to one floor connection.

Design

Number of anchors:

It was found that a minimum of 8 anchors are required for the braces (together), and 4 anchors are required for the column. The following analysis method was used to find this conclusion:

Number of anchors = (Tension load + shear load/friction coeff.)/anchor capacity

Number of anchors = (685 + 393/0.5)/200 kips = 7.36, say 8 anchors required for one **brace**

Number of anchors = (816 + 2/0.5)/200 kips = 4.1, say 4 anchors required for each **column**

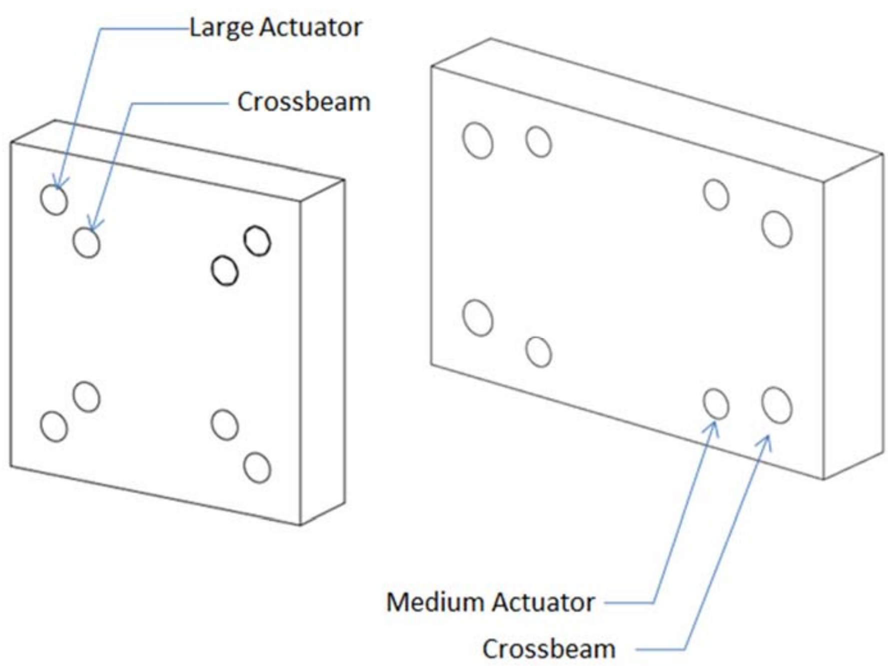
Since two anchors remain in between the braces and column, it was decided to make the plate continuous between them, gaining the benefit of the extra two anchors as well as sharing the strength of all anchors (and the additional strength of friction from compression).

Anchor Plate Thickness:

Approximating the plate as a continuous beam using Finite Element Software (SAP200), it was found that the anchor plate will see a maximum moment of 526 kip-in caused by the brace or column in tension. The required thickness of the plate is 1.85”. It is recommended that this thickness be increased to 3” to be conservative and to account for unpredicted behavior. The threaded connectors will also need sufficient embedment into the anchor plate. 3” is more than the required thread length of engagement by the 1984 Federal Standard for screw thread standards (Fed 1984).

The final plate dimensions will be 13.5’ x 3’ x 4”. Two plates are needed for one frame.

ACTUATOR ADAPTER



General

The actuators need to attach to the crossbeams. However, the bolt sizes and spacing vary between the three different types of actuators that we have at the STReSS Lab. Due to the overlap between the hole patterns, the different options cannot be drilled into the beam itself. Adapters are required for different actuators. The following hole patterns are considered:

Alternative Crossbeam Hole Pattern	8” x 16” pattern, 1.5” dia. Bolts
MTS 243.35T (existing smallest actuator)	7.25” square, 1.125” dia. bolts
Crossbeam Hole Pattern	8” square, 1.5” dia. Bolts
MTS 201.45 (Medium actuator)	9.5” square, 1.25” dia. bolts
MTS 201.70 (large actuator)	11.75” square, 1. 5” dia. bolts

The adapter is a 4” plate with threaded holes to accommodate different hole patterns of actuators.

