STEEL TO CONCRETE CONNECTIONS

Senior Project:

Spring 2021

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Project Description:

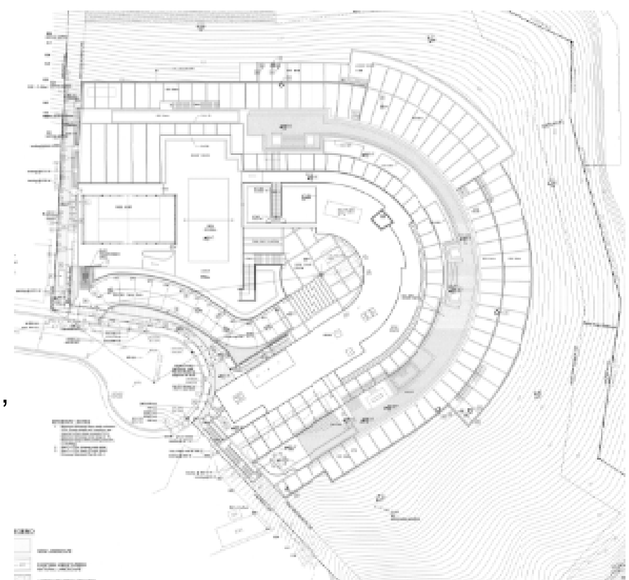
Angelo View Residence is a new single-family home located in Beverly Hills California. The site of the home sits atop Benedict Canyon at the end of Angelo View Dr. cul-de-sac. With enthralling views of the Santa Monica Mountains to the heart of downtown Los Angeles, the structure is quite a spectacle. The project was initiated sometime in 2016, and is set to finish construction on the beginning of 2022. This report presents the detailed design and construction of steel beam to concrete wall connections the Senior Architectural Engineering student, Dylan Thompson, participated in over the summer of 2020 and spring of 2021.

***Image 1**: Rendering by Uberion Architects*

Residence Description:

Architectural Features:

Filling 25,520 sq-ft, the modern designed home contains 7 bedrooms, 16 bathrooms, 7 lounge rooms, 2 offices, an indoor spa/grotto, an indoor sauna room, an indoor steam room, an indoor salon, an indoor message rooms, 2 game rooms, an indoor gym, a kitchen, an electrical room, 2 pool equipment rooms, 4 storage rooms, a security guard room, an outdoor fire pit,

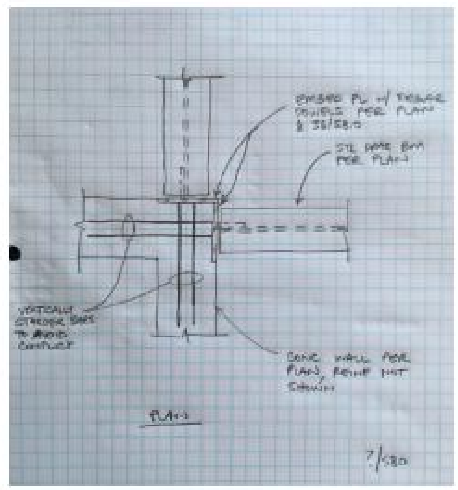


***Drawing 1**: Overall Site Plan by Uberion Architects*

33,589 sq-ft of concrete decks, 6,560 sq-ft of pools, a 12 car covered parking lot, a 12 car uncovered parking lot, and a 2 car garage. **Drawing 1** above shows the overall site plan of the project, and the U-shape of the structure. The site of the project lays on 3.66 acres of undeveloped land, which sits at an average slope of 1:2.

Structural Features:

The lower portion of the structure contains 52 concrete piles spanning from 20 ft deep to 38 ft deep of embedment into soil, 668 ft of grade beams connecting the piles, 1,105 ft of concrete retained walls, and 1,470 ft of concrete shear walls. The upper portion of the structure contains 20 steel drag beams, 14 wood drag beams, 254.5 ft of wood shear walls, 45-2' long Simpson Strong Tie 'Hardy Walls', and 625 ft of metal straps.



***Sketch 1**: Initial Concept Sketch of Corner Condition by Sage Shingle*

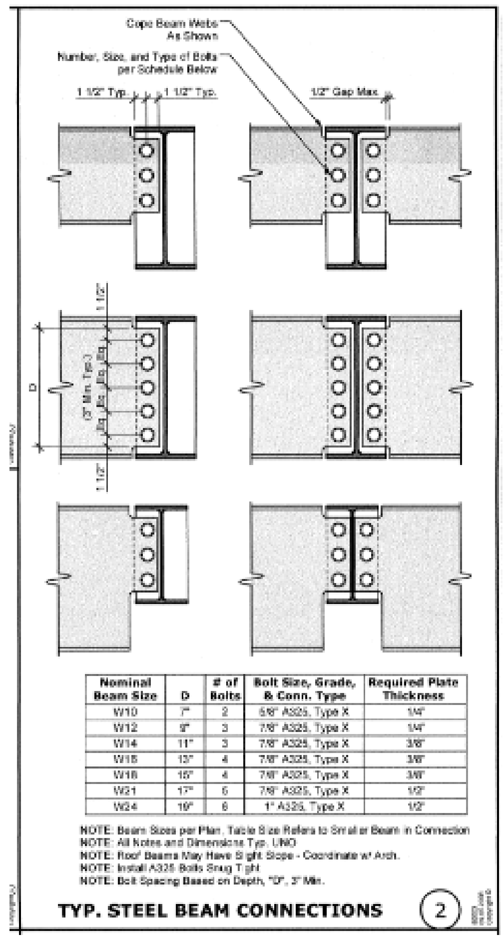
The structure was designing using multiple programs: excel spreadsheets was used for material dead loading and seismic distribution of forces to shear walls, ADAPT-Builder was used for design of concrete diaphragms, and RISA 3D was used for design of the complex framing systems.

Companies Involved:

Multiple companies were involved on the project due to the scope of work required for the structure. Greg Smith from *Uberion Architecture & Design* was the lead architect, Tyler Gold from *The L.A. Group* was the lead landscape architect, Garrett Mills from *Taylor and Syfan Consulting Engineers* was the lead structural engineer, Chris Peck from *CM PECK Inc.* was the lead civil engineer, Greg Byrne from *Grover-Hollingsworth and Associates*

Inc. was the lead geotechnical engineer, Sam Nakhla from *NAI Consulting* Engineers was the lead mechanical engineer, Eric Widmer from *Peak Surveys Inc.* was the lead land surveyor, and Aric Entwistle from *H2O Developement, Inc.* was the lead pool designer.

With big, complex projects such as this, multiple engineers and designers from each discipline are brought into the project to accomplish the tasks at hand in an organized fashion. Taylor and Syfan, for example, had 6 engineers, including myself, working on the project at at any given time. Coming onto the project during the design phase, I was thrown into this mixing pot of structural engineers and had to communicate to individuals with varying levels of experience and engineering "know how". One of which, Sage Shingle, a licensed structural engineer with over 20 years of experience, was my lead reference for the job. He guided me through the detailing of the connections, and provided references to perform necessary calculations. **Sketch 1** above is a concept hand-drawn sketch provided by Sage to guide me through the design of a specific corner condition where 2 perpendicular beams framed into one wall. Tony Rosemann, another seasoned engineer at Taylor and Syfan, was my next reference on the project and provided me with loads from his RISA-3D model, that I would later use to justify my connections.



Drawing 2*: Typical Steel Beam Connection Detail by Taylor & Syfan Engineers*

Basis of Design:

The building system is composed of a rigid diaphragm concrete deck with concrete bearing/shear at the second floor, a flexible wood diaphragm with concrete bearing/shear walls at the third floor, and a flexible wood diaphragm with wood shear walls at the roof. This allowed for the systems to be designed separately per ASCE 7-16: 12.2.3.2 (Two-Stage Analysis Procedure). **Table 1** below summarizes the structural parameters of the building and the resulting design base shear. The flexible diaphragms and wood shear walls were designed using the NDS 2018 and the Equivalent Lateral Force Procedure while the rigid diaphragm and concrete shear walls were designed using ACI 318 and RISA-3d software. The relative loads from the superimposed flexible structure above were applied and corresponding stress distributions in the concrete decks and loads applied to the concrete shear walls were found.

Redundancy Factor (ρ)	1.0
Seismic Design Category	E
Spectral Response Coefficients (S_{DS}, S_{D1})	1.566, 0.728
Design Wind Pressure	52.5 psf
Response Modification Coefficient (R)	5 (Special Reinforced Conc. S.W.) 6.5 (Light Framed Wd. S.W. w/ Plywood)
Design Base Shear (1.0E)	633.1 kips

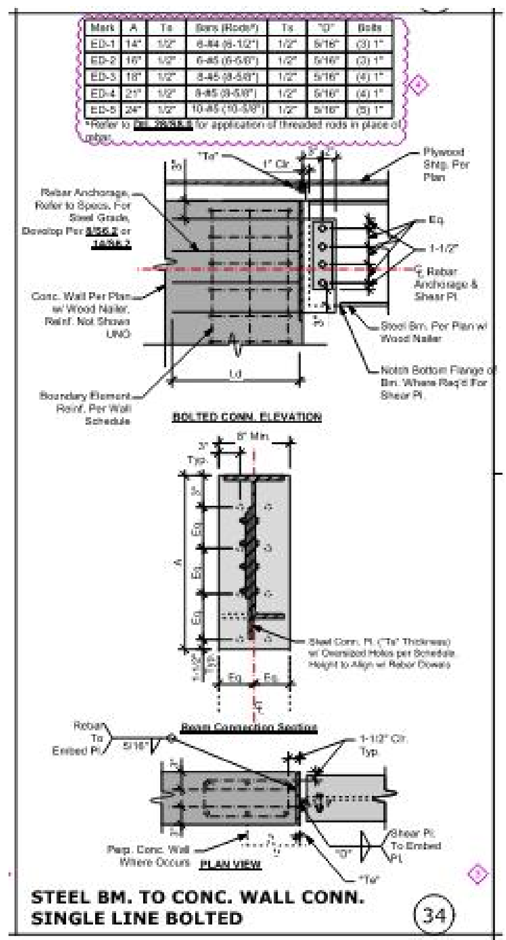
Table 1: *Structural Parameters*

To perform the necessary calculations described later in this report for the steel drag and gravity beams, the AISC Steel Construction Manual and equivalent loads from the Equivalent Lateral Force Procedure were used. Design examples from AISC were used to categorize connections and perform necessary calculations per code.

Design of Steel Beam Connections:

Background:

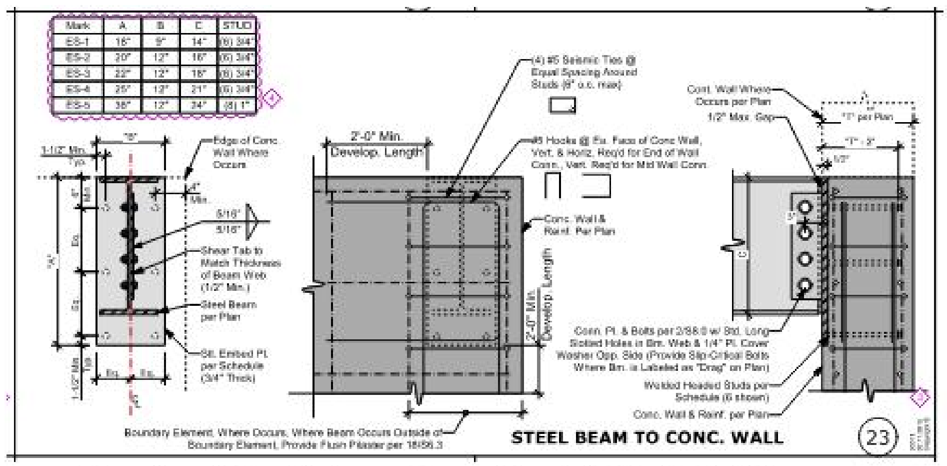
I was brought onto the project in July of 2020 to design connections for steel beams to concrete walls. The project had already been submitted to the city of Los Angeles for plan check with these connections under a deferred submital. All the connections occurred on the second floor level, and at similar heights. The beams themselves had already been designed to withstand the horizontal and vertical loads imposed on them from the structure. The reaction loads from these beams varied from 4.4 thousand pounds to 125.7 thousand pounds horizontally and 4.2 thousand pounds to 98.3 thousand pounds vertically. Given the loads and dimensions of the beams and walls, I was assigned to develop a spreadsheet to summarize all the connections, develop a spreadsheet to design the connections, provide a detail that satisfies all the connections and plan notes that provide information on the type and location of the connection.



***Drawing 3**: Low Load Steel Drag Beam Connection Detail by Dylan Thompson*

Categorizing Beam Connections:

Upon receiving a plan sheet with reaction loads from Tony Rosemann, I compiled the connections into subcategories: gravity beam connections and drag beam connections. The reactions for the gravity connections reflected the size of the beam being connected (i.e. the higher the load the bigger the beam). For this reason, it was determined that a connection would be designed for the maximum load for each depth of beam. 4 connections in total were designed for gravity beams ranging from W14x to W24x excluding W18x as there was none of these in the project. The drag beam connections proved harder to categorize as vertical and horizontal reactions varied from connection to connection as well as the beam depth. For each beam depth, multiple connections were required for low combinations of loads, medium combinations of loads, and high combinations of loads totaling 15 connections.



Drawing 4: *Gravity Beam Connection Detail by Dylan Thompson*

Gravity Beams:

Once categorized, the design of the connections started. For the gravity connections, one detail was desired to satisfy all connections utilizing a table and a generic layout of connections. **Drawing 4** below shows the finished detail with 3 section views and a table for elements that vary. The connection was also required to be placed perpendicular to concrete walls varying from 8” to 18” thick and have the ability to be placed at the end-of or in the mid-span of such wall. Starting with the concrete, headed studs would be welded to a steel plate embedded in the wall to be flush with the edge of concrete. #5 rebar U hooks would be placed on near and far side in the concrete to develop our load into the wall. This is important as this eliminates concrete breakout as a failure mechanism as we provide reinforcing across the failure plane. This allows for the connection to rely on the strength of the welded studs to the plate which results in higher capacities for the connection. A shear-connection plate with slotted holes is then welded perpendicular to the embed plate, and the steel gravity beam can be directly bolted to it. Since the bolt line occurs 3” from the face of concrete, an induced moment was accounted for in design of the embed plate and headed studs. In order to satisfy all connections, the length and width of the embed plate and number of studs was varied over the 4 connections and summarized in the table within the detail.

3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb)
1	4563.4	0.0	12287.5	12287.5
2	2584.6	0.0	12287.5	12287.5
3	605.8	0.0	12287.5	12287.5
4	0.0	0.0	12287.5	12287.5
5	0.0	0.0	12287.5	12287.5
6	605.8	0.0	12287.5	12287.5
7	2584.6	0.0	12287.5	12287.5
8	4563.4	0.0	12287.5	12287.5
Sum	15507.7	0.0	98300.0	98300.0

Maximum concrete compression strain (‰): 0.07
Maximum concrete compression stress (psi): 288
Resultant tension force (lb): 0
Resultant compression force (lb): 15508
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.00
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 0.00
Eccentricity of resultant shear forces in x-axis, e'_{Vx} (inch): 0.00
Eccentricity of resultant shear forces in y-axis, e'_{Vy} (inch): 0.00

<Figure 3>

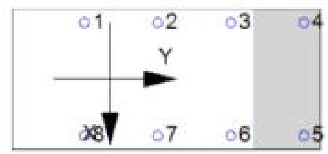


Figure 1: *Resulting Distribution of Forces to Welded Rebar Anchors by 'Simpson Anchor Designer'*

Drag Beams:

After completing the steel gravity beam detail, it was time to move onto the steel drag beam connections. Since it was determined 3 combinations of load demands would be required for each beam depth (low, medium, high), 3 separate details would be designed utilizing a single line of bolts, a double line of bolts, and a direct welded connection. **Drawing 3** above shows the finalized detail for low-load combinations. Starting at the concrete, since the steel drag beams frame parallel into the end of the concrete shear walls, rebar (instead of headed studs) was welded to a steel plate embedded in the wall to be flush with the edge of concrete. No additional reinforcing in the wall was required as the concrete shear walls had been previously designed with boundary elements and vertical and horizontal reinforcement curtains. A connection plate would then be welded perpendicular to the embed plate and connected to the drag beam. This connection is where each detail would vary. For smaller combinations of loads, a single line of bolts from the connection plate to the steel beam web would satisfy the demand of the connection. For medium combinations of loads, a double line of bolts would be utilized and for high combinations of loads the connection plate would be welded directly to the steel beam web.

Calculation of Steel Beam Connections:

Gravity Beams:

Once the general plan of attack was determined, the individual connection would need to be designed for a complete load path. For the gravity connections, *Simpson Anchor Designer* was used to determine the size and spacing of the headed studs and geometry of the embed plate. The software automatically distributed the forces to each anchor and provided the distribution of stresses in the embed plate shown in **Figure 1** and **Figure 2** above. The connection plate weld to the embed plate was checked for all load combinations to ensure a 5/16 Fillet weld on both sides would work. The bolts from the connection plate to the embed plate were not designed as a typical detail in the project was used for all steel beam bolted connections. This typical detail is shown in **Drawing 2** above.

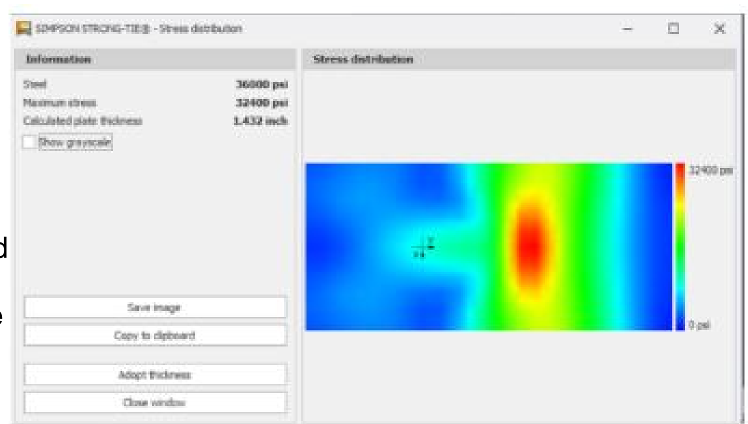


***Figure 2**: Resulting Stress Distribution of Embedded Steel Plate by 'Simpson Anchor Designer'*

Drag Beams:

Next was to calculate the drag beam connections. For simplicity, I created an excel spreadsheet that automatically performed all necessary calculations and gave a DCR ratio for the connection. The first sheet was used for inputting the beam's general information: geometry of the beam, horizontal and vertical loads at the connection, size and layout of the rebar, size and layout of bolts or size of fillet weld, geometry of the shear-connection plate and geometry of the embed plate. The first sheet would then calculate the resultant load on each rebar anchor using a centroid calculation (similar to a concrete beam) and the stress demand on the weld to the connection plate. A diagram for this distribution is provided in **Figure 3** above. All of this would remain the same from a low-load connection to a high-load connection as to why it was located on the first sheet. Past the connection plate, a separate sheet was created for each combination of loads (single bolt line, double bolt line, and weld). AISC defines the single bolt line drag beam connection as a conventional configuration and only requires a calculation check for the bolts and a maximum thickness requirement of the connection plate. The beam web and connection plate were checked for bolt bearing strength and bolt tear-out strength. The double bolt line drag beam connection is defined by AISC as an extended configuration and requires more checks than that of the conventional

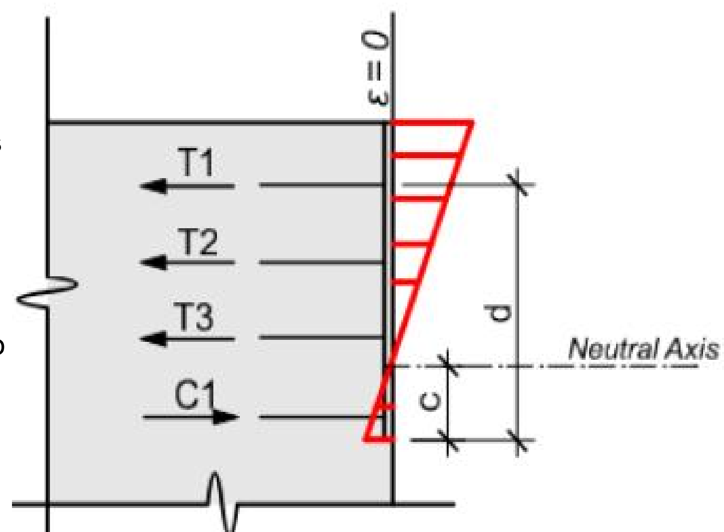


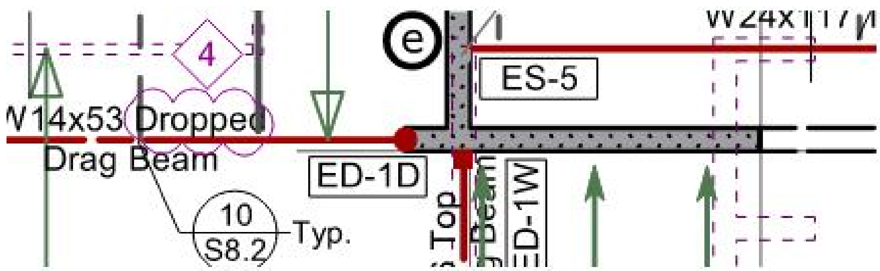
***Figure 3**: Diagram of Distribution of Forces to Welded Rebar Anchors by Dylan Thompson*

configuration. Similar bolt bearing and tear-out calculations were performed, followed by shear/tensile yielding and shear/tensile rupture of the beam web and the connection plate. The connection plate would also be checked for lateral-torsional-buckling, interaction of axial/flexural/shear yielding and axial/flexural/shear rupture, and 4 possible block shear ruptures. The welded connection did not have a definition given to it by AISC, and as such was checked for all similar failure mechanisms as the extended configuration excluding bolt bearing, bolt tear-out, and block shear of the plate.