CONNECTIONS

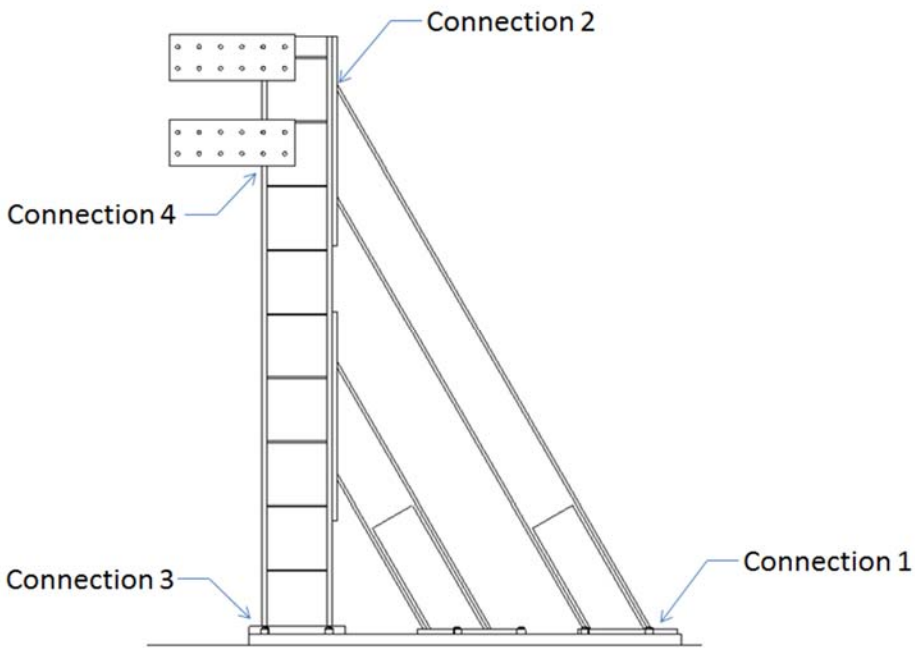


Figure 6 : Connection Overview

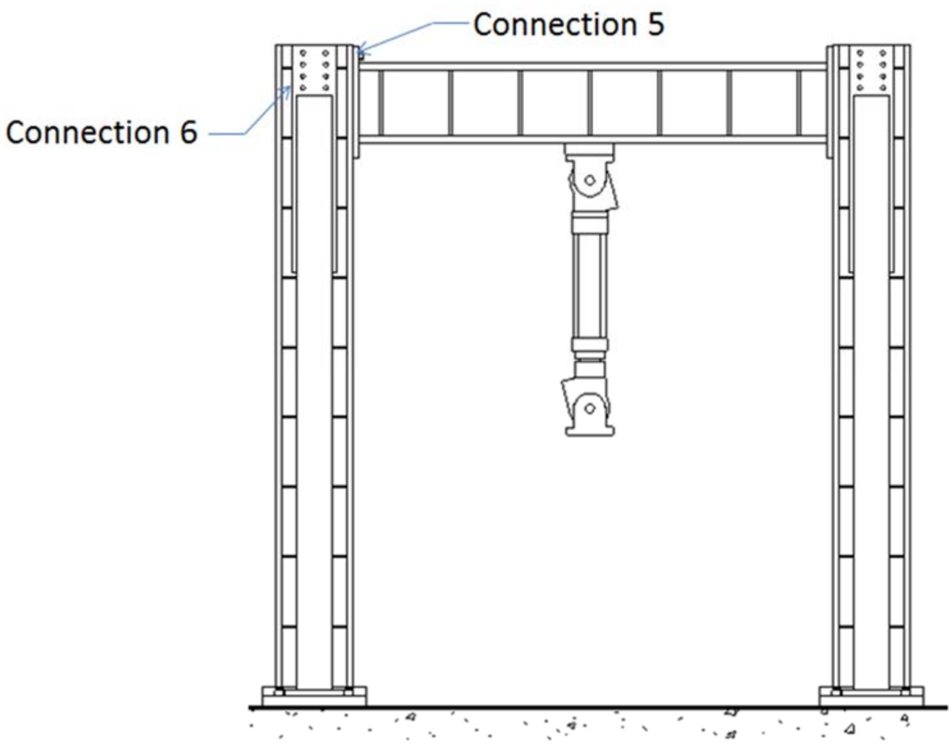


Figure 7 : Connection Overview Continued

Connectors – Geometry:

Since the frame is designed for slip-critical, oversized holes are allowed. Oversized holes are preferred for fit-up and constructability. All connectors will be 1.5" diameter threaded B7 bar. Anchor rod will be 2" diameter threaded rod, posttensioned with CY Series supernuts.

- Min. edge distance: 1 5/8" * d = 2.44" (AISC Table J3.4)
- Min spacing = 2.67 * d = 4.0" (AISC J3.3).
- Hole sizing = d + 5/16" = 1-13/16" (AISC Table J3.3 for oversized holes)

Connectors – Strength:

Due to the high strength requirements of the connections in this frame, high strength B7 bar is preferred over traditional bolts. A325, A490 and B7 bar connectors were considered for all connections. B7 threaded rod connectors are recommended because of their strength and fatigue properties. Connections will be designed using AISC ASD method with the following Strength Checks performed on all bolted connections:

- J3.6 Tension and Shear strength of bolts and threaded parts
- J3.7 Combined tension and shear in bearing-type connections
- J3.9 Combined tension and shear in slip-critical connections

Connectors – Fatigue:

Bolts in all connections of the reaction frame will see repeated cyclic loading and unloading, and so they must be designed for fatigue. The 2009 RCSC Specification for high-strength bolts (RCSC 2009), section 5.5 and the 2011 AISC specification (AISC 2011), Appendix 3 section 3.4(b) were used to calculate the fatigue strength for the connectors. See table 2 for results, and Appendix A for detailed calculations. If was found that since $F_{SR} < F_{TH}$ for all connectors (AISC), they will have indefinite design life.

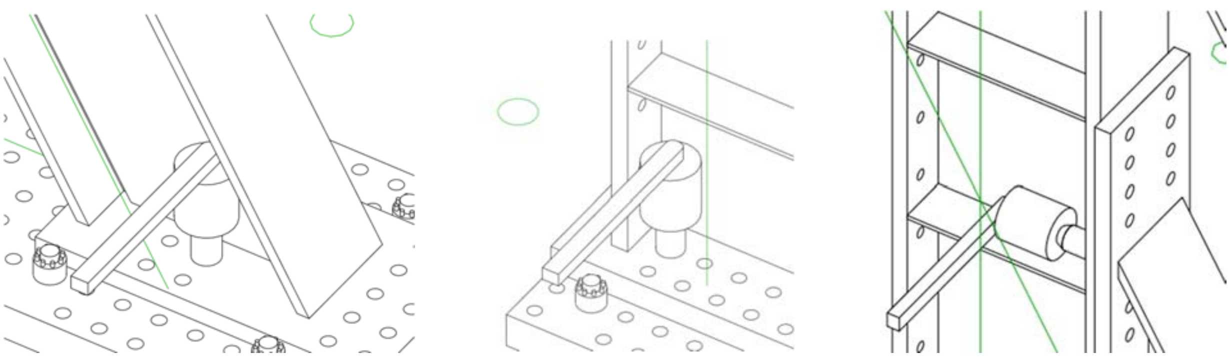
Connectors - Prying

AISC 9-10 was used to consider prying forces in connections. This section states that prying forces need not be considered if a t_{min} can be found that will make prying force = 0. Connector plate thicknesses were sized or checked to avoid prying forces.