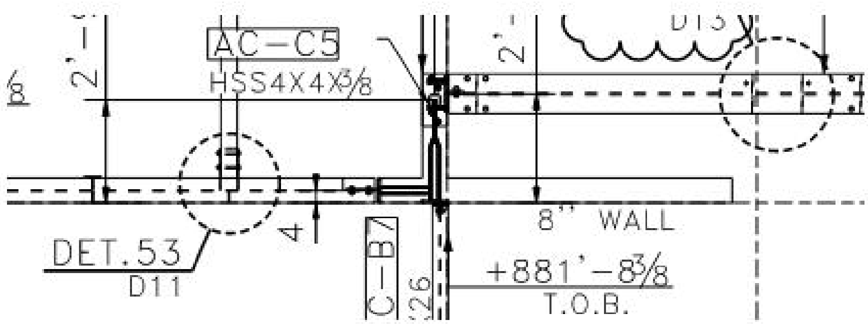
Application of Steel Beam Connections:

Steel Shop Drawings:

Once the details and necessary calculations were concluded, the drawings were resubmitted to the city for approval and to the steel fabricator for shop drawings to be drawn. On April 22, 2021 the steel shop drawings were returned to the structural engineer (my company) and architect for approval/coordination. The shop drawings included plans, details, and fabrication documents for the base plates, the steel columns, the steel beams, and the exterior steel trellis



Engineer's Construction Document



Steel Fabricator's Shop Drawing

Figure 4: *Comparison of Construction Documents to Steel Fabrication Shop Drawings*

framing system. I was tasked with scanning the 382 page long document and verifying that all steel members and connections reflected the construction documents provided to the fabricator. At this stage of the building process, all minute areas of member alignment and connection configurations, down to the 1/16 of inch, were resolved. In other words, the steel fabricator took our schematic documents, created an extremely accurate drawing set, and returned it to us with highlighted areas that they found to be inconsistent with our detailing. **Figure 4** above communicates this process by comparing the detailing of a wall with multiple members framing into it at different locations. The fabricator took some liberty here and offset the beam at the bottom of the figure from its embed plate to have its rebar align with the parallel wall it was framing into. Once these issues were resolved and coordinated between the architect and engineer, the shop drawings were returned to the fabricator and all steel elements were issues to be cut, hole-punched, and welded.

***Image 2**: View of Downtown Santa Monica During Site Visit by Dylan Thompson*

Site Visit:

On May 14th, I was given the opportunity to accompany Garrett Mills (the principal engineer in charge of the

***Image 3**: Experimental Board Form Concrete Wall Observed on Site Visit by Dylan Thompson*

project) on a routine site visit to meet with Mike Thrane (the general contractor) to inspect some areas where reinforcement has been placed and is ready for concrete pouring. Steel elements were still in the process of being constructed in the fabrictor's shop at the time of the visit. While on site, I was able communicate with Mike and the laborers on how they usually construct the embedded plates I designed, observe some various kinds of board-form they were experimenting with (**Image 3**), and view parts of the shoring system that were still exposed at the time of the visit (**Image 4**). This gave me a great sense of scale of the connections I designed and process by which they were constructed.

__Image 4__: Exposed Soldier Pile and Timber Lagging Shoring System Observed on Site Visit by Dylan Thompson

Takeaway:

Social and Economic Considerations:

Beyond the engineering, tremendous information was gained on the construction industry itself. Being a part of the engineering, the steel shop drawing review, and site visit documentation gave me a great perspective of the process as a whole. I now know why engineering documents for these scale of projects are very schematic in plan, as issues will be resolved with the steel shop drawings and revisions later on to keep the project moving. This is key as jobs of this scale could cost the owner thousands of dollars a day to be on hold if timelines between respective traits do not line up.

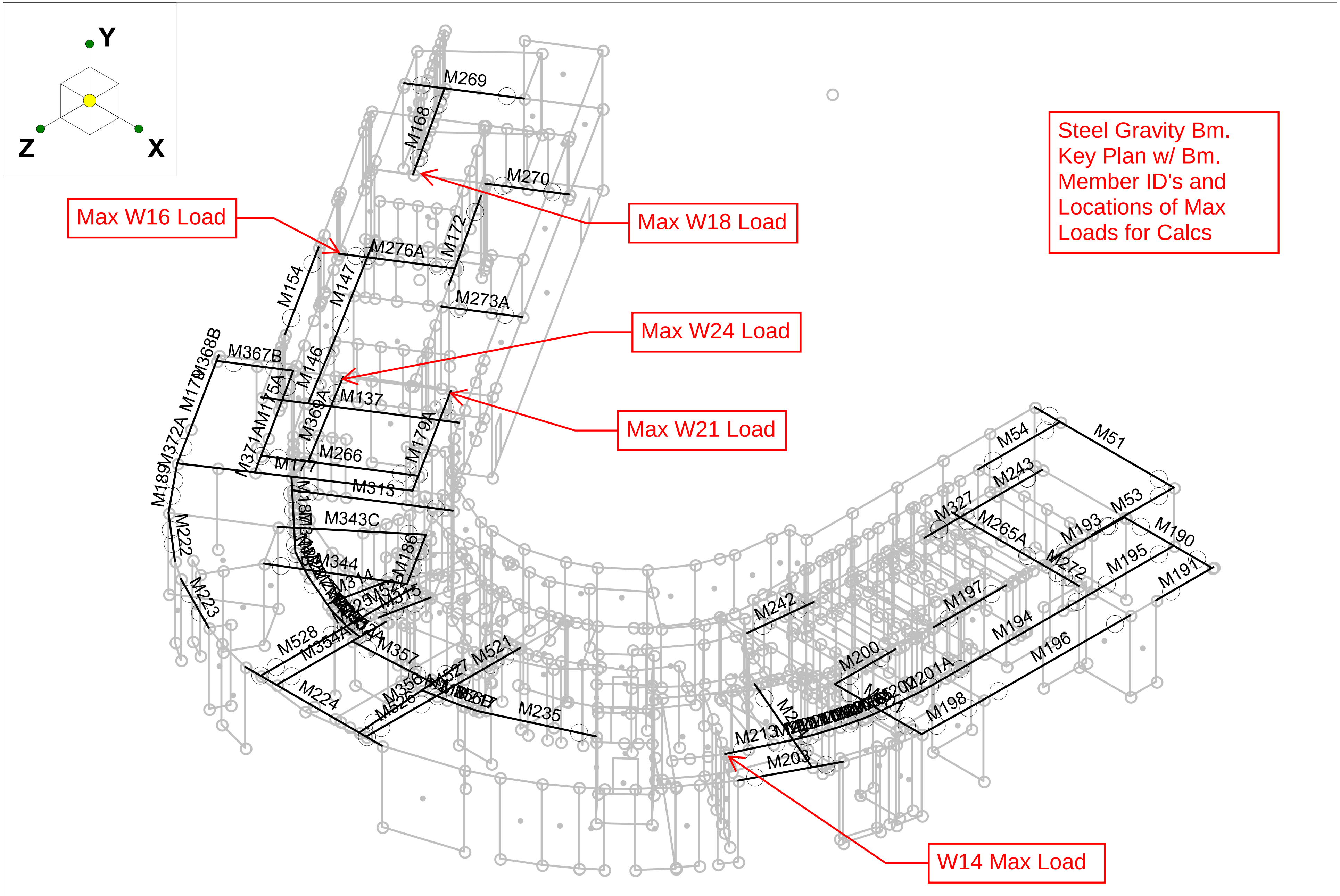
Reviewing the steel shop drawings gave me insight into why these connections are desired to be field bolted, with all welding to be done in the shop unless specifically required (i.e. welded drag beam connection). Again, this saves the owner money in construction costs. Being able to communicate directly with the steel fabricator and architect let me develop a perspective on the relationships between respective traits. I learned how to ask questions and address RFI's clearly and consisely. I also have a scale for these types of connections now and am able to visualize the 2D drawings on paper in 3D in my head.

Overall:

Over the course of the project, I gained a tremendous amount of insight into the application of steel construction. I witnessed first hand the implications of combining flexible diaphragms with rigid diaphragms. Performing necessary calculations for drag beam connections gave me critical knowledge on how to connect steel elements to concrete. When detailing the connections, I absorbed the added benefit for creating generic details that could apply to multiply connections and simplify the process. I also gained experience on an appropriate timeline for the engineering of this type of structure and connection.

Overall, I am very grateful I got this opportunity to be a part of the team on this immaculate house. I gained a great appreciation for the construction industry and the laborers who make this work possible. I will think twice before specifying a W24x steel beam again.

Appendix A: Steel Gravity Beam Calculations



T&S	18412 - Villa Capri	SK - 3
ABR		Oct 8, 2020 at 4:16 PM
18412		18412 - Angelo View - FLEX Lateral 19F.r3d

Calculation For W14<
Gravity Anchorage

1.Project information

Customer company:
Customer contact name:
Customer e-mail:
Comment:

Project description:
Location:
Fastening description:

2. Input Data & Anchor Parameters

General
Design method:ACI 318-14
Units: Imperial units

Anchor Information:
Anchor type: Cast-in-place
Material: AWS Type A
Diameter (inch): 0.750
Effective Embedment depth, h_{ef} (inch): 4.125
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 5.63
C_{min} (inch): 1.38
S_{min} (inch): 3.00

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 8.00
State: Uncracked
Compressive strength, f'_c (psi): 4000
$\Psi_{c,V}$: 1.0
Reinforcement condition: B tension, B shear
Supplemental reinforcement: Yes
Reinforcement provided at corners: Yes
Ignore concrete breakout in tension: Yes
Ignore concrete breakout in shear: Yes
Ignore 6do requirement: No
Build-up grout pad: No

Base Plate
Length x Width x Thickness (inch): 9.00 x 18.00 x 0.50
Yield stress: 34084 psi

Profile type/size: W14X53

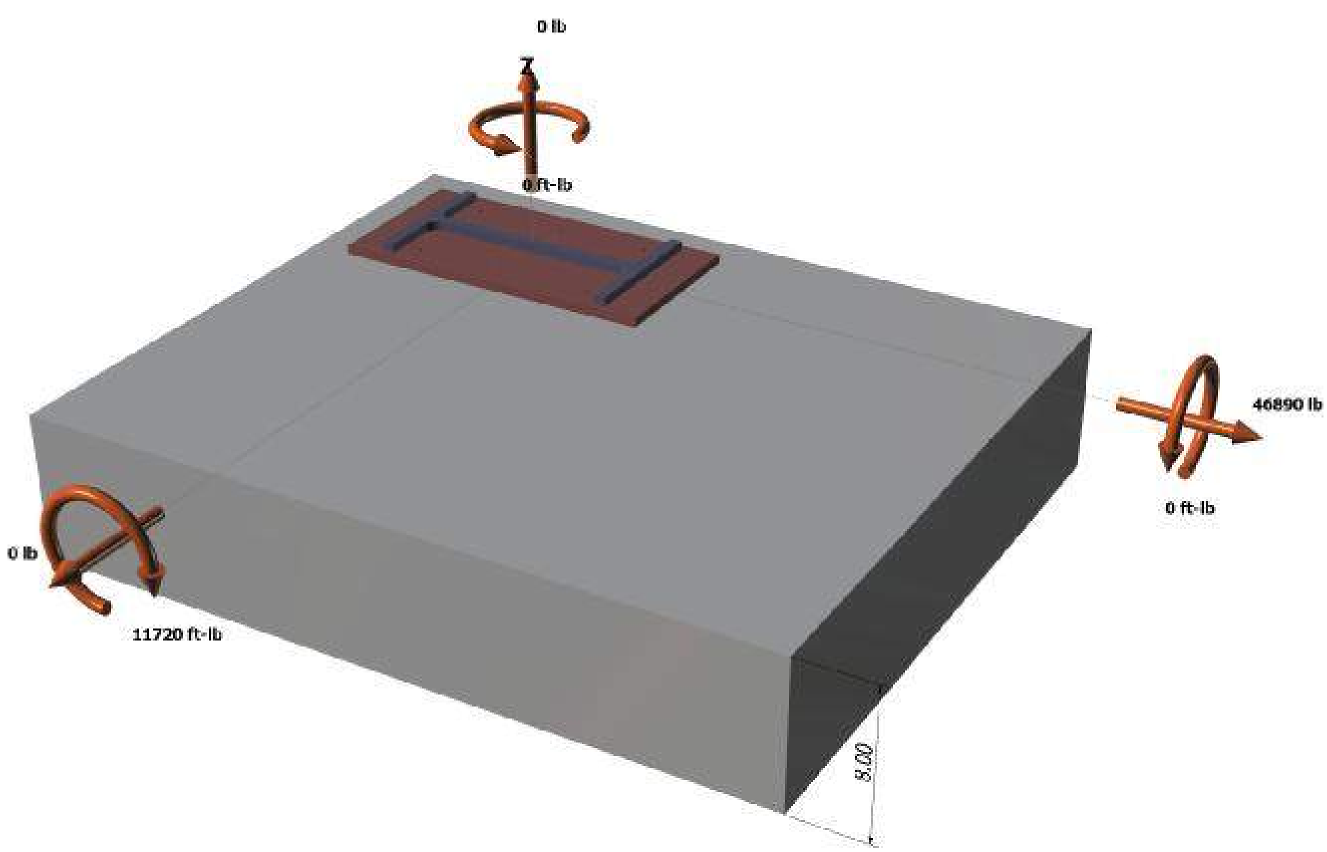
Recommended Anchor
Anchor Name: Headed Stud - 3/4"Ø AWS Type A Headed Stud

Load and Geometry
Load factor source: ACI 318 Section 5.3
Load combination: not set
Seismic design: No
Anchors subjected to sustained tension: Not applicable
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: No

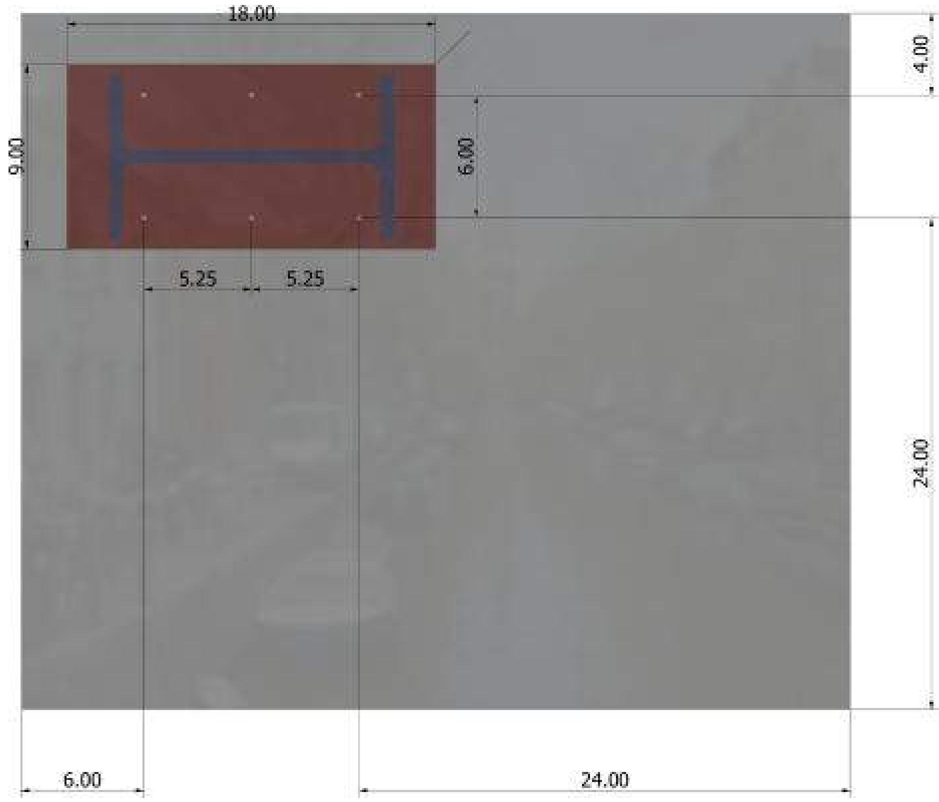
Strength level loads:

N_{ua} [lb]: 0
V_{uax} [lb]: 0
V_{uay} [lb]: 46890
M_{ux} [ft-lb]: -11720
M_{uy} [ft-lb]: 0
M_{uz} [ft-lb]: 0

<Figure 1>



<Figure 2>

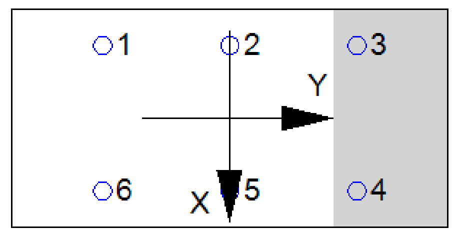


3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb)
1	4382.2	0.0	7815.0	7815.0
2	1977.2	0.0	7815.0	7815.0
3	0.0	0.0	7815.0	7815.0
4	0.0	0.0	7815.0	7815.0
5	1977.2	0.0	7815.0	7815.0
6	4382.2	0.0	7815.0	7815.0
Sum	12718.8	0.0	46890.0	46890.0

Maximum concrete compression strain (‰): 0.14
Maximum concrete compression stress (psi): 603
Resultant tension force (lb): 0
Resultant compression force (lb): 12719
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.00
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 0.00
Eccentricity of resultant shear forces in x-axis, e'_{Vx} (inch): 0.00
Eccentricity of resultant shear forces in y-axis, e'_{Vy} (inch): 0.00

<Figure 3>



4. Steel Strength of Anchor in Tension (Sec. 17.4.1)

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
26950	0.75	20213

6. Pullout Strength of Anchor in Tension (Sec. 17.4.3)

$\phi N_{pn} = \phi \Psi_{c,P} N_p = \phi \Psi_{c,P} 8 A_{brg} f'_c$ (Sec. 17.3.1, Eq. 17.4.3.1 & 17.4.3.4)

$\Psi_{c,P}$	A_{brg} (in^2)	f'_c (psi)	ϕ	ϕN_{pn} (lb)
1.4	0.79	4000	0.70	24618

8. Steel Strength of Anchor in Shear (Sec. 17.5.1)

V_{sa} (lb)	ϕ_{grout}	ϕ	$\phi_{grout}\phi V_{sa}$ (lb)
26950	1.0	0.65	17518

10. Concrete Pryout Strength of Anchor in Shear (Sec. 17.5.3)

$\phi V_{cpg} = \phi k_{cp} N_{cbg} = \phi k_{cp}(A_{Nc} / A_{Nco}) \Psi_{ec,N} \Psi_{ed,N} \Psi_{c,N} \Psi_{cp,N} N_b$ (Sec. 17.3.1 & Eq. 17.5.3.1b)

k_{cp}	A_{Nc} (in^2)	A_{Nco} (in^2)	$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	ϕV_{cpg} (lb)
2.0	367.25	153.14	1.000	0.894	1.250	1.000	12717	0.70	47709

11. Results

Interaction of Tensile and Shear Forces (Sec. 17.6.)

Tension	Factored Load, N_{ua} (lb)	Design Strength, ϕN_n (lb)	Ratio	Status
Steel	**4382**	**20213**	**0.22**	**Pass (Governs)**
Pullout	4382	24618	0.18	Pass

Shear	Factored Load, V_{ua} (lb)	Design Strength, ϕV_n (lb)	Ratio	Status
Steel	7815	17518	0.45	Pass
Pryout	**46890**	**47709**	**0.98**	**Pass (Governs)**

Interaction check	$N_{ua}/\phi N_n$	$V_{ua}/\phi V_n$	Combined Ratio	Permissible	Status
Sec. 17.6..3	0.22	0.98	120.0%	1.2	Pass

3/4"Ø AWS Type A Headed Stud with hef = 4.125 inch meets the selected design criteria.

Base Plate Thickness

Required base plate thickness: 0.451 inch

Max W14 Load = 35.6k
DCR = 35.6/46.89 = 0.759

12. Warnings

- Concrete breakout strength in tension has not been evaluated against applied tension load(s) per designer option. Refer to ACI 318 Section 17.3.2.1 for conditions where calculations of the concrete breakout strength may not be required.

- Concrete breakout strength in shear has not been evaluated against applied shear load(s) per designer option. Refer to ACI 318 Section 17.3.2.1 for conditions where calculations of the concrete breakout strength may not be required.

- Designer must exercise own judgement to determine if this design is suitable.

Calculation For W16
Gravity Anchorage

1.Project information

Customer company:
Customer contact name:
Customer e-mail:
Comment:

Project description:
Location:
Fastening description:

2. Input Data & Anchor Parameters

General
Design method:ACI 318-14
Units: Imperial units

Anchor Information:
Anchor type: Cast-in-place
Material: AWS Type A
Diameter (inch): 0.750
Effective Embedment depth, h_{ef} (inch): 4.125
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 5.63
C_{min} (inch): 1.38
S_{min} (inch): 3.00

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 8.00
State: Uncracked
Compressive strength, f'_c (psi): 4000
$\Psi_{c,V}$: 1.0
Reinforcement condition: B tension, B shear
Supplemental reinforcement: Yes
Reinforcement provided at corners: Yes
Ignore concrete breakout in tension: Yes
Ignore concrete breakout in shear: Yes
Ignore 6do requirement: No
Build-up grout pad: No

Base Plate
Length x Width x Thickness (inch): 12.00 x 20.00 x 0.75
Yield stress: 34084 psi

Profile type/size: W16X36

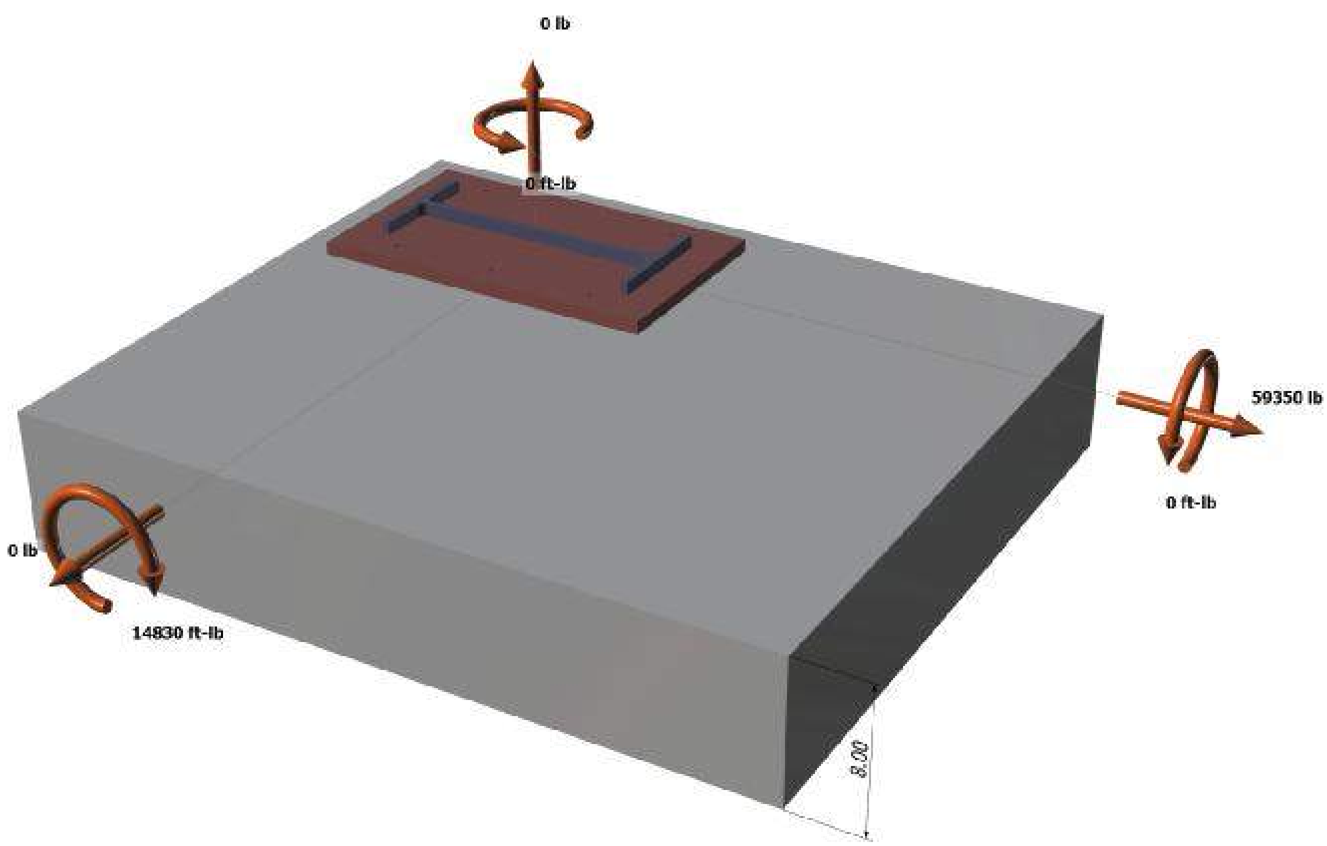
Recommended Anchor
Anchor Name: Headed Stud - 3/4"Ø AWS Type A Headed Stud

Load and Geometry
Load factor source: ACI 318 Section 5.3
Load combination: not set
Seismic design: No
Anchors subjected to sustained tension: Not applicable
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: No

Strength level loads:

N_{ua} [lb]: 0
V_{uax} [lb]: 0
V_{uay} [lb]: 59350
M_{ux} [ft-lb]: -14830
M_{uy} [ft-lb]: 0
M_{uz} [ft-lb]: 0

<Figure 1>



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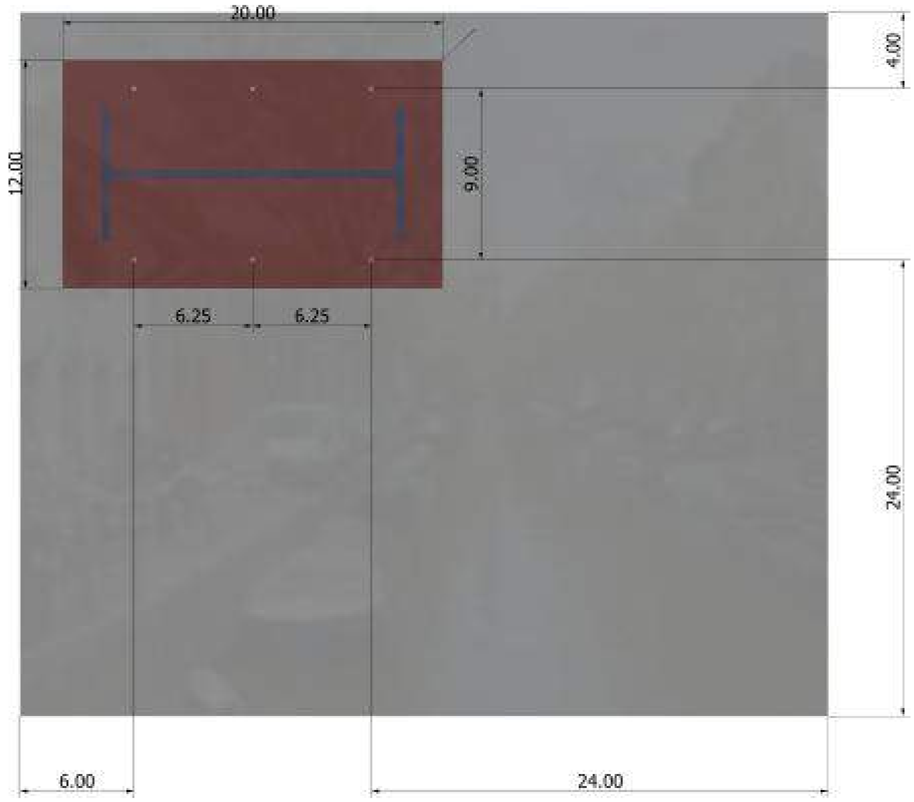
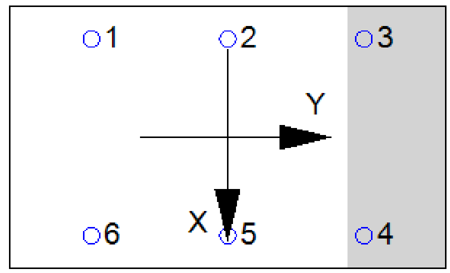


3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb)
1	4754.8	0.0	9891.7	9891.7
2	2221.9	0.0	9891.7	9891.7
3	0.0	0.0	9891.7	9891.7
4	0.0	0.0	9891.7	9891.7
5	2221.9	0.0	9891.7	9891.7
6	4754.8	0.0	9891.7	9891.7
Sum	13953.4	0.0	59350.0	59350.0

Maximum concrete compression strain (‰): 0.12
Maximum concrete compression stress (psi): 515
Resultant tension force (lb): 0
Resultant compression force (lb): 13954
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.00
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 0.00
Eccentricity of resultant shear forces in x-axis, e'_{Vx} (inch): 0.00
Eccentricity of resultant shear forces in y-axis, e'_{Vy} (inch): 0.00

<Figure 3>



4. Steel Strength of Anchor in Tension (Sec. 17.4.1)

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
26950	0.75	20213

6. Pullout Strength of Anchor in Tension (Sec. 17.4.3)

$\phi N_{pn} = \phi \Psi_{c,P} N_p = \phi \Psi_{c,P} 8 A_{brg} f'_c$ (Sec. 17.3.1, Eq. 17.4.3.1 & 17.4.3.4)

$\Psi_{c,P}$	A_{brg} (in^2)	f'_c (psi)	ϕ	ϕN_{pn} (lb)
1.4	0.79	4000	0.70	24618

8. Steel Strength of Anchor in Shear (Sec. 17.5.1)

V_{sa} (lb)	ϕ_{grout}	ϕ	$\phi_{grout}\phi V_{sa}$ (lb)
26950	1.0	0.65	17518

10. Concrete Pryout Strength of Anchor in Shear (Sec. 17.5.3)

$\phi V_{cpg} = \phi k_{cp} N_{cbg} = \phi k_{cp}(A_{Nc} / A_{Nco})\Psi_{ec,N}\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b$ (Sec. 17.3.1 & Eq. 17.5.3.1b)

k_{cp}	A_{Nc} (in^2)	A_{Nco} (in^2)	$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	ϕV_{cpg} (lb)
2.0	473.69	153.14	1.000	0.894	1.250	1.000	12717	0.70	61536

11. Results

Interaction of Tensile and Shear Forces (Sec. 17.6.)

Tension	Factored Load, N_{ua} (lb)	Design Strength, ϕN_n (lb)	Ratio	Status
Steel	**4755**	**20213**	**0.24**	**Pass (Governs)**
Pullout	4755	24618	0.19	Pass

Shear	Factored Load, V_{ua} (lb)	Design Strength, ϕV_n (lb)	Ratio	Status
Steel	9892	17518	0.56	Pass
Pryout	**59350**	**61536**	**0.96**	**Pass (Governs)**

Interaction check	$N_{ua}/\phi N_n$	$V_{ua}/\phi V_n$	Combined Ratio	Permissible	Status
Sec. 17.6..3	0.24	0.96	120.0%	1.2	Pass

3/4"Ø AWS Type A Headed Stud with hef = 4.125 inch meets the selected design criteria.

Base Plate Thickness

Required base plate thickness: 0.482 inch

Max W16 Load = 25.8k
DCR = 25.8/59.35 = 0.435

12. Warnings

- Concrete breakout strength in tension has not been evaluated against applied tension load(s) per designer option. Refer to ACI 318 Section 17.3.2.1 for conditions where calculations of the concrete breakout strength may not be required.

- Concrete breakout strength in shear has not been evaluated against applied shear load(s) per designer option. Refer to ACI 318 Section 17.3.2.1 for conditions where calculations of the concrete breakout strength may not be required.

- Designer must exercise own judgement to determine if this design is suitable.

Calculation For W18
Gravity Anchorage

1.Project information

Customer company:
Customer contact name:
Customer e-mail:
Comment:

Project description:
Location:
Fastening description:

2. Input Data & Anchor Parameters

General
Design method:ACI 318-14
Units: Imperial units

Anchor Information:
Anchor type: Cast-in-place
Material: AWS Type A
Diameter (inch): 0.750
Effective Embedment depth, h_{ef} (inch): 4.125
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 5.63
C_{min} (inch): 1.38
S_{min} (inch): 3.00

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 8.00
State: Uncracked
Compressive strength, f'_c (psi): 4000
$\Psi_{c,V}$: 1.0
Reinforcement condition: B tension, B shear
Supplemental reinforcement: Yes
Reinforcement provided at corners: Yes
Ignore concrete breakout in tension: Yes
Ignore concrete breakout in shear: Yes
Ignore 6do requirement: No
Build-up grout pad: No

Base Plate
Length x Width x Thickness (inch): 14.00 x 25.00 x 0.50
Yield stress: 34084 psi

Profile type/size: W18X50

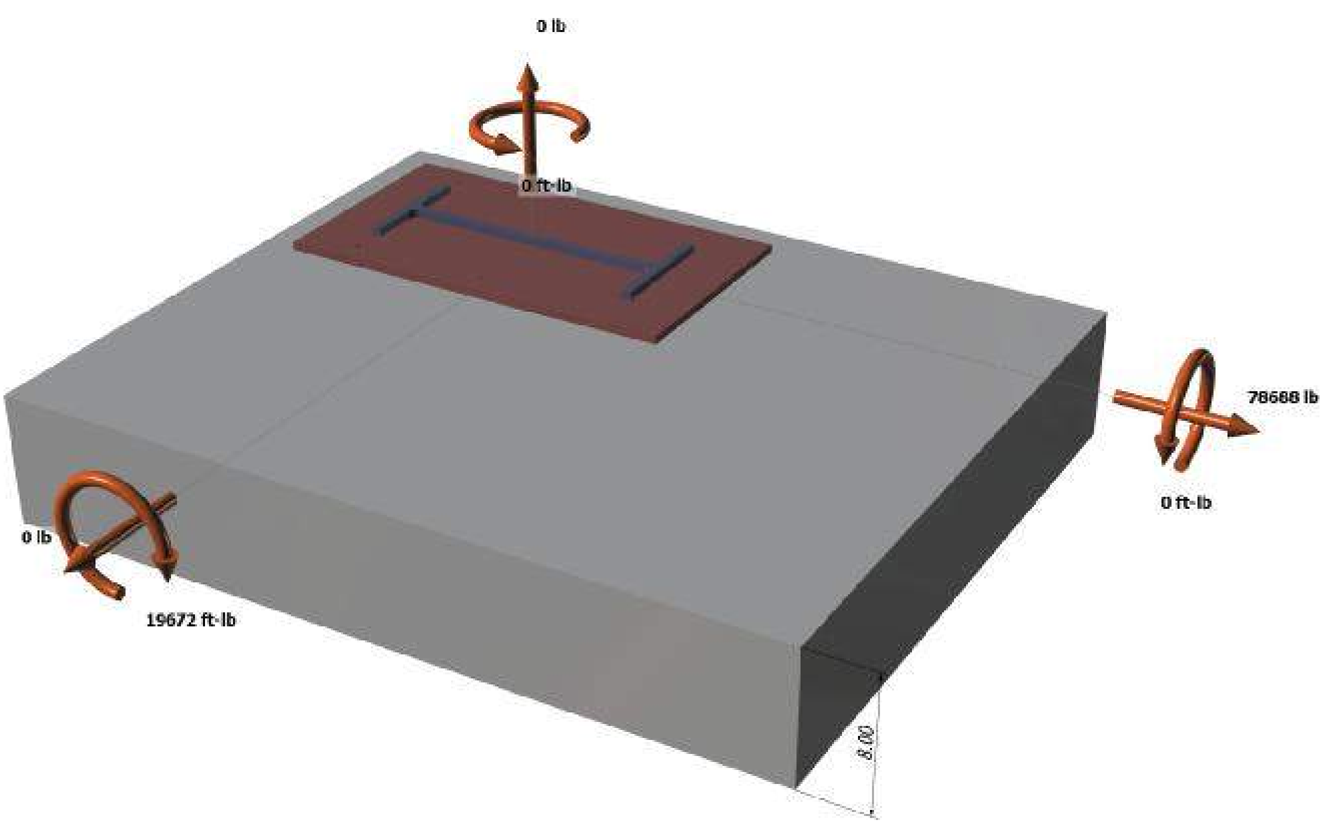
Recommended Anchor
Anchor Name: Headed Stud - 3/4"Ø AWS Type A Headed Stud

Load and Geometry
Load factor source: ACI 318 Section 5.3
Load combination: not set
Seismic design: No
Anchors subjected to sustained tension: Not applicable
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: No

Strength level loads:

N_{ua} [lb]: 0
V_{uax} [lb]: 0
V_{uay} [lb]: 78688
M_{ux} [ft-lb]: -19672
M_{uy} [ft-lb]: 0
M_{uz} [ft-lb]: 0

<Figure 1>



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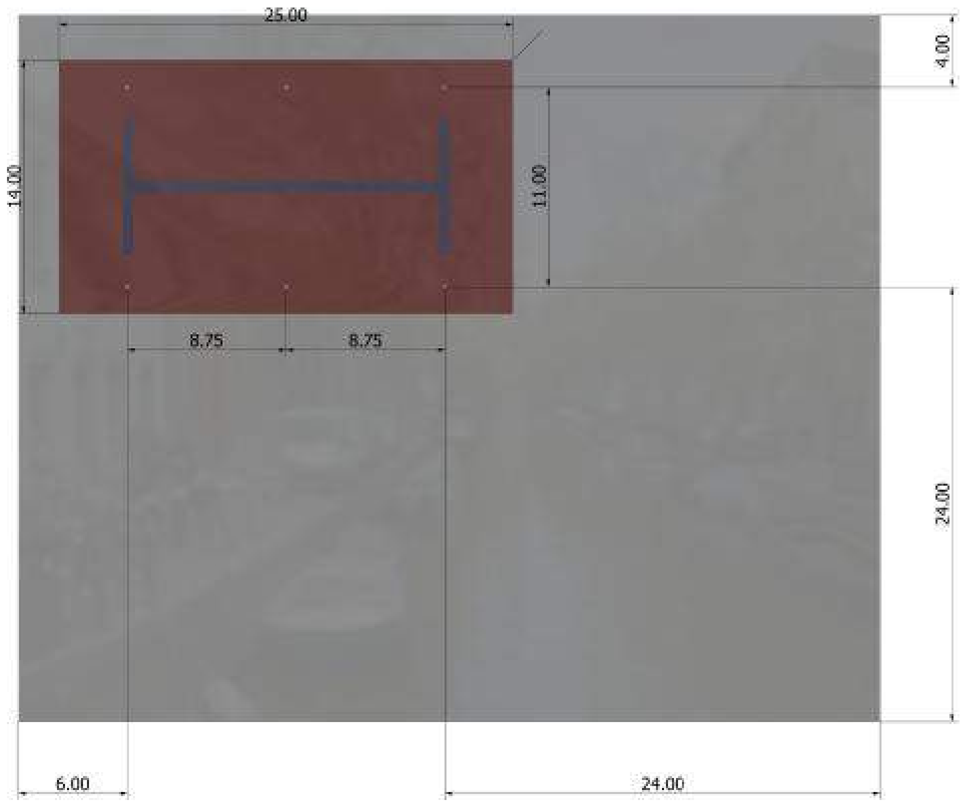
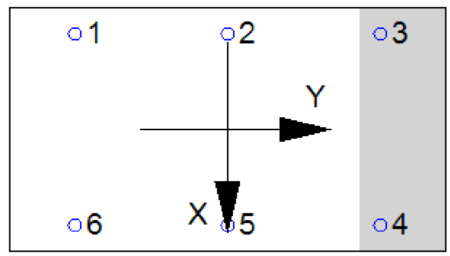


3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb)
1	4788.1	0.0	13114.7	13114.7
2	2221.4	0.0	13114.7	13114.7
3	0.0	0.0	13114.7	13114.7
4	0.0	0.0	13114.7	13114.7
5	2221.4	0.0	13114.7	13114.7
6	4788.1	0.0	13114.7	13114.7
Sum	14019.1	0.0	78688.0	78688.0

Maximum concrete compression strain (‰): 0.09
Maximum concrete compression stress (psi): 407
Resultant tension force (lb): 0
Resultant compression force (lb): 14021
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.00
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 0.00
Eccentricity of resultant shear forces in x-axis, e'_{Vx} (inch): 0.00
Eccentricity of resultant shear forces in y-axis, e'_{Vy} (inch): 0.00

<Figure 3>



4. Steel Strength of Anchor in Tension (Sec. 17.4.1)

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
26950	0.75	20213

6. Pullout Strength of Anchor in Tension (Sec. 17.4.3)

$\phi N_{pn} = \phi \Psi_{c,P} N_p = \phi \Psi_{c,P} 8 A_{brg} f'_c$ (Sec. 17.3.1, Eq. 17.4.3.1 & 17.4.3.4)

$\Psi_{c,P}$	A_{brg} (in^2)	f'_c (psi)	ϕ	ϕN_{pn} (lb)
1.4	0.79	4000	0.70	24618

8. Steel Strength of Anchor in Shear (Sec. 17.5.1)

V_{sa} (lb)	ϕ_{grout}	ϕ	$\phi_{grout}\phi V_{sa}$ (lb)
26950	1.0	0.65	17518

10. Concrete Pryout Strength of Anchor in Shear (Sec. 17.5.3)

$\phi V_{cpg} = \phi k_{cp} N_{cbg} = \phi k_{cp}(A_{Nc} / A_{Nco}) \Psi_{ec,N} \Psi_{ed,N} \Psi_{c,N} \Psi_{cp,N} N_b$ (Sec. 17.3.1 & Eq. 17.5.3.1b)

k_{cp}	A_{Nc} (in^2)	A_{Nco} (in^2)	$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	ϕV_{cpg} (lb)
2.0	629.00	153.14	1.000	0.894	1.250	1.000	12717	0.70	81712

11. Results

Interaction of Tensile and Shear Forces (Sec. 17.6.)

Tension	Factored Load, N_{ua} (lb)	Design Strength, ϕN_n (lb)	Ratio	Status
Steel	**4788**	**20213**	**0.24**	**Pass (Governs)**
Pullout	4788	24618	0.19	Pass

Shear	Factored Load, V_{ua} (lb)	Design Strength, ϕV_n (lb)	Ratio	Status
Steel	13115	17518	0.75	Pass
Pryout	**78688**	**81712**	**0.96**	**Pass (Governs)**

Interaction check	$N_{ua}/\phi N_n$	$V_{ua}/\phi V_n$	Combined Ratio	Permissible	Status
Sec. 17.6..3	0.24	0.96	120.0%	1.2	Pass

3/4"Ø AWS Type A Headed Stud with hef = 4.125 inch meets the selected design criteria.

Base Plate Thickness

Required base plate thickness: 0.600 inch
Warning: input base plate thickness does not meet required base plate thickness.

Max W18 Load = 76.9k
DCR = 76.9/78.7 = 0.977

12. Warnings

- Concrete breakout strength in tension has not been evaluated against applied tension load(s) per designer option. Refer to ACI 318 Section 17.3.2.1 for conditions where calculations of the concrete breakout strength may not be required.

- Concrete breakout strength in shear has not been evaluated against applied shear load(s) per designer option. Refer to ACI 318 Section 17.3.2.1 for conditions where calculations of the concrete breakout strength may not be required.

- Designer must exercise own judgement to determine if this design is suitable.