Connectors - Connection Plates

Drawing from experience and recommendation from other structural engineering laboratory managers, all connection plates will be 2" thick, with the exception of the column connection plates which will be 3" thick due to a significant moment that could potentially occur at that connection.

Connectors - Wrench Clearance



Because B7 bar connectors require such high torque, a torque multiplier is needed. All connections were designed so that a torque wrench with multiplier can access each bolt.

Stiffeners

Based on the worst case horizontal reaction of 330 kips at this connection, stiffeners are NOT required in either the brace or the column. However, stiffeners may be added to the members to increase stiffness additionally. The following checks from the AISC steel manual have been performed on the crossbeam (point load from the actuator) and the column (point load from the crossbeam), See "stiffener design.pdf" for details.

Update: Stiffeners are included for the brace connections in order to keep the connection plates from warping from the CJP weld.

- J10.1 Flange Local Buckling
- J10.2 Web Local Buckling
- J10.3 Web Crippling

Results

The connection checks and calculation results are summarized in the table below.

Connection No.	1	2	3	4	5	6
Description	Brace - Anchor	Brace - Column	Column - Anchor	Crossbeam - Column	crossbeam to column	brace to column
worst-case	case 1	case 1	case 9	any	vertical	vertical
Connectors	B7 Threaded	B7 Threaded	B7 Threaded	B7 Threaded	B7 Threaded	B7 Threaded
Connector Diameter	1.5"	1.5"	1.5"	1.5"	1.5"	1.5"
No. of Connectors	20	30	20	4		
AISC Checks	J3.6, J3.7, J3.9	J3.6, J3.7, J3.9	J3.6, J3.7, J3.9	J3.6	J3.6, J3.7, J3.9	J3.6, J3.7, J3.9
Limiting Check	J3.9	J3.9	J3.9	J3.6	J3.7	J3.7
Fatigue Check	ok	ok	ok	ok	ok	ok
Connector Plate Thickness (in.)	2	2	3	2.48	2	0.81
t_{min} (in.)	1.81	1.64	2.64	1.19	1.27	0.35
Prying Check	ok	ok	ok	ok	ok	ok

Table 3: Connection Results Summary

WELDS AND BASE METAL FATIGUE

Weld Fatigue

Fatigue is the main restricting factor for the welds on the reaction frame. Welds are located at connection plates, and transfer the force from the elements to the plates. Fillet welds do not have enough fatigue strength in this situation and so CJP welds are required. The welds will experience both tension and compression loading. The welds are checked by AISC Appendix 3, and are taken to be class C. Our weld stress range is known from analysis, and N, the number of cycles allowed at that stress range is found using equation (A-3-1). See table 3 below for a summary of the weld fatigue strength. It was found that the sections alone couldn't take the applied forces in the worst cases, and so doubler plates were added to the webs of the braces and the columns to increase the weld areas.

Base Metal Fatigue

Fatigue in the base metal of the elements is subject to tension and compression, similar to welds. The normal stress used is a combination of axial and bending stresses and the AISC category is B because of the holes in the base metal. The results of the fatigue analysis are also listed in the table below.