SIMPSON Strong-Tie

Anchor Designer™
Software
Version 2.9.7376.0

Company:		Date:	9/29/2020
Engineer:		Page:	1/6
Project:			
Address:			
Phone:			
E-mail:			

Calculation For W21 Gravity Anchorage

1.Project information

Customer company:
Customer contact name:
Customer e-mail:
Comment:

Project description:
Location:
Fastening description:

2. Input Data & Anchor Parameters

General
Design method:ACI 318-14
Units: Imperial units

Anchor Information:
Anchor type: Cast-in-place
Material: AWS Type A
Diameter (inch): 0.750
Effective Embedment depth, h_{ef} (inch): 4.125
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 5.63
C_{min} (inch): 1.38
S_{min} (inch): 3.00

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 8.00
State: Uncracked
Compressive strength, f'_c (psi): 4000
$\Psi_{c,V}$: 1.0
Reinforcement condition: B tension, B shear
Supplemental reinforcement: Yes
Reinforcement provided at corners: Yes
Ignore concrete breakout in tension: Yes
Ignore concrete breakout in shear: Yes
Ignore 6do requirement: No
Build-up grout pad: No

Base Plate
Length x Width x Thickness (inch): 12.00 x 25.00 x 0.50
Yield stress: 34084 psi

Profile type/size: W21X50

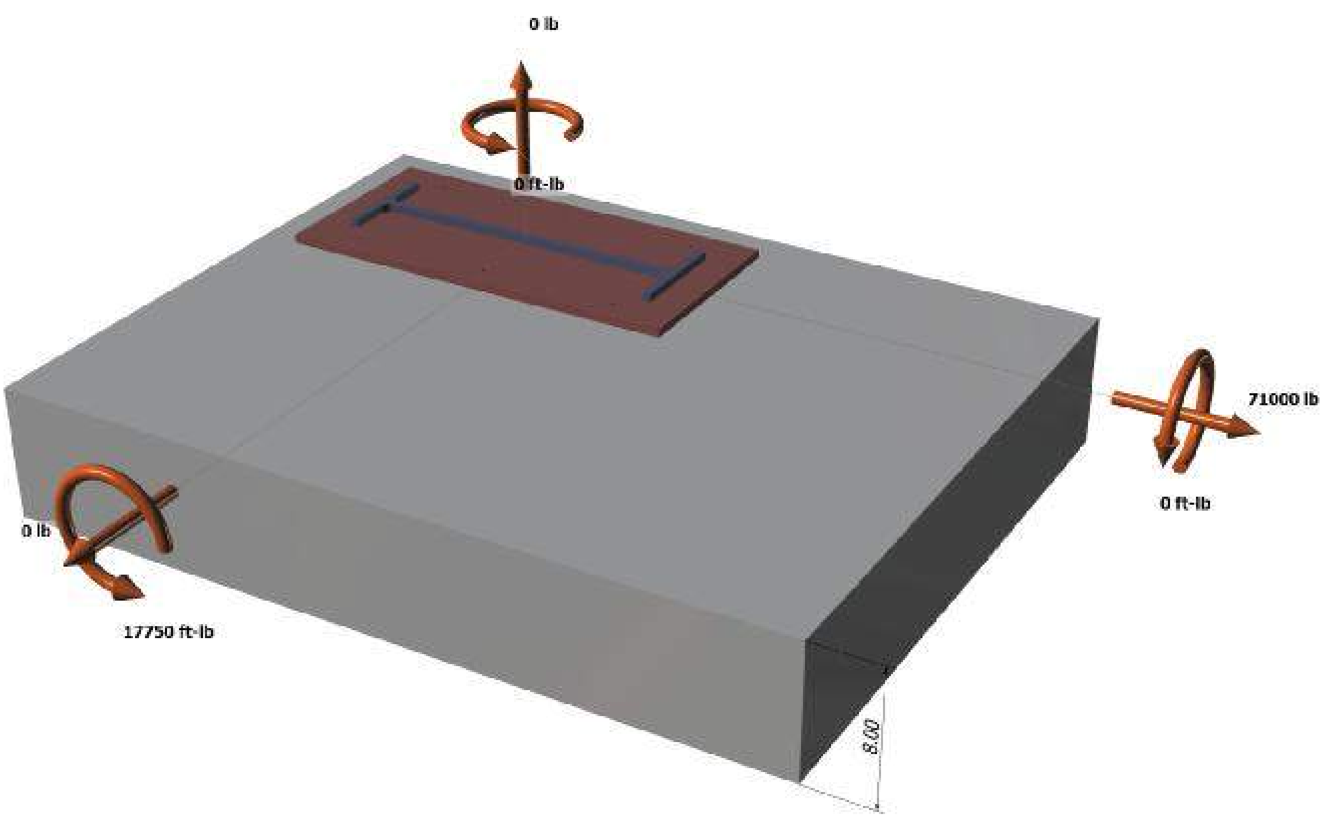
Recommended Anchor
Anchor Name: Headed Stud - 3/4"Ø AWS Type A Headed Stud

Load and Geometry
Load factor source: ACI 318 Section 5.3
Load combination: not set
Seismic design: Yes
Anchors subjected to sustained tension: Not applicable
Ductility section for tension: 17.2.3.4.2 not applicable
Ductility section for shear: 17.2.3.5.2 not applicable
Ω_0 factor: not set
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: No

Strength level loads:

N_{ua} [lb]: 0
V_{uax} [lb]: 0
V_{uay} [lb]: 71000
M_{ux} [ft-lb]: 17750
M_{uy} [ft-lb]: 0
M_{uz} [ft-lb]: 0

<Figure 1>



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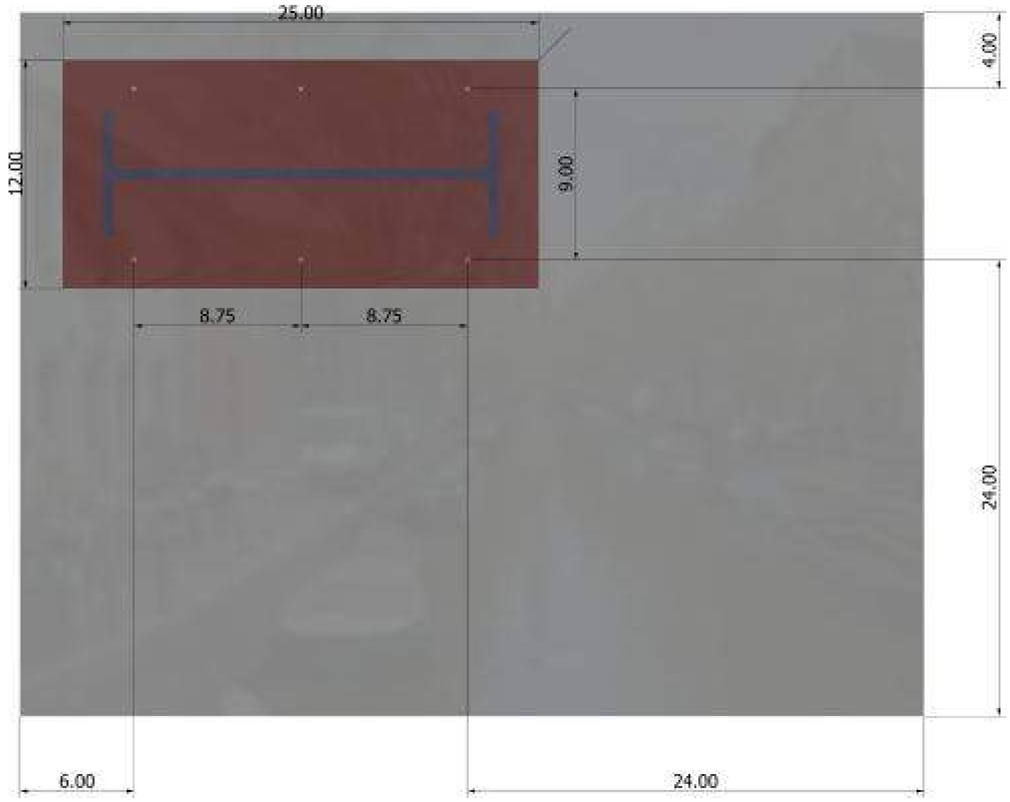
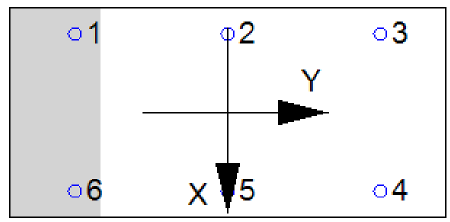


3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb)
1	0.0	0.0	11833.3	11833.3
2	1980.1	0.0	11833.3	11833.3
3	4369.8	0.0	11833.3	11833.3
4	4369.8	0.0	11833.3	11833.3
5	1980.1	0.0	11833.3	11833.3
6	0.0	0.0	11833.3	11833.3
Sum	12699.9	0.0	71000.0	71000.0

Maximum concrete compression strain (‰): 0.09
Maximum concrete compression stress (psi): 403
Resultant tension force (lb): 0
Resultant compression force (lb): 12700
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.00
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 0.00
Eccentricity of resultant shear forces in x-axis, e'_{Vx} (inch): 0.00
Eccentricity of resultant shear forces in y-axis, e'_{Vy} (inch): 0.00

<Figure 3>



4. Steel Strength of Anchor in Tension (Sec. 17.4.1)

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
26950	0.75	20213

6. Pullout Strength of Anchor in Tension (Sec. 17.4.3)

$0.75\phi N_{pn} = 0.75\phi \Psi_{c,P} N_p = 0.75\phi \Psi_{c,P} 8 A_{brg} f'_c$ (Sec. 17.3.1, Eq. 17.4.3.1 & 17.4.3.4)

$\Psi_{c,P}$	A_{brg} (in^2)	f'_c (psi)	ϕ	$0.75\phi N_{pn}$ (lb)
1.4	0.79	4000	0.70	18463

8. Steel Strength of Anchor in Shear (Sec. 17.5.1)

V_{sa} (lb)	ϕ_{grout}	ϕ	$\phi_{grout}\phi V_{sa}$ (lb)
26950	1.0	0.65	17518

10. Concrete Pryout Strength of Anchor in Shear (Sec. 17.5.3)

$\phi V_{cpg} = \phi k_{cp} N_{cbg} = \phi k_{cp}(A_{Nc} / A_{Nco})\, \Psi_{ec,N}\, \Psi_{ed,N}\, \Psi_{c,N}\, \Psi_{cp,N} N_b$ (Sec. 17.3.1 & Eq. 17.5.3.1b)

k_{cp}	A_{Nc} (in^2)	A_{Nco} (in^2)	$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	ϕV_{cpg} (lb)
2.0	569.63	153.14	1.000	0.894	1.250	1.000	12717	0.70	73999

11. Results

Interaction of Tensile and Shear Forces (Sec. 17.6.)

Tension	Factored Load, N_{ua} (lb)	Design Strength, $øN_n$ (lb)	Ratio	Status
Steel	4370	20213	0.22	Pass
Pullout	**4370**	**18463**	**0.24**	**Pass (Governs)**

Shear	Factored Load, V_{ua} (lb)	Design Strength, $øV_n$ (lb)	Ratio	Status
Steel	11833	17518	0.68	Pass
Pryout	**71000**	**73999**	**0.96**	**Pass (Governs)**

Interaction check	$N_{ua}/\phi N_n$	$V_{ua}/\phi V_n$	Combined Ratio	Permissible	Status
Sec. 17.6..3	0.24	0.96	119.6%	1.2	Pass

3/4"Ø AWS Type A Headed Stud with hef = 4.125 inch meets the selected design criteria.

Base Plate Thickness

Required base plate thickness: 0.440 inch

Max W21 Load = 57.6k
DCR = 57.6/71 = 0.811

12. Warnings

- Concrete breakout strength in tension has not been evaluated against applied tension load(s) per designer option. Refer to ACI 318 Section 17.3.2.1 for conditions where calculations of the concrete breakout strength may not be required.

- Concrete breakout strength in shear has not been evaluated against applied shear load(s) per designer option. Refer to ACI 318 Section 17.3.2.1 for conditions where calculations of the concrete breakout strength may not be required.

- Per designer input, the tensile component of the strength-level earthquake force applied to anchors does not exceed 20 percent of the total factored anchor tensile force associated with the same load combination. Therefore the ductility requirements of ACI 318 17.2.3.4.2 for tension need not be satisfied – designer to verify.

- Per designer input, the shear component of the strength-level earthquake force applied to anchors does not exceed 20 percent of the total factored anchor shear force associated with the same load combination. Therefore the ductility requirements of ACI 318 17.2.3.5.2 for shear need not be satisfied – designer to verify.

- Designer must exercise own judgement to determine if this design is suitable.

SIMPSON Strong-Tie

Anchor Designer™
Software
Version 2.9.7376.0

Company:		Date:	9/29/2020
Engineer:		Page:	1/5
Project:			
Address:			
Phone:			
E-mail:			

Calculation For W24
Gravity Anchorage

1.Project information

Customer company:
Customer contact name:
Customer e-mail:
Comment:

Project description:
Location:
Fastening description:

2. Input Data & Anchor Parameters

General
Design method:ACI 318-14
Units: Imperial units

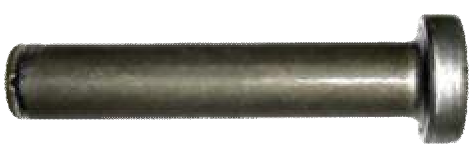
Anchor Information:
Anchor type: Cast-in-place
Material: AWS Type A
Diameter (inch): 1.000
Effective Embedment depth, h_{ef} (inch): 5.500
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 7.25
C_{min} (inch): 1.56
S_{min} (inch): 4.00

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 8.00
State: Uncracked
Compressive strength, f'_c (psi): 4000
$\Psi_{c,V}$: 1.0
Reinforcement condition: B tension, B shear
Supplemental reinforcement: Yes
Reinforcement provided at corners: Yes
Ignore concrete breakout in tension: Yes
Ignore concrete breakout in shear: Yes
Ignore 6do requirement: No
Build-up grout pad: No

Base Plate
Length x Width x Thickness (inch): 12.00 x 38.00 x 0.50
Yield stress: 34084 psi

Profile type/size: W24X62

Recommended Anchor
Anchor Name: Headed Stud - 1"Ø AWS Type A Headed Stud

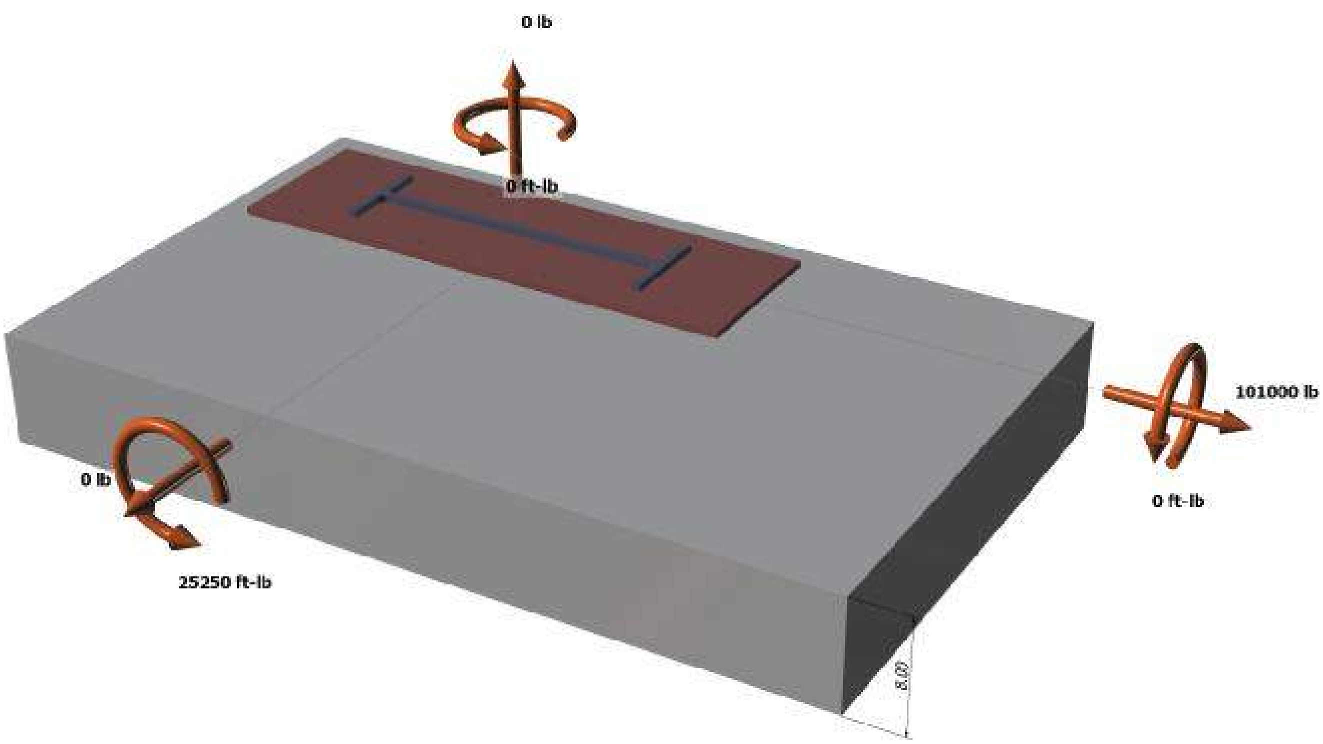


Load and Geometry
Load factor source: ACI 318 Section 5.3
Load combination: not set
Seismic design: Yes
Anchors subjected to sustained tension: Not applicable
Ductility section for tension: 17.2.3.4.2 not applicable
Ductility section for shear: 17.2.3.5.2 not applicable
Ω_0 factor: not set
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: No

Strength level loads:

N_{ua} [lb]: 0
V_{uax} [lb]: 0
V_{uay} [lb]: 101000
M_{ux} [ft-lb]: 25250
M_{uy} [ft-lb]: 0
M_{uz} [ft-lb]: 0

<Figure 1>



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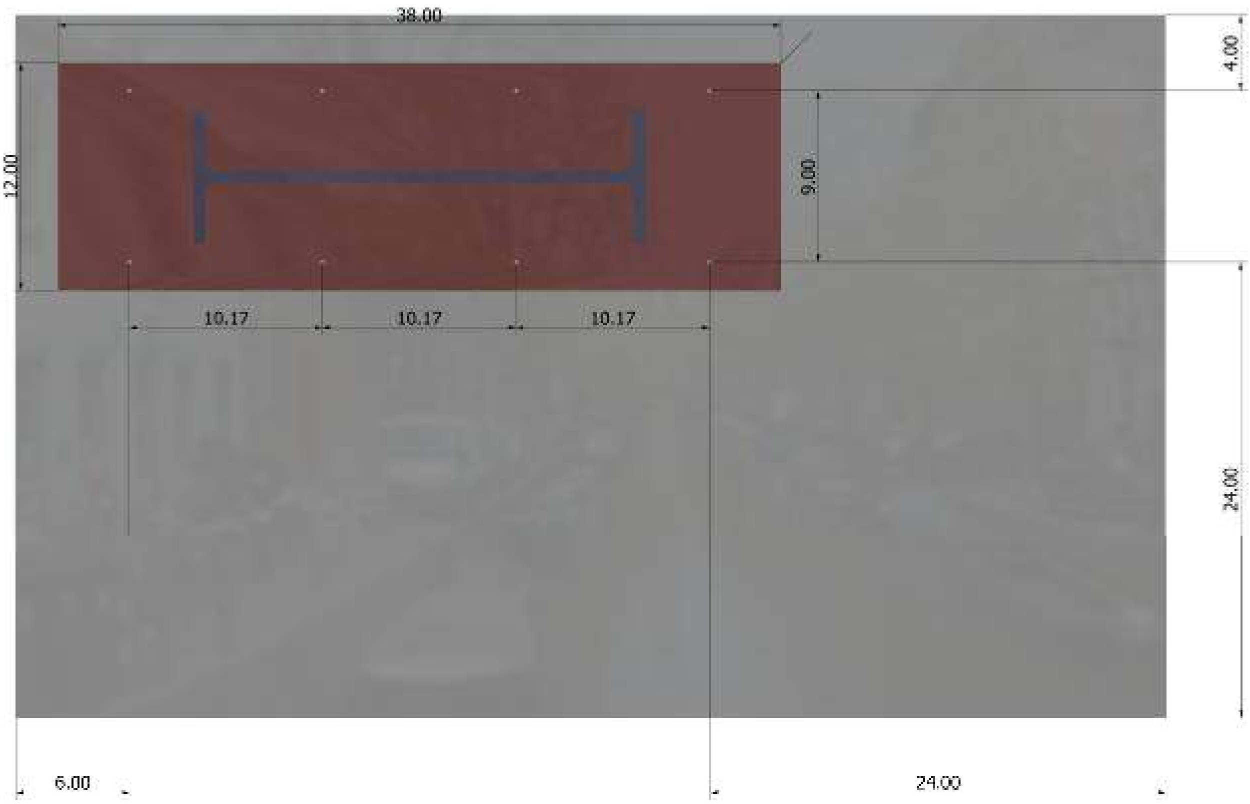
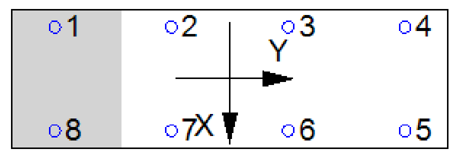


3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb)
1	0.0	0.0	12625.0	12625.0
2	589.2	0.0	12625.0	12625.0
3	1969.1	0.0	12625.0	12625.0
4	3349.1	0.0	12625.0	12625.0
5	3349.1	0.0	12625.0	12625.0
6	1969.1	0.0	12625.0	12625.0
7	589.2	0.0	12625.0	12625.0
8	0.0	0.0	12625.0	12625.0
Sum	11814.8	0.0	101000.0	101000.0

Maximum concrete compression strain (‰): 0.05
Maximum concrete compression stress (psi): 206
Resultant tension force (lb): 0
Resultant compression force (lb): 11814
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.00
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 0.00
Eccentricity of resultant shear forces in x-axis, e'_{Vx} (inch): 0.00
Eccentricity of resultant shear forces in y-axis, e'_{Vy} (inch): 0.00

<Figure 3>



4. Steel Strength of Anchor in Tension (Sec. 17.4.1)

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
47910	0.75	35933

6. Pullout Strength of Anchor in Tension (Sec. 17.4.3)

$0.75\phi N_{pn} = 0.75\phi \Psi_{c,P} N_p = 0.75\phi \Psi_{c,P} 8 A_{brg} f'_c$ (Sec. 17.3.1, Eq. 17.4.3.1 & 17.4.3.4)

$\Psi_{c,P}$	A_{brg} (in^2)	f'_c (psi)	ϕ	$0.75\phi N_{pn}$ (lb)
1.4	1.29	4000	0.70	30317

8. Steel Strength of Anchor in Shear (Sec. 17.5.1)

V_{sa} (lb)	ϕ_{grout}	ϕ	$\phi_{grout}\phi V_{sa}$ (lb)
47910	1.0	0.65	31142

10. Concrete Pryout Strength of Anchor in Shear (Sec. 17.5.3)

$\phi V_{cpg} = \phi k_{cp} N_{cbg} = \phi k_{cp}(A_{Nc} / A_{Nco}) \Psi_{ec,N} \Psi_{ed,N} \Psi_{c,N} \Psi_{cp,N} N_b$ (Sec. 17.3.1 & Eq. 17.5.3.1b)

k_{cp}	A_{Nc} (in^2)	A_{Nco} (in^2)	$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	ϕV_{cpg} (lb)
2.0	951.15	272.25	1.000	0.845	1.250	1.000	19579	0.70	101203

11. Results

Interaction of Tensile and Shear Forces (Sec. 17.6.)

Tension	Factored Load, N_{ua} (lb)	Design Strength, øN_n (lb)	Ratio	Status
Steel	3349	35933	0.09	Pass
Pullout	**3349**	**30317**	**0.11**	**Pass (Governs)**

Shear	Factored Load, V_{ua} (lb)	Design Strength, øV_n (lb)	Ratio	Status
Steel	12625	31142	0.41	Pass
Pryout	**101000**	**101203**	**1.00**	**Pass (Governs)**

Interaction check	$N_{ua}/\phi N_n$	$V_{ua}/\phi V_n$	Combined Ratio	Permissible	Status
Sec. 17.6..2	0.00	1.00	99.8%	1.0	Pass

1"Ø AWS Type A Headed Stud with hef = 5.500 inch meets the selected design criteria.

Max W24 Load = 98.3k
DCR = 98.3/101 = 0.973

12. Warnings

- Concrete breakout strength in tension has not been evaluated against applied tension load(s) per designer option. Refer to ACI 318 Section 17.3.2.1 for conditions where calculations of the concrete breakout strength may not be required.

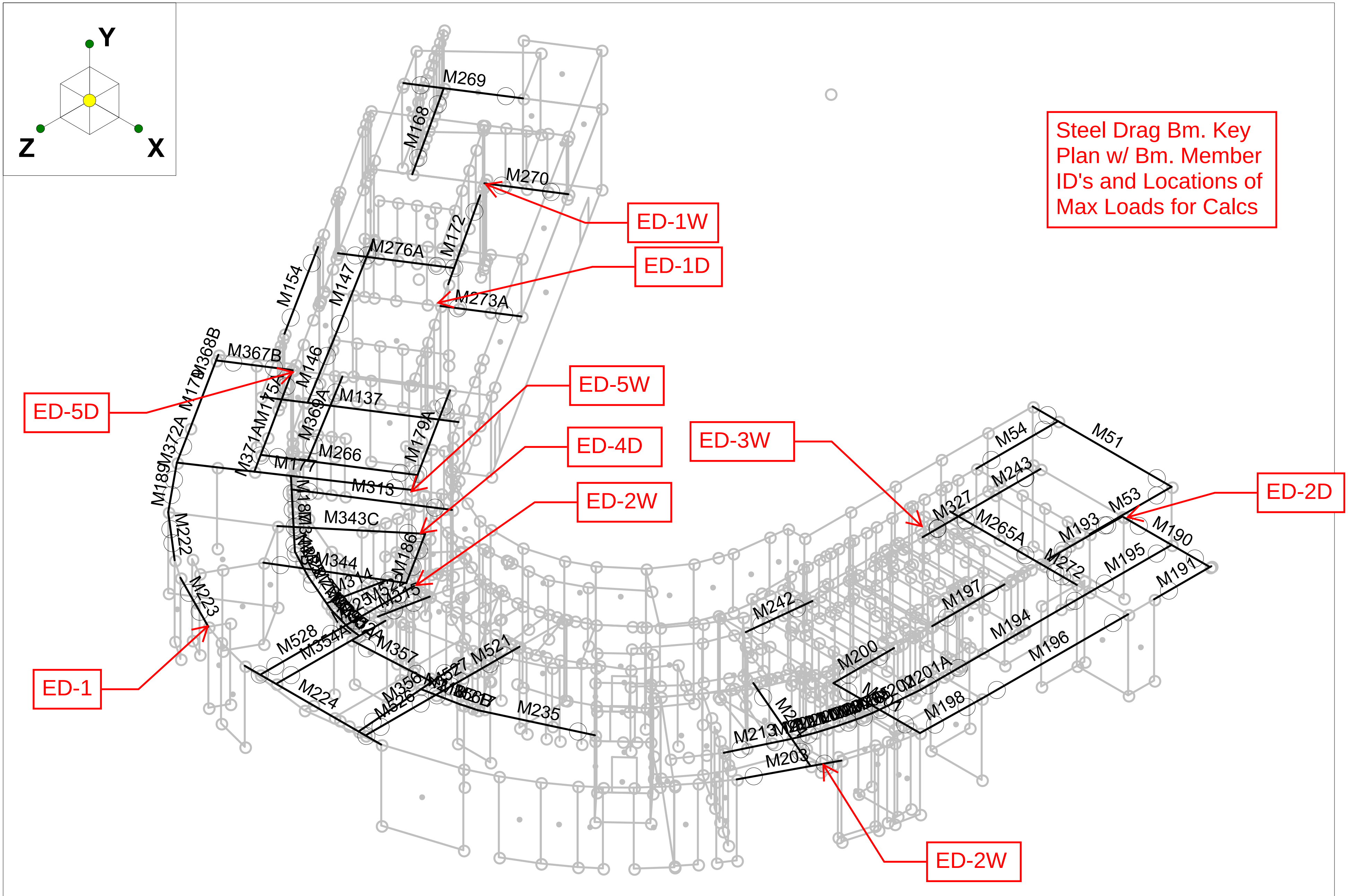
- Concrete breakout strength in shear has not been evaluated against applied shear load(s) per designer option. Refer to ACI 318 Section 17.3.2.1 for conditions where calculations of the concrete breakout strength may not be required.

- Per designer input, the tensile component of the strength-level earthquake force applied to anchors does not exceed 20 percent of the total factored anchor tensile force associated with the same load combination. Therefore the ductility requirements of ACI 318 17.2.3.4.2 for tension need not be satisfied – designer to verify.

- Per designer input, the shear component of the strength-level earthquake force applied to anchors does not exceed 20 percent of the total factored anchor shear force associated with the same load combination. Therefore the ductility requirements of ACI 318 17.2.3.5.2 for shear need not be satisfied – designer to verify.

- Designer must exercise own judgement to determine if this design is suitable.

Appendix B: Steel Drag Beam Calculations



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STEEL DRAG BEAM CONNECTION DESIGNATION

Bm. ID	Bm. Size	Drag Load	Gravity Load	Connection
M193	W14x26	8.8	5.7	ED-1
M195	W14x53	31.8	16	ED-1D
M193	W14x53	15.9	18.1	ED-1
M272	W14x26	68.8	4.2	ED-1W
M197	W14x53	4.4	22.8	ED-1
M197	W14x53	10.6	19.5	ED-1
M198	W14x53	33.5	6.8	ED-1D
M200	W14x53	5.5	29.8	ED-1
M200	W14x53	7.3	25.8	ED-1
M242	W14x53	29	40.9	ED-1W
	W14x30	31.3	0	ED-1D
M223	W14x53	15.1	21.7	ED-1
M223	W14x53	8.4	20.1	ED-1
M222	W14x53	24.3	17.4	ED-1D
M222	W14x53	16.8	16.2	ED-1
M186	W14x53	16.8	20.9	ED-1
M273A	W14x30	44.1	6	ED-1D
M270	W14x30	59.1	45.2	ED-1W
M527	W14x30	83.3	0	ED-1W
M528	W16x36	89.1	0	ED-1W
M190	W16x77	37.5	28.1	ED-2D
	W16x36	57.1	0	ED-2D
M521	W16x36	115.4	0	ED-2W
M522	W16x36	123.4	0	ED-2W
M243	W18x106	60.2	72.3	ED-3W
M203	W18x50	19.7	44.7	ED-2
M203	W18x50	27.5	22.5	ED-2
M269	W18x97	40.7	31.8	ED-3W
M196	W21x93	2.5	16.5	ED-4D
M344	W21x93	45	37.7	ED-4D
M343C	W21x93	45	70	ED-4D
M224	W24x117	12.7	60.6	ED-5D
M224	W24x117	24.8	53	ED-5D
M177	W24x117	125.7	27.9	ED-5W
M175A	W24x117	43.6	52.8	ED-5D
M367B	W24x84	26.3	-53.7	ED-5D

Notes:

- Max Load for Calculation of Conn.

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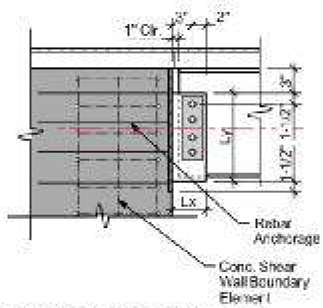
STEEL DRAG BEAM CONNECTION CALCULATION

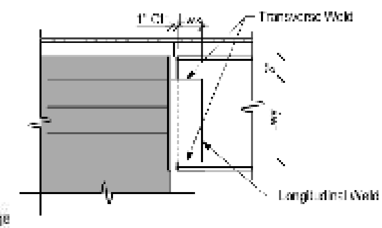
WHAT TYPE OF CONNECTION IS BEING USED? **BOLT: SINGLE ROW** *CONVENTIONAL CONFIGURATION*

MATERIAL AND CONFIGURATION

Embed Plate:	Height =	14	in	
	Width =	8	in	
	t =	1/2	in	
	F_y =	36	ksi	*(A36 Steel)*
	F_U =	58	ksi	*(A36 Steel)*
Shear Plate:	L_X =	5	in	
	L_Y =	9.5	in	
	t =	1/4	in	
	Weld to Embed Plate =	3/16	in	
	F_y =	36	ksi	*(A36 Steel)*
	F_U =	58	ksi	*(A36 Steel)*
Rebar:	Size =	#4		
	# of Bars per Row =	2		
	# of Rows =	3		
	Vertical Spacing =	4.75	in	
	F_y (Rebar)=	60	ksi	*(A706 Steel)*
Bolts:	Ø =	1	in	
	# of Rows =	3		
	Vert. Spacing =	2.45	in	
	Horiz. Spacing =	0	in	
	Fnt (Bolts)=	45	ksi	*(A307 Steel)*
	Fnv (Bolts)=	27	ksi	*(A307 Steel)*
Beam Size:	W14X26			
	tf =	0.42		
	tw =	0.26		
	bf =	5.03		
	d =	13.9		
	Ag =	7.69		



BOLT: SINGLE ROW



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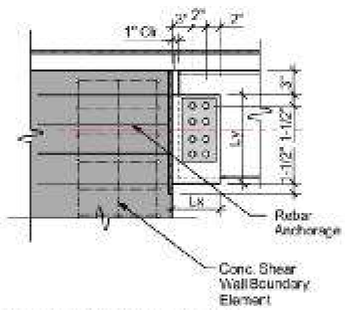


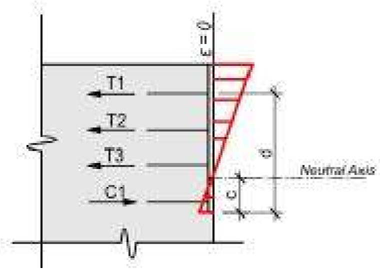
BOLT: DOUBLE ROW

APPLIED LOADS

Drag Load =	**15.1**	kips	*(ASD)*
Gravity Load =	**21.7**	kips	*(LRFD)*

$$\text{Drag Tensile Load} = \frac{\text{Drag Load}}{0.7*(\#\ \text{Of Bars})}$$

Drag Tensile Load =	3.6	kips	*(LRFD/Per Anchor)*
Gravity Tensile Load =	2.4	kips	*(LRFD/Per Anchor)*
Total Tensile Load =	6.0	kips	*(LRFD/Per Anchor)*
Gravity Shear Load =	3.6	kips	*(LRFD/Per Anchor)*



RESULTANT LOAD

$$R_U = \sqrt{(V_U^2 + N_U^2)}$$

R_U = 30.6 kips

Θ = 44.83

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REBAR DESIGN

TENSILE CAPACITY OF SINGLE BAR

A_s = 0.2 in^2

Φ = 0.75 *(ACI 318-19 17.5.3a)*

ΦN_N = 9.0 kips

SHEAR CAPACITY OF SINGLE BAR

A_s = 0.2 in^2

Φ = 0.65 *(ACI 318-19 17.5.3a)*

ΦV_N = 7.8 kips

TENSION & SHEAR INTERACTION

N_{UA} = 6.0 kips V_{UA} = 3.6 kips

$N_{UA}/\Phi N_N$ = 0.67 $V_{UA}/\Phi V_N$ = 0.46

$(N_{UA}/\Phi N_N) + (V_{UA}/\Phi V_N)$ = 1.13 <1.2

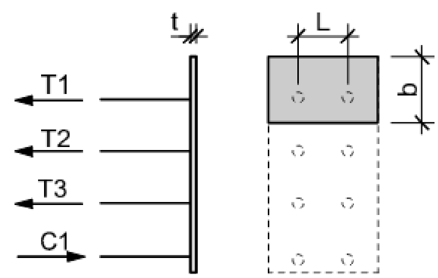
EMBED PLATE DESIGN

EMBED PLATE THICKNESS

L = 2 in

b = 5.38 in

T1 = 12.0 kips *(LRFD)*



$M_{pl} = \frac{T1*L}{8}$ *(AISC 15th Ed. 3-23.16)*

M_{pl} = 3.01 K-in

Flexure Yield: $Z = \frac{b*t^2}{4}$

Z = 0.34 in^3

$\phi M_n = \phi F_y Z$ *(AISC 15th Ed. F11-1)*

Φ = 0.9

ΦM_N = 10.88 K-in

DCR = **0.28** <1

Shear Yield: A_{gv} = 5.5 in^2

$\phi R_n = \phi 0.6 F_y A_{gv}$ *(AISC 15th Ed. J4-3)*

Φ = 0.75

ΦR_n = 89.1 kips

DCR = **0.24** <1

STRENGTH OF WELD

$\mu = 1.0 + 0.5 \sin^{1.5} \theta$ *(AISC 15th Ed. J2-5)*

Θ = 44.83

μ = 1.3

$R_n = (1.392 \text{ kip/in}) Dl\, \mu (2\ sides)$ *(AISC 15th Ed. 8-2a)*

$R_n =$ 102.83 kips

DCR = **0.3** <1

STRENGTH OF BOLTED CONN.

RESULTANT LOAD

$$R_U = \sqrt{(V_U^2 + N_U^2)}$$

R_U = 30.6 kips

Θ = 44.83

e = 1.5 *(AISC 15th Ed. T.10-9)*

C = 2.45

BEAM WEB STRENGTH

Bolt Shear: $\phi r_n = \phi F_n A_b$ *(AISC 15th Ed. J3-1)*

Φ = 0.75

Φr_n = 15.9 kips/bolt

Bolt Bearing Strength: $\phi r_n = \phi 3.0 dt F_U$ *(AISC 15th Ed. J3-6b)*

Φ = 0.75

Φr_n = 37.29 kips/bolt

Bolt Tearout Strength: $\phi r_n = \phi 1.5 l_c t F_U$ *(AISC 15th Ed. J3-6d)*

Φ = 0.75

Φr_n = 26.8 kips/bolt

$Governing\ \phi r_n =$ 15.9 kips

$\phi R_n = C \phi r_n$

ΦR_n = 38.89 kips

DCR = **0.79** <1

SHEAR PLATE STRENGTH

Bolt Bearing Strength: $\phi r_n = \phi 3.0 dt F_U$ *(AISC 15th Ed. J3-6b)*

Φ = 0.75

Φr_n = 32.63 kips/bolt

Bolt Tearout Strength: $\phi r_n = \phi 1.5 l_c t F_U$ *(AISC 15th Ed. J3-6d)*

Φ = 0.75

Φr_n = 15.29 kips/bolt

$Governing\ \phi r_n =$ 15.29 kips

$\phi R_n = C \phi r_n$

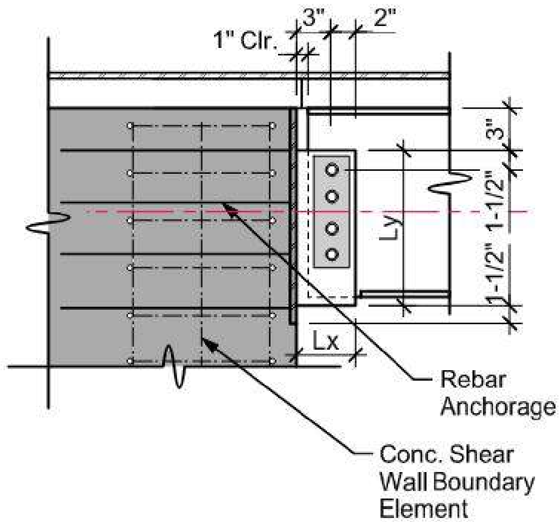
ΦR_n = 37.39 kips

DCR = **0.82** <1

PLATE CHECKS

Maximum Plate Thick: *(Conventional)* $t_{MAX} = (D_{BOLT}/2) + (1/16)$ *(AISC 15th Ed. T.10-9)*

t = 0.56 in



BOLT: SINGLE ROW

10/08/2020

TAYLOR & SYFAN CONSULTING ENGINEERS, INC

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Project: **18412 – Angelo View Revisions – Uberion**

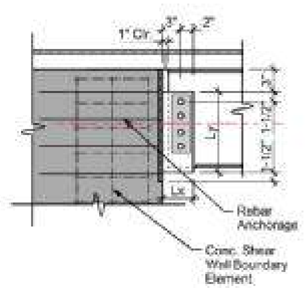
STEEL DRAG BEAM CONNECTION CALCULATION

(2016 CBC Section 16__)

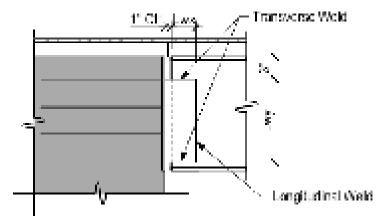
WHAT TYPE OF CONNECTION IS BEING USED? **BOLT:DOUBLE ROW** *EXTENDED CONFIGURATION*

MATERIAL AND CONFIGURATION

Embed Plate:	Height =	14	in	
	Width =	8	in	
	t =	1/2	in	
	F_y =	36	ksi	*(A36 Steel)*
	F_U =	58	ksi	*(A36 Steel)*
Shear Plate:	L_X =	5	in	
	L_Y =	9.5	in	
	t =	1/4	in	
	Weld to Embed Plate =	3/16	in	
	F_y =	36	ksi	*(A36 Steel)*
	F_U =	58	ksi	*(A36 Steel)*
Rebar:	Size =	#4		
	# of Bars per Row =	2		
	# of Rows =	4		
	Vertical Spacing =	3.17	in	
	F_y (Rebar)=	60	ksi	*(A706 Steel)*
Bolts:	Ø =	7/8	in	
	# of Rows =	3		
	Vert. Spacing =	2.4	in	
	Horiz. Spacing =	3	in	
	Fnt (Bolts)=	45	ksi	*(A307 Steel)*
	Fnv (Bolts)=	27	ksi	*(A307 Steel)*
Beam Size:	W14X30			
	tf = 0.39			
	tw = 0.27			
	bf = 6.73			
	d = 13.8			
	Ag = 8.85			



BOLT: SINGLE ROW



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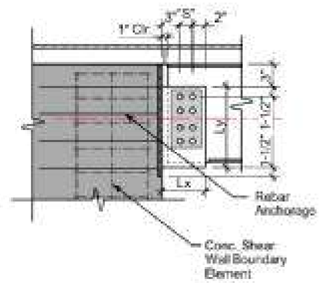


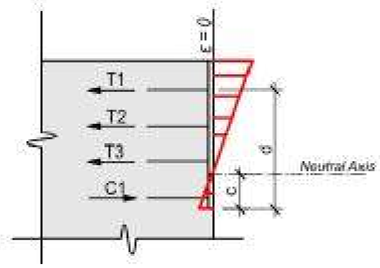
BOLT: DOUBLE ROW

APPLIED LOADS

Drag Load =	**44.1** kips	*(ASD)*
Gravity Load =	**6.0** kips	*(LRFD)*

$$\text{Drag Tensile Load} = \frac{\text{Drag Load}}{0.7 * (\text{\# Of Bars})}$$

Drag Tensile Load =	7.9 kips	*(LRFD/Per Anchor)*
Gravity Tensile Load =	0.8 kips	*(LRFD/Per Anchor)*
Total Tensile Load =	8.7 kips	*(LRFD/Per Anchor)*
Gravity Shear Load =	0.8 kips	*(LRFD/Per Anchor)*



RESULTANT LOAD

$$R_U = \sqrt{(V_U^2 + N_U^2)}$$

R_U = 63.29 kips

Θ = 84.56

REBAR DESIGN

TENSILE CAPACITY OF SINGLE BAR

A_s = 0.2 in^2

Φ = 0.75 *(ACI 318-19 17.5.3a)*

$ΦN_N$ = 9.0 kips

SHEAR CAPACITY OF SINGLE BAR

A_s = 0.2 in^2

Φ = 0.65 *(ACI 318-19 17.5.3a)*

$ΦV_N$ = 7.8 kips

TENSION & SHEAR INTERACTION

N_{UA} = 8.7 kips V_{UA} = 0.8 kips

$N_{UA}/ΦN_N$ = 0.97 $V_{UA}/ΦV_N$ = 0.1

$(N_{UA}/ΦN_N) + (V_{UA}/ΦV_N)$ = **1.06** <1.2

EMBED PLATE DESIGN

EMBED PLATE THICKNESS

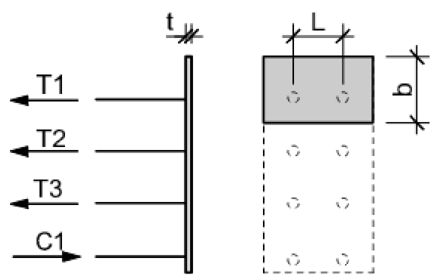
L = 2 in

b = 4.58 in

T1 = 17.4 kips *(LRFD)*

$M_{pl} = \frac{T1 \cdot L}{8}$ *(AISC 15th Ed. 3-23.16)*

M_{pl} = 4.34 K-in



Flexure Yield:

$Z = \frac{b \cdot t^2}{4}$

Z = 0.29 in^3

$\phi M_n = \phi F_y Z$ *(AISC 15th Ed. F11-1)*

Φ = 0.9

$ΦM_N$ = 9.28 K-in

DCR = **0.47** <1

Shear Yield:

A_{gv} = 5.5 in^2

$\phi R_n = \phi 0.6 F_y A_{gv}$ *(AISC 15th Ed. J4-3)*

Φ = 0.75

$Φ R_n$ = 89.1 kips

DCR = **0.07** <1

STRENGTH OF WELD

$\mu = 1.0 + 0.5 \sin^{1.5}\theta$ *(AISC 15th Ed. J2-5)*

Θ = 84.56

μ = 1.5

$R_n = (1.392\,\text{kip/in})\, Dl\, \mu\, (2\, sides)$ *(AISC 15th Ed. 8-2a)*

$R_n =$ 118.75 kips

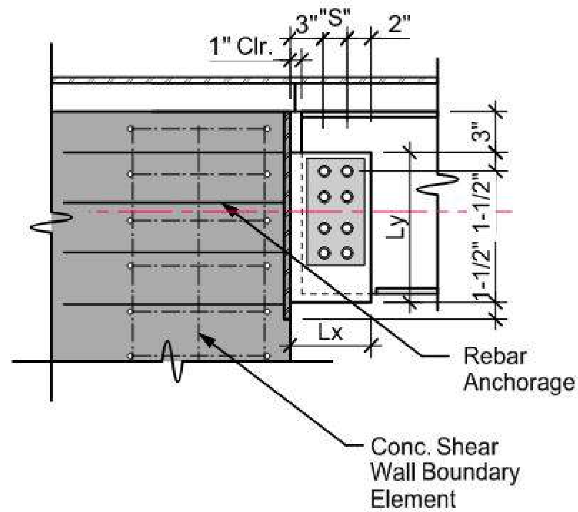
DCR = **0.53** <1

STRENGTH OF BOLTED CONN.