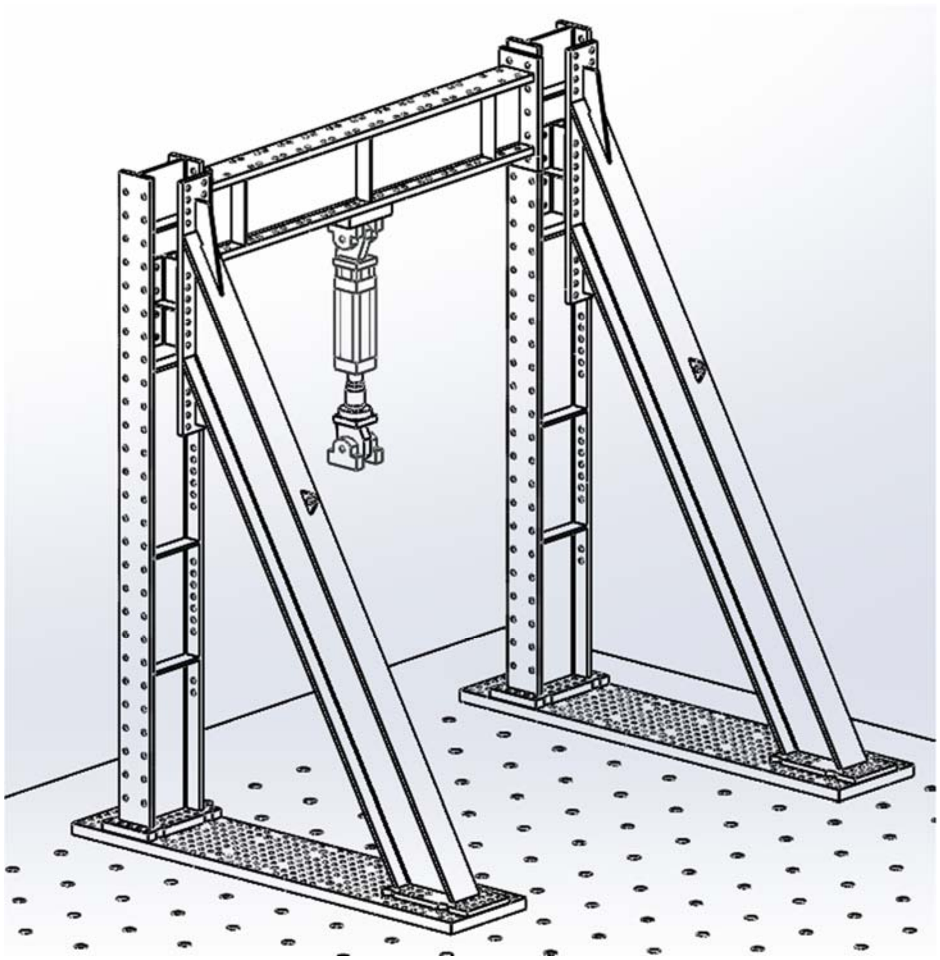
Loading Cases

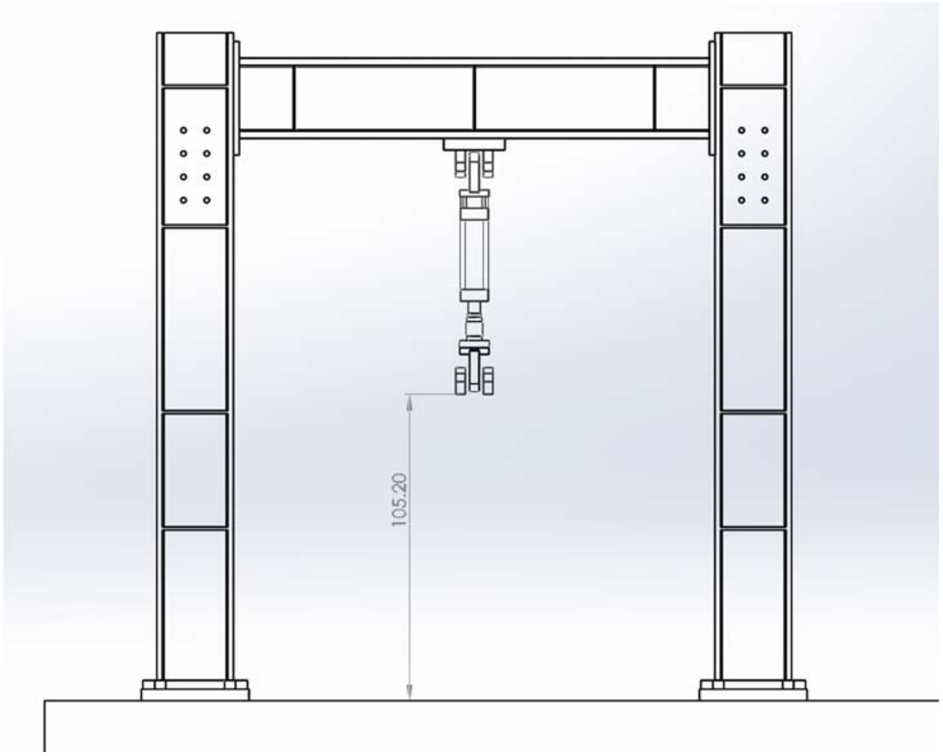
It was found that fatigue is significantly limiting for the worst loading cases. It is possible that the frame will never see such loading, so it is helpful to know the fatigue life N of welds and base metal at reduced loading. The results for number of cycles allowed, N, are summarized in Table 3.

		Total Actuator Force Applied (kips)	stress range (ksi)	$t_{doubler\ plate}$ (in.)	N (cycles)
Full Loading Case	Bolts				inf.
	Column Base Metal	950	20	na	322,642
	Weld @ Conn. 1	950	15.46	0	340,041
	Weld @ Conn. 2	950	12.56	0	2,017,086
	Weld @ Conn. 3	950	30.3	0	156,804
Half Loading Case	Column Base Metal	475	10	na	2,586,511
	Weld @ Conn. 1	475	7.75	0	2,725,995
	Weld @ Conn. 2	475	6.28	0	16,170,310
	Weld @ Conn. 3	475	15.15	0	1,257,050
Light Loading Case	Column Base Metal	250	5.26	na	17,775,107
	Weld @ Conn. 1	250	4.07	0	21,853,404
	Weld @ Conn. 2	250	3.31	0	129,632,031
	Weld @ Conn. 3	250	7.97	0	10,077,351

Table 4: Allowable Loading Cycles for Varying Loading Cases

VERTICAL CONFIGURATION

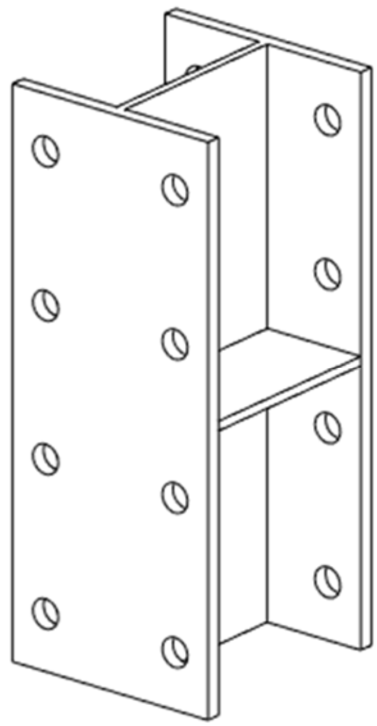




Overview

The reaction frame has been designed with the ability to change from the horizontal loading configuration to the vertical configuration using the same components. And adapter connection between the brace and the column is required due to the unique geometry. The vertical height of the crossbeam can be changed by 8” increments along the height of the columns. Specimen height can be up to 7’6” tall with the largest actuator or 8’-9” for the medium actuator (shown).

Brace to Column Connection:



Since the columns are rotated 90 degrees for the vertical configuration, the braces no longer align with the columns and will require an adapter for the connection. The requirement for the adapter is that it should be able to handle 10% of the vertical force as an out of plane load.

REFERENCES

AISC (2011). Steel Construction Manual, Fourteenth Edition, American Institute of Steel Construction, Chicago, IL.

Fed (1984). Federal Standard Screw-Thread Standards for Federal Services Section 2, Office of Federal Supply and Services

RCSC (2009), Specification for Structural Joints Using High-Strength Bolts, Research Council on Structural Connections, Chicago, IL.