RESULTANT LOAD

$$R_U = \sqrt{(V_U^2 + N_U^2)}$$

R_U = 63.29 kips
Θ = 84.56
e = 4.5 *(AISC 15th Ed. T.10-9)*
C = 5.28



BOLT: DOUBLE ROW

BEAM WEB STRENGTH

Bolt Shear: $\phi r_n = \phi F_n A_b$ *(AISC 15th Ed. J3-1)*
Φ = 0.75
Φr_n = 12.18 kips/bolt

Bolt Bearing Strength: $\phi r_n = \phi 3.0 dt F_U$ *(AISC 15th Ed. J3-6b)*
Φ = 0.75
Φr_n = 34.55 kips/bolt

Bolt Tearout Strength: $\phi r_n = \phi 1.5 l_c t F_U$ *(AISC 15th Ed. J3-6d)*
Φ = 0.75
Φr_n = 29.62 kips/bolt

$Governing\ \phi r_n =$ 12.18 kips
$\phi R_n = C \phi r_n$
ΦR_n = 64.23 kips

DCR = **0.99** <1

SHEAR PLATE STRENGTH

Bolt Bearing Strength: $\phi r_n = \phi 3.0 dt F_U$ *(AISC 15th Ed. J3-6b)*
Φ = 0.75
Φr_n = 28.55 kips/bolt

Bolt Tearout Strength: $\phi r_n = \phi 1.5 l_c t F_U$ *(AISC 15th Ed. J3-6d)*
Φ = 0.75
Φr_n = 24.47 kips/bolt

$Governing\ \phi r_n =$ 12.18 kips
$\phi R_n = C \phi r_n$
ΦR_n = 64.23 kips

DCR = **0.99** <1

BEAM CHECKS

Shear Yielding: A_{gv} = 3.73 in²
$\phi R_n = \phi 0.6 F_y A_{gv}$ *(AISC 15th Ed. J4-3)*

$\phi R_n = \phi 0.6 F_y A_{gv}$

Φ = 1

Φ R_n = 111.78 kips

DCR = **0.05** <1

Tensile Yielding: A_g = 8.85 in^2

$\phi R_n = \phi F_y A_g$ *(AISC 15th Ed. J4-1)*

Φ = 0.9

Φ R_n = 398.25 kips

DCR = **0.16** <1

Tensile Rupture: A_n = 8.09 in^2

U = 1 - (x-bar/l)

$$\bar{x} = \frac{2b_f^2 t_f + t_w^2 (d - 2t_f)}{8b_f t_f + 4t_w (d - 2t_f)}$$

x_bar = 1.03 in

U = 0.66

$\phi R_n = \phi F_u A_e$ *(AISC 15th Ed. J4-2)*

Φ = 0.75

Φ R_n = 259.07 kips

DCR = **0.24** <1

Block Shear Rupture: A_{gv} = 2.7 in^2

A_{nv} = 2.32 in^2

A_{nt} = 1.28 in^2

U_{bs} = 1

Φ = 0.75

$\phi R_n = \phi 0.6 F_u A_{nv} + U_{bs} F_u A_{nt} \leq \phi 0.6 F_y A_{gv} + U_{bs} F_u A_{nt}$ *(AISC 15th Ed. J4-5)*

$\phi 0.6 F_y A_{gv} + U_{bs} F_u A_{nt} =$ 144.11 kips

$\phi 0.6 F_u A_{nv} + U_{bs} F_u A_{nt} =$ 151.23 kips

$\phi R_n =$ 144.11 kips

DCR = **0.31** <1

PLATE CHECKS

Maximum Plate Thick: $t\,max. = \frac{6 M_{MAX}}{F_y T}$ *(AISC 15th Ed. 10-3)*

$M_{MAX} = \frac{F_{NV}}{0.9} (A_b C')$ *(AISC 15th Ed. 10-4)*

C' = 15.8

M_{MAX} = 285.03 k-in

t = 0.3 in

Flexure Yield: $\phi M_n = \phi F_y Z$ *(AISC 15th Ed. F11-1)*

$Z = \frac{t D^2}{4}$

Z = 5.64 in^3

Φ = 0.9

$\phi M_n =$ 182.76 k-in

$\phi R_n =$ 40.61 kips

DCR = **0.15** <1

Lateral-Torsional Buckling:

$\frac{0.08E}{F_y}$ = 64.44

$\frac{1.9E}{F_y}$ = 1530.56

$\frac{L_b d}{t^2}$ = 456

$\frac{0.08E}{F_y} < \frac{L_b d}{t^2} < \frac{1.9E}{F_y}$ So use AISC 15th Ed. F11-2b

$$M_N = C_b[1.52 - 0.274(\frac{L_b d}{t^2})\frac{F_y}{E}]M_y$$ *(AISC 15th Ed. F11-2)*

$$C_b = \frac{12.5M_{max}}{2.5M_{max} + 3M_A + 4M_B + 3M_C}$$ *(AISC 15th Ed. F1-1)*

M_y = 135.38 kip-in
C_b = 1.67
Φ = 0.9
$\phi M_n =$ 277.16 kip-in
$\phi R_n =$ 61.59 kips

DCR = **0.1** <1

Shear Yielding:

A_{gv} = 2.38 in²
$\phi R_n = \phi 0.6 F_y A_{gv}$ *(AISC 15th Ed. J4-3)*
Φ = 1
$\phi R_n =$ 51.3 kips

DCR = **0.12** <1

Tensile Yielding:

A_g = 2.38 in²
$\phi R_n = \phi F_y A_g$ *(AISC 15th Ed. J4-1)*
Φ = 0.9
$\phi R_n =$ 76.95 kips

DCR = **0.82** <1

Interaction of Axial, Flexure and Shear Yielding in Plate:

N_U = 63.0 kips
V_U = 6.0 kips
$\phi R_{np} =$ 76.95 kips
$\phi R_{nv} =$ 51.3 kips
$\phi M_n =$ 182.76 kip-in

$\frac{N_u}{\Phi R_{np}}$ = 0.82 >0.2 So use AISC 15th Ed. H1-1a

$$[\frac{N_u}{\Phi R_{np}} + \frac{8}{9}(\frac{V_u a}{\Phi M_n})]^2 + [\frac{V_u}{\Phi R_{nv}}]^2 =$$ **0.92** <1 *(AISC 15th Ed. H1-1a)*

$$\left(\frac{N_u}{2\phi R_{np}} + \frac{V_u a}{\phi M_n}\right)^2 + \left(\frac{V_u}{\phi R_{nv}}\right)^2 =$$ **N/A** >1 *(AISC 15th Ed. H1-1b)*

Flexure Rupture:

$$Z_{net} = \frac{tl^2}{4} - \frac{t}{4}\left[(d_h + 1/16 \text{ in.})(s)(n^2 - 1) + (d_h + 1/16 \text{ in.})^2\right]$$ *(AISC 15th Ed. 9-4)*

$\phi M_n = \phi F_u Z_{net}$

Z_{net} = 4.46 in³
Φ = 0.75
$\phi M_n =$

$\phi M_n =$ 194.04 k-in
$\phi R_n =$ 43.12

DCR = **0.14** <1

Shear Rupture: A_{nv} = 1.67 in²
$\phi R_n = \phi 0.6 F_u A_{nv}$ *(AISC 15th Ed. J4-4)*
Φ = 0.75
$\phi R_n =$ 43.64 kips

DCR = **0.14** <1

Tensile Rupture: A_{nt} = 1.67 in²
U = 1
$\phi R_n = \phi F_u A_e$ *(AISC 15th Ed. J4-2)*
Φ = 0.75
$\phi R_n =$ 72.73

DCR = **0.08** <1

Interaction of Axial, Flexure and Shear Rupture in Plate:
N_U = 63.0 kips
V_U = 6.0 kips
$\phi R_{np} =$ 72.73 kips
$\phi R_{nv} =$ 43.64 kips
$\phi M_n =$ 194.04 kip-in
$\frac{N_u}{\Phi R_{np}} =$ 0.87 >0.2 So use AISC 15th Ed. H1-1a

$$\left[\frac{N_u}{\Phi R_{np}} + \frac{8}{9}\left(\frac{V_u a}{\Phi M_n}\right)\right]^2 + \left[\frac{V_u}{\Phi R_{nv}}\right]^2 = \textbf{1} < 1$$ *(AISC 15th Ed. H1-1a)*

$$\left(\frac{N_u}{2\phi R_{np}} + \frac{V_u a}{\phi M_n}\right)^2 + \left(\frac{V_u}{\phi R_{nv}}\right)^2 = \textbf{N/A} > 1$$ *(AISC 15th Ed. H1-1b)*

Block Shear Rupture (Beam Shear Direction):
A_{gv} = 2 in²
A_{nv} = 1.45 in²
A_{nt} = 0.8 in²
U_{bs} = 0.5
Φ = 0.75
$\phi R_n = \phi 0.6 F_u A_{nv} + U_{bs} F_u A_{nt} \leq \phi 0.6 F_y A_{gv} + U_{bs} F_u A_{nt}$ *(AISC 15th Ed. J4-5)*
$\phi 0.6 F_y A_{gv} + U_{bs} F_u A_{nt} =$ 55.51 kips
$\phi 0.6 F_u A_{nv} + U_{bs} F_u A_{nt} =$ 61.04 kips
$\phi R_n =$ 55.51 kips

DCR = **0.11** <1

Block Shear Rupture (Beam Axial Direction L Shape):
A_{gv} = 1.13 in²
A_{nv} = 0.8 in²
A_{nt} = 1.45 in²
U_{bs} = 1
Φ = 0.75
$\phi R_n = \phi 0.6 F_u A_{nv} + U_{bs} F_u A_{nt} \leq \phi 0.6 F_y A_{gv} + U_{bs} F_u A_{nt}$ *(AISC 15th Ed. J4-5)*
$\phi 0.6 F_y A_{gv} + U_{bs} F_u A_{nt} =$ 102.51 kips
$\phi 0.6 F_u A_{nv} + U_{bs} F_u A_{nt} =$ 105.08 kips
$\phi R_n =$ 102.51 kips

DCR = **0.43** <1

Block Shear Rupture (Beam Axial Direction U Shape):
A_{gv} = 2.25 in²
A_{nv} = 1.59 in²
A_{nt} = 1.19 in²
U_{bs} = 1

Φ = 0.75

$$\phi R_n = \phi 0.6 F_u A_{nv} + U_{bs} F_u A_{nt} \leq \phi 0.6 F_y A_{gv} + U_{bs} F_u A_{nt}$$ *(AISC 15th Ed. J4-5)*

$\phi 0.6 F_y A_{gv} + U_{bs} F_u A_{nt} =$ 105.33 kips

$\phi 0.6 F_u A_{nv} + U_{bs} F_u A_{nt} =$ 110.47 kips

$\phi R_n =$ 105.33 kips

DCR = **0.42** <1

Block Shear Rupture (Comb. Axial & Shear U Shape):	V_U =	6.0 kips
	N_U =	44.1 kips
	ΦR_{bsv} =	55.51 kips
	ΦR_{bsn} =	102.51 kips

$$\left(\frac{V_u}{\phi R_{bsv}}\right)^2 + \left(\frac{N_u}{\phi R_{bsn}}\right)^2 = $$ **0.2** <1

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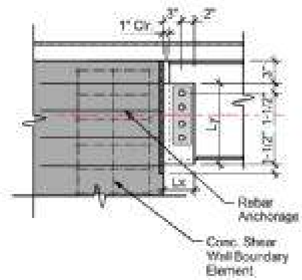
STEEL DRAG BEAM CONNECTION CALCULATION

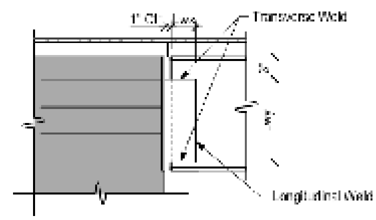
WHAT TYPE OF CONNECTION IS BEING USED? **WELDED**

MATERIAL AND CONFIGURATION

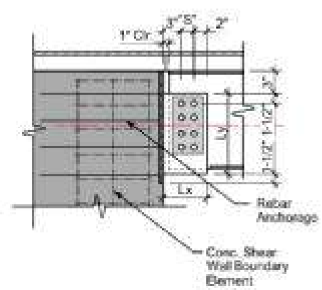
Embed Plate:	Height =	14 in	
	Width =	8 in	
	t =	5/8 in	
	F_y =	36 ksi	*(A36 Steel)*
	F_U =	58 ksi	*(A36 Steel)*
Shear Plate:	L_X =	5 in	
	L_Y =	9.5 in	
	t =	5/8 in	
	Weld to Embed Plate =	3/16 in	
	F_y =	36 ksi	*(A36 Steel)*
	F_U =	58 ksi	*(A36 Steel)*
Rebar:	Size =	#7	
	# of Bars per Row =	2	
	# of Rows =	4	
	Vertical Spacing =	3.17 in	
	F_y (Rebar)=	60 ksi	*(A706 Steel)*
Welds:	D =	5/16 in	
	Lwy	9.5 in	
	Lwx	4 in	



BOLT: SINGLE ROW



WELDED



BOLT: DOUBLE ROW

Beam Size:

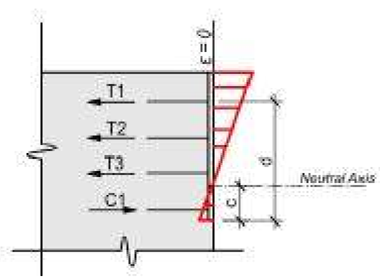
W14X30
tf = 0.39
tw = 0.27
bf = 6.73
d = 13.8
Ag = 8.85

APPLIED LOADS

Drag Load =	**59.1** kips	*(ASD)*
Gravity Load =	**45.2** kips	*(LRFD)*

$$\text{Drag Tensile Load} = \frac{\text{Drag Load}}{0.7*(\#\ \text{Of Bars})}$$

Drag Tensile Load =	10.6 kips	*(LRFD/Per Anchor)*
Gravity Tensile Load =	5.2 kips	*(LRFD/Per Anchor)*
Total Tensile Load =	15.8 kips	*(LRFD/Per Anchor)*
Gravity Shear Load =	5.7 kips	*(LRFD/Per Anchor)*



RESULTANT LOAD

$$R_U = \sqrt{(V_U^2 + N_U^2)}$$

R_U = 95.77 kips

Θ = 61.84

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REBAR DESIGN

TENSILE CAPACITY OF SINGLE BAR

$A_s =$ 0.6 in^2

$\Phi =$ 0.75 *(ACI 318-19 17.5.3a)*

$\Phi N_N =$ 27.0 kips

SHEAR CAPACITY OF SINGLE BAR

$A_s =$ 0.6 in^2

$\Phi =$ 0.65 *(ACI 318-19 17.5.3a)*

$\Phi V_N =$ 23.4 kips

TENSION & SHEAR INTERACTION

$N_{UA} =$ 15.8 kips $V_{UA} =$ 5.7 kips

$N_{UA}/\Phi N_N =$ 0.59 $V_{UA}/\Phi V_N =$ 0.24

$(N_{UA}/\Phi N_N) + (V_{UA}/\Phi V_N) =$ 0.83 <1.2

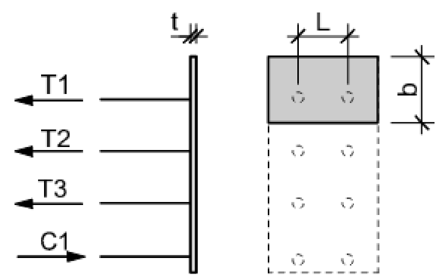
EMBED PLATE DESIGN

EMBED PLATE THICKNESS

L = 2 in

b = 4.58 in

T1 = 31.6 kips *(LRFD)*



$M_{pl} = \frac{T1 \cdot L}{8}$ *(AISC 15th Ed. 3-23.16)*

$M_{pl} =$ 7.9 K-in

Flexure Yield:

$Z = \frac{b \cdot t^2}{4}$

Z = 0.45 in^3

$\phi M_n = \phi F_y Z$ *(AISC 15th Ed. F11-1)*

$\Phi =$ 0.9

$\Phi M_N =$ 14.5 K-in

DCR = **0.54** <1

Shear Yield:

$A_{gv} =$ 6.88 in^2

$\phi R_n = \phi 0.6 F_y A_{gv}$ *(AISC 15th Ed. J4-3)*

$\Phi =$ 0.75

$\Phi R_n =$ 111.38 kips

DCR = **0.41** <1

STRENGTH OF WELD

$\mu = 1.0 + 0.5 \sin^{1.5}\theta$ *(AISC 15th Ed. J2-5)*

$\Theta =$ 61.84

$\mu =$ 1.41

$R_n = (1.392\ \text{kip/in})Dl\,\mu(2\ sides)$ *(AISC 15th Ed. 8-2a)*

$R_n =$ 112.18 kips

DCR = **0.85** <1

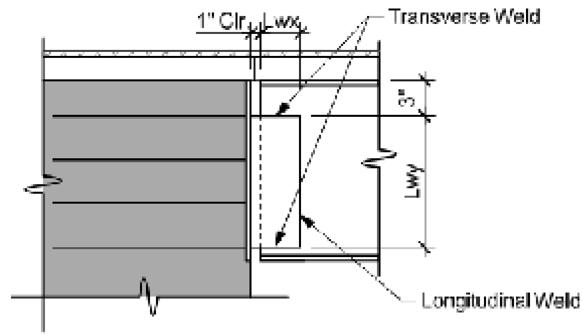
STRENGTH OF WELDED CONN.

RESULTANT LOAD

$$R_U = \sqrt{(V_U^2 + N_U^2)}$$

R_U = 95.77 kips

Θ = 61.84



WELDED

WELD STRENGTH

$$\frac{kl}{l} = \frac{4}{9.5} = k = 0.42$$

x = 0.10

xl = 0.92 in

e_x = 4.08 in

$$\frac{e_x}{l} = \quad a = 0.43$$

C = 3.13

$$\phi R_n = \phi C C_1 D l$$ *(AISC 15th Ed. 8-21)*

Φ = 0.75

C_1 = 1 *(AISC 15th Ed. T.8-3)*

Gravity Load: $\phi R_n =$ 111.68 kips

DCR = **0.4** <1

Drag Load: 94.04

DCR = **0.9** <1

BEAM CHECKS

Shear Rupture of Beam Web: $t_{min} = \frac{3.09\,D}{F_U}$ *(AISC 15th Ed. 8-21)*

t_{MIN} = 0.01 in

DCR = **0.06** <1

Shear Yielding: A_{gv} = 3.73 in^2

$$\phi R_n = \phi\, 0.6\, F_y A_{gv}$$ *(AISC 15th Ed. J4-3)*

Φ = 1

Φ R_n = 111.78 kips

DCR = **0.4** <1

Tensile Yielding: A_g = 8.85 in^2

$$\phi R_n = \phi F_y A_g$$ *(AISC 15th Ed. J4-1)*

Φ = 0.9

Φ R_n = 398.25 kips

DCR = **0.21** <1

Tensile Rupture: A_n = 8.85 in²

U = 1 - (x-bar/l)

$$\bar{x} = \frac{2b_f{}^2 t_f + t_w{}^2(d-2t_f)}{8b_f t_f + 4t_w(d-2t_f)}$$

x_bar = 1.03 in

U = 0.74

$\phi R_n = \phi F_u A_e$ (AISC 15th Ed. J4-2)

Φ = 0.75

Φ R_n = 320.4 kips

DCR = **0.26** <1

Block Shear Rupture:

A_{gv} = 2.16 in²

A_{nv} = 2.16 in²

A_{nt} = 2.57 in²

U_{bs} = 1

Φ = 0.75

$\phi R_n = \phi 0.6 F_u A_{nv} + U_{bs} F_u A_{nt} \leq \phi 0.6 F_y A_{gv} + U_{bs} F_u A_{nt}$ (AISC 15th Ed. J4-5)

$\phi 0.6 F_y A_{gv} + U_{bs} F_u A_{nt} =$ 215.33 kips

$\phi 0.6 F_u A_{nv} + U_{bs} F_u A_{nt} =$ 229.91 kips

$\phi R_n =$ 215.33 kips

DCR = **0.27** <1

PLATE CHECKS

Shear Rupture of Plate: $t_{min} = \frac{3.09\,D}{F_U}$ (AISC 15th Ed. 8-21)

t_{MIN} = 0.02 in

DCR = **0.03** <1

Flexure Yield: $\phi M_n = \phi F_y Z$ (AISC 15th Ed. F11-1)

$Z = \frac{t*D^2}{4}$

Z = 14.1 in³

Φ = 0.9

$\phi M_n =$ 456.89 k-in

$\phi R_n =$ 111.91 kips

DCR = **0.4** <1

Lateral-Torsional Buckling:

$\frac{0.08E}{F_y} =$ 64.44

$\frac{1.9E}{F_y} =$ 1530.56

$\frac{L_b d}{t^2} =$ 97.28

$\frac{0.08E}{F_y} < \frac{L_b d}{t^2} < \frac{1.9E}{F_y}$ So use AISC 15th Ed. F11-2b

$$M_N = C_b[1.52\text{-}0.274(\frac{L_b d}{t^2})\frac{F_y}{E}]M_y$$ *(AISC 15th Ed. F11-2)*

$$C_b = \frac{12.5M_{max}}{2.5\,M_{max} + 3M_A + 4M_B + 3M_C}$$ *(AISC 15th Ed. F1-1)*

M_y = 507.66 kip-in
C_b = 1.67
Φ = 0.9
$\phi M_n =$ 1132.26 kip-in
$\phi R_n =$ 277.34 kips

DCR = **0.16** <1

Shear Yielding:

A_{gv} = 5.94 in^2
$\phi R_n = \phi 0.6 F_y A_{gv}$ *(AISC 15th Ed. J4-3)*
Φ = 1
$\phi R_n =$ 128.25 kips

DCR = **0.35** <1

Tensile Yielding:

A_g = 5.94 in^2
$\phi R_n = \phi F_y A_g$ *(AISC 15th Ed. J4-1)*
Φ = 0.9
$\phi R_n =$ 192.38 kips

DCR = **0.44** <1

Interaction of Axial, Flexure and Shear Yielding in Plate:

N_U = 84.4 kips
V_U = 45.2 kips
$\phi R_{np} =$ 192.38 kips
$\phi R_{nv} =$ 111.91 kips
$\phi M_n =$ 456.89 kip-in

$$\frac{N_u}{\Phi R_{np}} =$$ 0.44 >0.2 So use AISC 15th Ed. H1-1a

$$[\frac{N_u}{\Phi R_{np}} + \frac{8}{9}(\frac{V_u a}{\Phi M_n})]^2 + [\frac{V_u}{\Phi R_{nv}}]^2 =$$ **0.8** <1 *(AISC 15th Ed. H1-1a)*

$$\left(\frac{N_u}{2\phi R_{np}} + \frac{V_u a}{\phi M_n}\right)^2 + \left(\frac{V_u}{\phi R_{nv}}\right)^2 =$$ **N/A** >1 *(AISC 15th Ed. H1-1b)*

Flexure Rupture:

$$Z = \frac{t^*D^{\wedge}2}{4}$$

Z_{net} = 14.1 in^3
Φ = 0.75
$\phi M_n =$ 613.42 k-in
$\phi R_n =$ 150.26

DCR = **0.3** <1

Shear Rupture:

A_{nv} = 5.94 in^2
$\phi R_n = \phi 0.6 F_u A_n$ *(AISC 15th Ed. J4-4)*
Φ = 0.75
$\phi R_n =$ 154.97 kips

DCR = **0.29** <1

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Tensile Rupture:

A_n = 5.94 in^2

U = 1

$\phi R_n = \phi F_u A_e$ *(AISC 15th Ed. J4-2)*

Φ = 0.75

$\phi R_n =$ 258.28

DCR = **0.18** <1

Interaction of Axial, Flexure and Shear Rupture in Plate:

N_U = 84.4 kips

V_U = 45.2 kips

$\phi R_{np} =$ 258.28 kips

$\phi R_{nv} =$ 150.26 kips

$\phi M_n =$ 613.42 kip-in

$\frac{N_u}{\Phi R_{np}} =$ 0.33 >0.2 So use AISC 15th Ed. H1-1a

$[\frac{N_u}{\Phi R_{np}} + \frac{8}{9}(\frac{V_u a}{\Phi M_n})]^2 + [\frac{V_u}{\Phi R_{nv}}]^2 =$ **0.44** <1 *(AISC 15th Ed. H1-1a)*

$\left(\frac{N_u}{2\phi R_{np}} + \frac{V_u a}{\phi M_n}\right)^2 + \left(\frac{V_u}{\phi R_{nv}}\right)^2 =$ **N/A** >1 *(AISC 15th Ed. H1-1b)*

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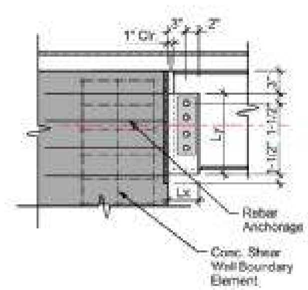
ED-2

STEEL DRAG BEAM CONNECTION CALCULATION

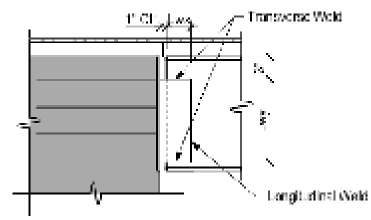
WHAT TYPE OF CONNECTION IS BEING USED? **BOLT: SINGLE ROW** *CONVENTIONAL CONFIGURATION*

MATERIAL AND CONFIGURATION

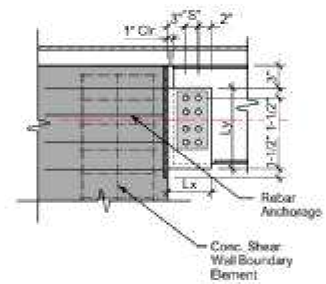
Embed Plate:	Height =	18 in	
	Width =	8 in	
	t =	1/2 in	
	F_y =	36 ksi	(A36 Steel)
	F_u =	58 ksi	(A36 Steel)
Shear Plate:	L_x =	5 in	
	L_y =	13.5 in	
	t =	3/8 in	
	Weld to Embed Plate =	3/16 in	
	F_y =	36 ksi	(A36 Steel)
	F_u =	58 ksi	(A36 Steel)
Rebar:	Size =	#5	
	# of Bars per Row =	2	
	# of Rows =	4	
	Vertical Spacing =	4.5 in	
	F_y (Rebar)=	60 ksi	(A706 Steel)
Bolts:	Ø =	1 in	
	# of Rows =	4	
	Vert. Spacing =	3 in	
	Horiz. Spacing =	0 in	
	Fnt (Bolts)=	45 ksi	(A307 Steel)
	Fnv (Bolts)=	27 ksi	(A307 Steel)
Beam Size:	W18X50		
	tf = 0.57		
	tw = 0.36		
	bf = 7.5		
	d = 18		
	Ag = 14.7		



BOLT: SINGLE ROW



WELDED



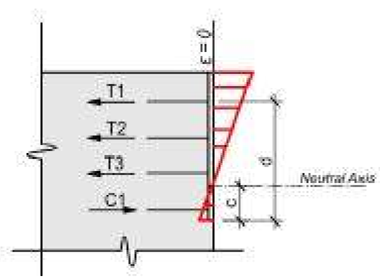
BOLT: DOUBLE ROW

APPLIED LOADS

Drag Load =	19.7 kips	(ASD)
Gravity Load =	44.7 kips	(LRFD)

$$\text{Drag Tensile Load} = \frac{\text{Drag Load}}{0.7*(\#\text{ Of Bars})}$$

Drag Tensile Load =	3.5 kips	(LRFD/Per Anchor)
Gravity Tensile Load =	3.1 kips	(LRFD/Per Anchor)
Total Tensile Load =	6.6 kips	(LRFD/Per Anchor)
Gravity Shear Load =	5.6 kips	(LRFD/Per Anchor)



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RESULTANT LOAD

$$R_U = \sqrt{(V_U^2 + N_U^2)}$$

R_U = 52.82 kips
Θ = 32.19

REBAR DESIGN

TENSILE CAPACITY OF SINGLE BAR

A_s = 0.31 in^2
Φ = 0.75 *(ACI 318-19 17.5.3a)*
$ΦN_N$ = 14.0 kips

SHEAR CAPACITY OF SINGLE BAR

A_s = 0.31 in^2
Φ = 0.65 *(ACI 318-19 17.5.3a)*
$ΦV_N$ = 12.1 kips

TENSION & SHEAR INTERACTION

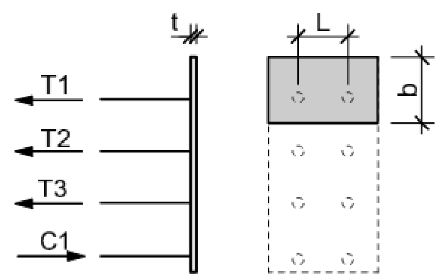
N_{UA} = 6.6 kips V_{UA} = 5.6 kips
$N_{UA}/ΦN_N$ = 0.47 $V_{UA}/ΦV_N$ = 0.46

$(N_{UA}/ΦN_N) + (V_{UA}/ΦV_N)$ = **0.94** <1.2

EMBED PLATE DESIGN

EMBED PLATE THICKNESS

L = 2 in
b = 5.25 in
T1 = 13.2 kips *(LRFD)*



$$M_{pl} = \frac{T1 \cdot L}{8}$$ *(AISC 15th Ed. 3-23.16)*

M_{pl} = 3.3 K-in

Flexure Yield:

$$Z = \frac{b \cdot t^2}{4}$$

Z = 0.33 in^3

$$\phi M_n = \phi F_y Z$$ *(AISC 15th Ed. F11-1)*

Φ = 0.9
$ΦM_N$ = 10.63 K-in

DCR = **0.31** <1

Shear Yield: A_{gv} = 7.5 in^2

$$\phi R_n = \phi 0.6 F_y A_{gv}$$ *(AISC 15th Ed. J4-3)*

Φ = 0.75
$Φ R_n$ = 121.5 kips

DCR = **0.37** <1

STRENGTH OF WELD

$$\mu = 1.0 + 0.5 \sin^{1.5}\theta$$ *(AISC 15th Ed. J2-5)*

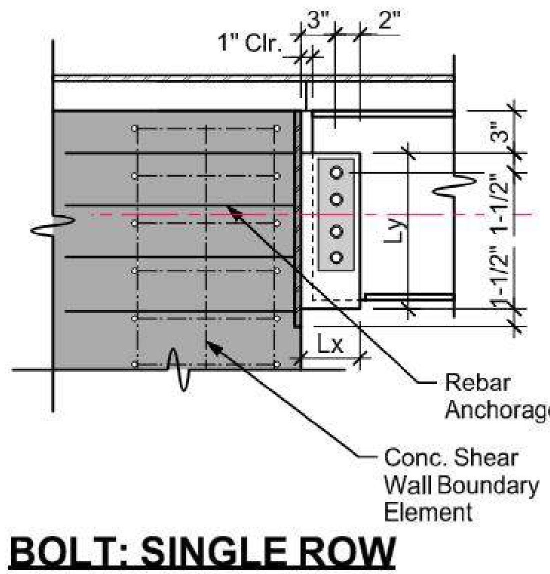
Θ = 32.19
μ = 1.19

$$R_n = (1.392 \text{ kip/in}) Dl\, \mu (2\ sides)$$ *(AISC 15th Ed. 8-2a)*

R_n = 134.68 kips

DCR = **0.39** <1

STRENGTH OF BOLTED CONN.



RESULTANT LOAD

$$R_U = \sqrt{(V_U^2 + N_U^2)}$$

R_U = 52.82 kips

Θ = 32.19

e = 1.5 *(AISC 15th Ed. T.10-9)*

C = 3.47

BEAM WEB STRENGTH

Bolt Shear: $\phi r_n = \phi F_n A_b$ *(AISC 15th Ed. J3-1)*

Φ = 0.75

Φ r_n = 15.9 kips/bolt

Bolt Bearing Strength: $\phi r_n = \phi 3.0 dt F_U$ *(AISC 15th Ed. J3-6b)*

Φ = 0.75

Φ r_n = 51.92 kips/bolt

Bolt Tearout Strength: $\phi r_n = \phi 1.5 l_c t F_U$ *(AISC 15th Ed. J3-6d)*

Φ = 0.75

Φ r_n = 37.32 kips/bolt

$Governing \, \phi r_n =$ 15.9 kips

$\phi R_n = C \phi r_n$

Φ R_n = 55.19 kips

DCR = **0.96** <1

SHEAR PLATE STRENGTH

Bolt Bearing Strength: $\phi r_n = \phi 3.0 dt F_U$ *(AISC 15th Ed. J3-6b)*

Φ = 0.75

Φ r_n = 48.94 kips/bolt

Bolt Tearout Strength: $\phi r_n = \phi 1.5 l_c t F_U$ *(AISC 15th Ed. J3-6d)*

Φ = 0.75

Φ r_n = 22.94 kips/bolt

$Governing \, \phi r_n =$ 15.9 kips

$\phi R_n = C \phi r_n$

Φ R_n = 55.19 kips

DCR = **0.96** <1

PLATE CHECKS

Maximum Plate Thick:
(Conventional) $t_{MAX} = (D_{BOLT}/2) + (1/16)$ *(AISC 15th Ed. T.10-9)*

t = 0.56 in

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STEEL DRAG BEAM CONNECTION CALCULATION

WHAT TYPE OF CONNECTION IS BEING USED? **BOLT:DOUBLE ROW** *EXTENDED CONFIGURATION*

MATERIAL AND CONFIGURATION

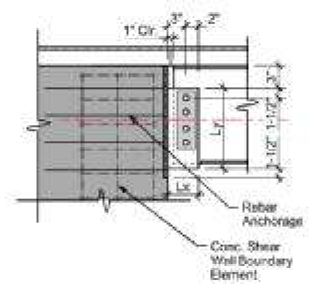
Embed Plate:
- Height = 16 in
- Width = 8 in
- t = 1/2 in
- F_y = 36 ksi (A36 Steel)
- F_U = 58 ksi (A36 Steel)

Shear Plate:
- L_X = 5 in
- L_Y = 11.5 in
- t = 3/8 in
- Weld to Embed Plate = 3/16 in
- F_y = 36 ksi (A36 Steel)
- F_U = 58 ksi (A36 Steel)

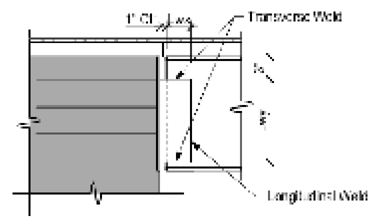
Rebar:
- Size = #5
- # of Bars per Row = 2
- # of Rows = 4
- Vertical Spacing = 3.83 in
- F_y (Rebar)= 60 ksi (A706 Steel)

Bolts:
- Ø = 1 in
- # of Rows = 3
- Vert. Spacing = 3.45 in
- Horiz. Spacing = 5.5 in
- Fnt (Bolts)= 45 ksi (A307 Steel)
- Fnv (Bolts)= 27 ksi (A307 Steel)

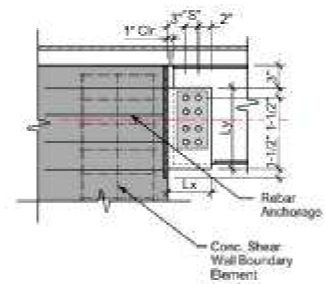
Beam Size: W16X36
- tf = 0.43
- tw = 0.3
- bf = 6.99
- d = 15.9
- Ag = 10.6



BOLT: SINGLE ROW



WELDED



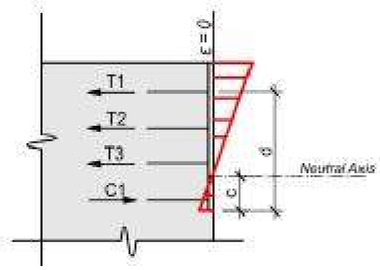
BOLT: DOUBLE ROW

APPLIED LOADS

- Drag Load = **37.5** kips (ASD)
- Gravity Load = **28.1** kips (LRFD)

$$\text{Drag Tensile Load} = \frac{\text{Drag Load}}{0.7^*(\#\text{ Of Bars})}$$

- Drag Tensile Load = 6.7 kips (LRFD/Per Anchor)
- Gravity Tensile Load = 4.3 kips (LRFD/Per Anchor)
- Total Tensile Load = 11.0 kips (LRFD/Per Anchor)
- Gravity Shear Load = 3.5 kips (LRFD/Per Anchor)



RESULTANT LOAD

$$R_U = \sqrt{(V_U^2 + N_U^2)}$$

- R_U = 60.49 kips
- Θ = 62.32

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REBAR DESIGN

TENSILE CAPACITY OF SINGLE BAR

A_s = 0.31 in^2

Φ = 0.75 *(ACI 318-19 17.5.3a)*

ΦN_N = 14.0 kips

SHEAR CAPACITY OF SINGLE BAR

A_s = 0.31 in^2

Φ = 0.65 *(ACI 318-19 17.5.3a)*

ΦV_N = 12.1 kips

TENSION & SHEAR INTERACTION

N_{UA} = 11.0 kips | V_{UA} = 3.5 kips

$N_{UA}/\Phi N_N$ = 0.79 | $V_{UA}/\Phi V_N$ = 0.29

$(N_{UA}/\Phi N_N) + (V_{UA}/\Phi V_N)$ = **1.08** <1.2

EMBED PLATE DESIGN

EMBED PLATE THICKNESS

L = 2 in

b = 4.92 in

T1 = 22.1 kips *(LRFD)*

$M_{pl} = \frac{T1 \cdot L}{8}$ *(AISC 15th Ed. 3-23.16)*

M_{pl} = 5.51 K-in

Flexure Yield:

$Z = \frac{b \cdot t^2}{4}$

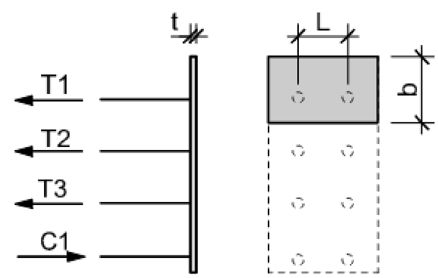
Z = 0.31 in^3

$\phi M_n = \phi F_y Z$ *(AISC 15th Ed. F11-1)*

Φ = 0.9

ΦM_N = 9.96 K-in

DCR = **0.55** <1



Shear Yield:

A_{gv} = 6.5 in^2

$\phi R_n = \phi\, 0.6 F_y A_{gv}$ *(AISC 15th Ed. J4-3)*

Φ = 0.75

ΦR_n = 105.3 kips

DCR = **0.27** <1

STRENGTH OF WELD

$\mu = 1.0 + 0.5 \sin^{1.5}\theta$ *(AISC 15th Ed. J2-5)*

Θ = 62.32

μ = 1.42

$R_n = (1.392\ \text{kip/in})\, Dl\, \mu\, (2\ sides)$ *(AISC 15th Ed. 8-2a)*

$R_n =$ 136.07 kips

DCR = **0.44** <1

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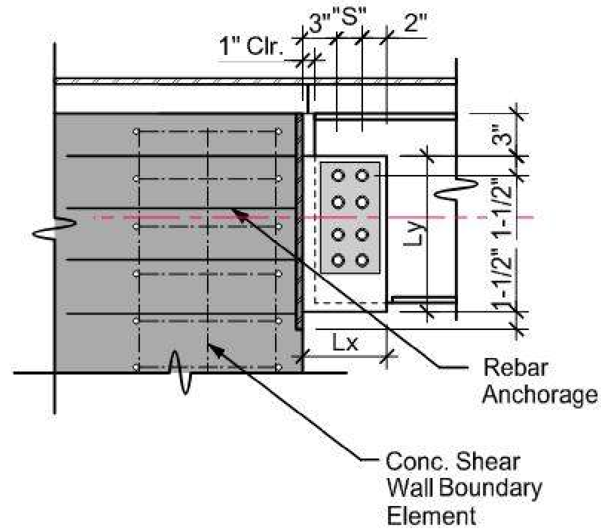
STRENGTH OF BOLTED CONN.



BOLT: DOUBLE ROW

RESULTANT LOAD

$$R_U = \sqrt{(V_U^2 + N_U^2)}$$

R_U = 60.49 kips
Θ = 62.32
e = 5.75 (AISC 15th Ed. T.10-9)
C = 4.08

BEAM WEB STRENGTH

Bolt Shear: $\phi r_n = \phi F_n A_b$ (AISC 15th Ed. J3-1)
Φ = 0.75
Φr_n = 15.9 kips/bolt

Bolt Bearing Strength: $\phi r_n = \phi 3.0 dt F_U$ (AISC 15th Ed. J3-6b)
Φ = 0.75
Φr_n = 43.14 kips/bolt

Bolt Tearout Strength: $\phi r_n = \phi 1.5 l_c t F_U$ (AISC 15th Ed. J3-6d)
Φ = 0.75
Φr_n = 31.01 kips/bolt

$Governing\ \phi r_n =$ 15.9 kips
$\phi R_n = C \phi r_n$
ΦR_n = 64.83 kips

DCR = **0.93** <1

SHEAR PLATE STRENGTH

Bolt Bearing Strength: $\phi r_n = \phi 3.0 dt F_U$ (AISC 15th Ed. J3-6b)
Φ = 0.75
Φr_n = 48.94 kips/bolt

Bolt Tearout Strength: $\phi r_n = \phi 1.5 l_c t F_U$ (AISC 15th Ed. J3-6d)
Φ = 0.75
Φr_n = 35.17 kips/bolt

$Governing\ \phi r_n =$ 15.9 kips
$\phi R_n = C \phi r_n$
ΦR_n = 64.83 kips

DCR = **0.93** <1

BEAM CHECKS

Shear Yielding: A_{gv} = 4.69 in²
$\phi R_n = \phi 0.6 F_y A_{gv}$ (AISC 15th Ed. J4-3)

$\phi R_n = \phi 0.6 F_y A_{gv}$

Φ = 1

Φ R_n = 140.72 kips

DCR = **0.2** <1

Tensile Yielding: A_g = 10.6 in^2

$\phi R_n = \phi F_y A_g$ *(AISC 15th Ed. J4-1)*

Φ = 0.9

Φ R_n = 477 kips

DCR = **0.11** <1

Tensile Rupture: A_n = 9.66 in^2

U = 1 - (x-bar/l)

$$\bar{x} = \frac{2b_f^2 t_f + t_w^2 (d - 2t_f)}{8b_f t_f + 4t_w (d - 2t_f)}$$

x_bar = 1.04 in

U = 0.81

$\phi R_n = \phi F_u A_e$ *(AISC 15th Ed. J4-2)*

Φ = 0.75

Φ R_n = 382.14 kips

DCR = **0.14** <1

Block Shear Rupture: A_{gv} = 4.43 in^2

A_{nv} = 3.95 in^2

A_{nt} = 1.92 in^2

U_{bs} = 1

Φ = 0.75

$\phi R_n = \phi 0.6 F_u A_{nv} + U_{bs} F_u A_{nt} \leq \phi 0.6 F_y A_{gv} + U_{bs} F_u A_{nt}$ *(AISC 15th Ed. J4-5)*

$\phi 0.6 F_y A_{gv} + U_{bs} F_u A_{nt} =$ 224.2 kips

$\phi 0.6 F_u A_{nv} + U_{bs} F_u A_{nt} =$ 240.32 kips

$\phi R_n =$ 224.2 kips

DCR = **0.17** <1

PLATE CHECKS

Maximum Plate Thick: $t\ max. = \frac{6^*M_{MAX}}{F_y{}^*Y}$ *(AISC 15th Ed. 10-3)*

$M_{MAX} = \frac{F_{wr}}{0.9}(A_b{}^*C')$ *(AISC 15th Ed. 10-4)*

C' = 21.2 *(AISC 15th Ed. 10-4)*

M_{MAX} = 499.51 k-in

t = 0.4 in

Flexure Yield: $\phi M_n = \phi F_y Z$ *(AISC 15th Ed. F11-1)*

$Z = \frac{t^*D\text{^}2}{4}$

Z = 12.4 in^3

Φ = 0.9

$\phi M_n =$ 401.71 k-in

$\phi R_n =$ 69.86 kips

DCR = **0.4** <1

Lateral-Torsional Buckling:

$\frac{0.08E}{F_y}$ = 64.44

$\frac{1.9E}{F_y}$ = 1530.56

$\frac{L_b d}{t^2}$ = 245.33

$\frac{0.08E}{F_y} < \frac{L_b d}{t^2} < \frac{1.9E}{F_y}$ So use AISC 15th Ed. F11-2b

$$M_N = C_b[1.52-0.274(\frac{L_b d}{t^2})\frac{F_y}{E}]M_y$$ *(AISC 15th Ed. F11-2)*

$$C_b = \frac{12.5M_{max}}{2.5M_{max}+3M_A+4M_B+3M_C}$$ *(AISC 15th Ed. F1-1)*

M_y = 297.56 kip-in

C_b = 1.67

Φ = 0.9

ϕM_n = 641.2 kip-in

ϕR_n = 111.51 kips

DCR = **0.25** <1

Shear Yielding: A_{gv} = 4.31 in²

$\phi R_n = \phi 0.6 F_y A_{gv}$ *(AISC 15th Ed. J4-3)*

Φ = 1

ϕR_n = 93.15 kips

DCR = **0.3** <1

Tensile Yielding: A_g = 4.31 in²

$\phi R_n = \phi F_y A_g$ *(AISC 15th Ed. J4-1)*

Φ = 0.9

ϕR_n = 139.73 kips

DCR = **0.38** <1

Interaction of Axial, Flexure and Shear Yielding in Plate:

N_U = 53.6 kips

V_U = 28.1 kips

ϕR_{np} = 139.73 kips

ϕR_{nv} = 93.15 kips

ϕM_n = 401.71 kip-in

$\frac{N_u}{\Phi R_{np}}$ = 0.38 >0.2 So use AISC 15th Ed. H1-1a

$$[\frac{N_u}{\Phi R_{np}} + \frac{8}{9}(\frac{V_u a}{\Phi M_n})]^2 + [\frac{V_u}{\Phi R_{nv}}]^2 =$$ **0.64** <1 *(AISC 15th Ed. H1-1a)*

$$\left(\frac{N_u}{2\phi R_{np}} + \frac{V_u a}{\phi M_n}\right)^2 + \left(\frac{V_u}{\phi R_{nv}}\right)^2 =$$ **N/A** >1 *(AISC 15th Ed. H1-1b)*

Flexure Rupture:

$$Z_{net} = \frac{tl^2}{4} - \frac{t}{4}\left[\left(d_h + 1/16 \text{ in.}\right)(s)\left(n^2-1\right) + \left(d_h + 1/16 \text{ in.}\right)^2\right]$$ *(AISC 15th Ed. 9-4)*

$\phi M_n = \phi F_u Z_{net}$

Z_{net} = 9.54 in³

Φ = 0.75

ϕM_n =

$\phi M_n =$ 415.14 k-in
$\phi R_n =$ 72.2

DCR = **0.39** <1

Shear Rupture:
A_{nv} = 3.12 in²
$\phi R_n = \phi 0.6 F_u A_{nv}$ *(AISC 15th Ed. J4-4)*
Φ = 0.75
$\phi R_n =$ 81.36 kips

DCR = **0.35** <1

Tensile Rupture:
A_{nt} = 3.12 in²
U = 1
$\phi R_n = \phi F_u A_e$ *(AISC 15th Ed. J4-2)*
Φ = 0.75
$\phi R_n =$ 135.6

DCR = **0.21** <1

Interaction of Axial, Flexure and Shear Rupture in Plate:
N_U = 53.6 kips
V_U = 28.1 kips
$\phi R_{np} =$ 135.6 kips
$\phi R_{nv} =$ 81.36 kips
$\phi M_n =$ 415.14 kip-in

$\frac{N_u}{\Phi R_{np}} =$ 0.4 >0.2 So use AISC 15th Ed. H1-1a

$$\left[\frac{N_u}{\Phi R_{np}} + \frac{8}{9}\left(\frac{V_u a}{\Phi M_n}\right)\right]^2 + \left[\frac{V_u}{\Phi R_{nv}}\right]^2 = \mathbf{0.67} < 1$$ *(AISC 15th Ed. H1-1a)*

$$\left(\frac{N_u}{2\phi R_{np}} + \frac{V_u a}{\phi M_n}\right)^2 + \left(\frac{V_u}{\phi R_{nv}}\right)^2 = \textbf{N/A} > 1$$ *(AISC 15th Ed. H1-1b)*

Block Shear Rupture (Beam Shear Direction):
A_{gv} = 3.75 in²
A_{nv} = 2.81 in²
A_{nt} = 2.06 in²
U_{bs} = 0.5
Φ = 0.75
$\phi R_n = \phi 0.6 F_u A_{nv} + U_{bs} F_u A_{nt} \leq \phi 0.6 F_y A_{gv} + U_{bs} F_u A_{nt}$ *(AISC 15th Ed. J4-5)*
$\phi 0.6 F_y A_{gv} + U_{bs} F_u A_{nt} =$ 120.56 kips
$\phi 0.6 F_u A_{nv} + U_{bs} F_u A_{nt} =$ 133.22 kips
$\phi R_n =$ 120.56 kips

DCR = **0.23** <1

Block Shear Rupture (Beam Axial Direction L Shape):
A_{gv} = 2.63 in²
A_{nv} = 2.06 in²
A_{nt} = 2.81 in²
U_{bs} = 1
Φ = 0.75
$\phi R_n = \phi 0.6 F_u A_{nv} + U_{bs} F_u A_{nt} \leq \phi 0.6 F_y A_{gv} + U_{bs} F_u A_{nt}$ *(AISC 15th Ed. J4-5)*
$\phi 0.6 F_y A_{gv} + U_{bs} F_u A_{nt} =$ 205.65 kips
$\phi 0.6 F_u A_{nv} + U_{bs} F_u A_{nt} =$ 216.96 kips
$\phi R_n =$ 205.65 kips

DCR = **0.18** <1

Block Shear Rupture (Beam Axial Direction U Shape):
A_{gv} = 5.25 in²
A_{nv} = 4.13 in²
A_{nt} = 2.44 in²
U_{bs} = 1

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Φ = 0.75

$$\phi R_n = \phi 0.6 F_u A_{nv} + U_{bs} F_u A_{nt} \leq \phi 0.6 F_y A_{gv} + U_{bs} F_u A_{nt}$$ *(AISC 15th Ed. J4-5)*

$\phi 0.6 F_y A_{gv} + U_{bs} F_u A_{nt} =$ 226.43 kips

$\phi 0.6 F_u A_{nv} + U_{bs} F_u A_{nt} =$ 249.04 kips

$\phi R_n =$ 226.43 kips

DCR = **0.17** <1

Block Shear Rupture (Comb. Axial & Shear U Shape):

V_U = 28.1 kips

N_U = 37.5 kips

ΦR_{bsv} = 120.56 kips

ΦR_{bsn} = 205.65 kips

$$\left(\frac{V_u}{\phi R_{bsv}}\right)^2 + \left(\frac{N_u}{\phi R_{bsn}}\right)^2 =$$ **0.09** <1

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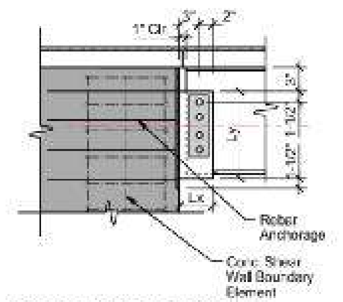
STEEL DRAG BEAM CONNECTION CALCULATION

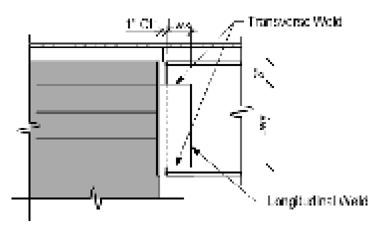
WHAT TYPE OF CONNECTION IS BEING USED? **WELDED**

MATERIAL AND CONFIGURATION

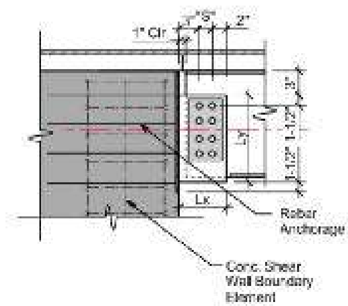
Embed Plate:	Height =	16 in	
	Width =	8 in	
	t =	5/8 in	
	F_y =	36 ksi	(A36 Steel)
	F_U =	58 ksi	(A36 Steel)
Shear Plate:	L_x =	5 in	
	L_Y =	11.5 in	
	t =	1/2 in	
	Weld to Embed Plate =	5/16 in	
	F_y =	36 ksi	(A36 Steel)
	F_U =	58 ksi	(A36 Steel)
Rebar:	Size =	#7	
	# of Bars per Row =	2	
	# of Rows =	4	
	Vertical Spacing =	3.83 in	
	F_y (Rebar)=	60 ksi	(A706 Steel)
Welds:	D =	5/16 in	
	Lwy	11.5 in	
	Lwx	8 in	



BOLT: SINGLE ROW



WELDED



BOLT: DOUBLE ROW

Beam Size:

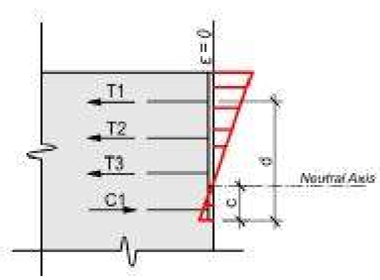
W16X36
tf = 0.43
tw = 0.3
bf = 6.99
d = 15.9
Ag = 10.6

APPLIED LOADS

Drag Load =	**123.4** kips	(ASD)
Gravity Load =	**0.0** kips	(LRFD)

$$\text{Drag Tensile Load} = \frac{\text{Drag Load}}{0.7 \cdot (\text{\# Of Bars})}$$

Drag Tensile Load =	22.0 kips	(LRFD/Per Anchor)
Gravity Tensile Load =	0.0 kips	(LRFD/Per Anchor)
Total Tensile Load =	22.0 kips	(LRFD/Per Anchor)
Gravity Shear Load =	0.0 kips	(LRFD/Per Anchor)



RESULTANT LOAD

$$R_U = \sqrt{(V_U^2 + N_U^2)}$$

R_U = 176.29 kips

Θ = 90

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REBAR DESIGN

TENSILE CAPACITY OF SINGLE BAR

A_s = 0.6 in^2
Φ = 0.75 *(ACI 318-19 17.5.3a)*
ΦN_N = 27.0 kips

SHEAR CAPACITY OF SINGLE BAR

A_s = 0.6 in^2
Φ = 0.65 *(ACI 318-19 17.5.3a)*
ΦV_N = 23.4 kips

TENSION & SHEAR INTERACTION

N_{UA} = 22.0 kips V_{UA} = 0.0 kips
$N_{UA}/\Phi N_N$ = 0.82 $V_{UA}/\Phi V_N$ = 0

$(N_{UA}/\Phi N_N) + (V_{UA}/\Phi V_N)$ = **0.82** <1.2

EMBED PLATE DESIGN

EMBED PLATE THICKNESS

L = 2 in
b = 4.92 in
T1 = 44.1 kips *(LRFD)*

$M_{pl} = \frac{T1 \cdot L}{8}$ *(AISC 15th Ed. 3-23.16)*

M_{pl} = 11.02 K-in

Flexure Yield:

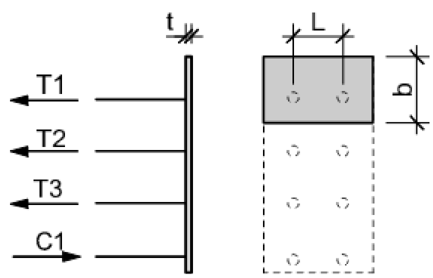
$Z = \frac{b \cdot t^2}{4}$

Z = 0.48 in^3

$\phi M_n = \phi F_y Z$ *(AISC 15th Ed. F11-1)*

Φ = 0.9
ΦM_N = 15.56 K-in

DCR = **0.71** <1



Shear Yield:

A_{gv} = 8.13 in^2

$\phi R_n = \phi 0.6 F_y A_{gv}$ *(AISC 15th Ed. J4-3)*

Φ = 0.75
ΦR_n = 131.63 kips

DCR = **0** <1

STRENGTH OF WELD

$\mu = 1.0 + 0.5 \sin^{1.5}\theta$ *(AISC 15th Ed. J2-5)*

Θ = 90
μ = 1.5

$R_n = (1.392\ \text{kip/in}) Dl\ \mu (2\ sides)$ *(AISC 15th Ed. 8-2a)*

$R_n =$ 240.12 kips

DCR = **0.73** <1

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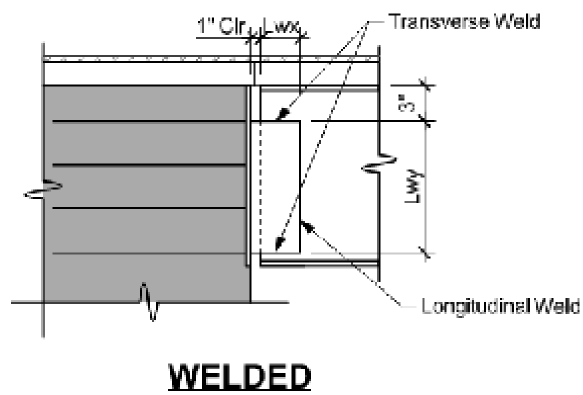
STRENGTH OF WELDED CONN.



RESULTANT LOAD

$$R_U = \sqrt{(V_U^2 + N_U^2)}$$

R_U = 176.29 kips

Θ = 90

WELD STRENGTH

$\frac{kl}{l}$ = $\frac{8}{11.5}$ = k = 0.7

x = 0.20

xl = 2.33 in

e_x = 6.67 in

$\frac{e_x}{l}$ = a = 0.58

C = 4.49

$$\phi R_n = \phi C C_1 D l$$ (AISC 15th Ed. 8-21)

Φ = 0.75

C_1 = 1 (AISC 15th Ed. T.8-3)

Gravity Load: $\phi R_n =$ 193.52 kips

DCR = **0** <1

Drag Load: 269.24

DCR = **0.65** <1

BEAM CHECKS

Shear Rupture of Beam Web: $$t_{min} = \frac{3.09 D}{F_U}$$ (AISC 15th Ed. 8-21)

t_{MIN} = 0.01 in

DCR = **0.05** <1

Shear Yielding: A_{gv} = 4.69 in^2

$$\phi R_n = \phi 0.6 F_y A_{gv}$$ (AISC 15th Ed. J4-3)

Φ = 1

Φ R_n = 140.72 kips

DCR = **0** <1

Tensile Yielding: A_g = 10.6 in^2

$$\phi R_n = \phi F_y A_g$$ (AISC 15th Ed. J4-1)

Φ = 0.9

Φ R_n = 477 kips

DCR = **0.37** <1

Tensile Rupture:

A_n = 10.6 in^2

U = 1 - (x-bar/l)

$$\bar{x} = \frac{2b_f^2 t_f + t_w^2 (d - 2t_f)}{8b_f t_f + 4t_w (d - 2t_f)}$$

x_bar = 1.04 in

U = 0.87

$\phi R_n = \phi F_u A_e$ *(AISC 15th Ed. J4-2)*

Φ = 0.75

ΦR_n = 449.78 kips

DCR = **0.39** <1

Block Shear Rupture:

A_{gv} = 4.72 in^2

A_{nv} = 4.72 in^2

A_{nt} = 3.39 in^2

U_{bs} = 1

Φ = 0.75

$\phi R_n = \phi 0.6 F_u A_{nv} + U_{bs} F_u A_{nt} \leq \phi 0.6 F_y A_{gv} + U_{bs} F_u A_{nt}$ *(AISC 15th Ed. J4-5)*

$\phi 0.6 F_y A_{gv} + U_{bs} F_u A_{nt} =$ 326.71 kips

$\phi 0.6 F_u A_{nv} + U_{bs} F_u A_{nt} =$ 358.57 kips

$\phi R_n =$ 326.71 kips

DCR = **0.38** <1

PLATE CHECKS

Shear Rupture of Plate:

$$t_{min} = \frac{3.09 D}{F_U}$$ *(AISC 15th Ed. 8-21)*

t_{MIN} = 0.02 in

DCR = **0.03** <1

Flexure Yield: $\phi M_n = \phi F_y Z$ *(AISC 15th Ed. F11-1)*

$$Z = \frac{t*D^2}{4}$$

Z = 16.53 in^3

Φ = 0.9

$\phi M_n =$ 535.61 k-in

$\phi R_n =$ 80.25 kips

DCR = **0** <1

Lateral-Torsional Buckling:

$\frac{0.08E}{F_y} =$ 64.44

$\frac{1.9E}{F_y} =$ 1530.56

$\frac{L_b d}{t^2} =$ 368

$\frac{0.08E}{F_y} < \frac{L_b d}{t^2} < \frac{1.9E}{F_y}$ So use AISC 15th Ed. F11-2b

F_y t^2 F_y

$$M_N = C_b[1.52 - 0.274(\frac{L_b d}{t^2})\frac{F_y}{E}]M_y$$ *(AISC 15th Ed. F11-2)*

$$C_b = \frac{12.5M_{max}}{2.5M_{max} + 3M_A + 4M_B + 3M_C}$$ *(AISC 15th Ed. F1-1)*

M_y = 595.13 kip-in

C_b = 1.67

Φ = 0.9

$\phi M_n =$ 1245.15 kip-in

$\phi R_n =$ 186.57 kips

DCR = **0** <1

Shear Yielding:

A_{gv} = 5.75 in²

$\phi R_n = \phi 0.6 F_y A_{gv}$ *(AISC 15th Ed. J4-3)*

Φ = 1

$\phi R_n =$ 124.2 kips

DCR = **0** <1

Tensile Yielding:

A_g = 5.75 in²

$\phi R_n = \phi F_y A_g$ *(AISC 15th Ed. J4-1)*

Φ = 0.9

$\phi R_n =$ 186.3 kips

DCR = **0.95** <1

Interaction of Axial, Flexure and Shear Yielding in Plate:

N_U = 176.3 kips

V_U = 0.0 kips

$\phi R_{np} =$ 186.3 kips

$\phi R_{nv} =$ 80.25 kips

$\phi M_n =$ 535.61 kip-in

$\frac{N_u}{\Phi R_{np}}$ = 0.95 >0.2 So use AISC 15th Ed. H1-1a

$$[\frac{N_u}{\Phi R_{np}} + \frac{8}{9}(\frac{V_u a}{\Phi M_n})]^2 + [\frac{V_u}{\Phi R_{nv}}]^2 =$$ **0.9** <1 *(AISC 15th Ed. H1-1a)*

$$\left(\frac{N_u}{2\phi R_{np}} + \frac{V_u a}{\phi M_n}\right)^2 + \left(\frac{V_u}{\phi R_{nv}}\right)^2 =$$ **N/A** >1 *(AISC 15th Ed. H1-1b)*

Flexure Rupture:

$$Z = \frac{t*D^2}{4}$$

Z_{net} = 16.53 in³

Φ = 0.75

$\phi M_n =$ 719.11 k-in

$\phi R_n =$ 107.75

DCR = **0** <1

Shear Rupture:

A_{nv} = 5.75 in²

$\phi R_n = \phi 0.6 F_u A_n$ *(AISC 15th Ed. J4-4)*

Φ = 0.75

$\phi R_n =$ 150.08 kips

DCR = **0** <1

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Tensile Rupture:

A_n = 5.75 in^2

U = 1

$\phi R_n = \phi F_u A_e$ *(AISC 15th Ed. J4-2)*

Φ = 0.75

$\phi R_n =$ 250.13 kips

DCR = **0.7** <1

Interaction of Axial, Flexure and Shear Rupture in Plate:

N_U = 176.3 kips

V_U = 0.0 kips

$\phi R_{np} =$ 250.13 kips

$\phi R_{nv} =$ 107.75 kips

$\phi M_n =$ 719.11 kip-in

$\frac{N_u}{\Phi R_{np}}$ = 0.7 >0.2 So use AISC 15th Ed. H1-1a

$[\frac{N_u}{\Phi R_{np}} + \frac{8}{9}(\frac{V_u a}{\Phi M_n})]^2 + [\frac{V_u}{\Phi R_{nv}}]^2 =$ **0.5** <1 *(AISC 15th Ed. H1-1a)*

$\left(\frac{N_u}{2\phi R_{np}} + \frac{V_u a}{\phi M_n}\right)^2 + \left(\frac{V_u}{\phi R_{nv}}\right)^2 =$ **N/A** >1 *(AISC 15th Ed. H1-1b)*

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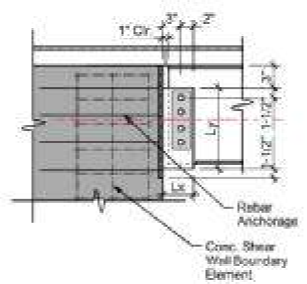
ED-3W

STEEL DRAG BEAM CONNECTION CALCULATION

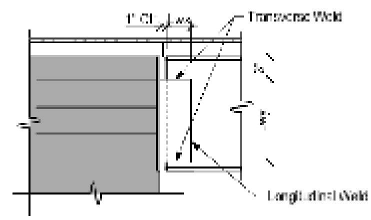
WHAT TYPE OF CONNECTION IS BEING USED? **WELDED**

MATERIAL AND CONFIGURATION

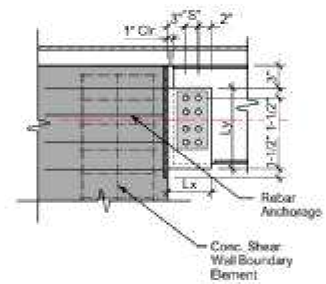
Embed Plate:	Height =	18 in	
	Width =	8 in	
	t =	1/2 in	
	F_y =	36 ksi	*(A36 Steel)*
	F_U =	58 ksi	*(A36 Steel)*
Shear Plate:	L_x =	5 in	
	L_Y =	13.5 in	
	t =	1/2 in	
	Weld to Embed Plate =	3/16 in	
	F_y =	36 ksi	*(A36 Steel)*
	F_U =	58 ksi	*(A36 Steel)*
Rebar:	Size =	#7	
	# of Bars per Row =	2	
	# of Rows =	4	
	Vertical Spacing =	4.5 in	
	F_y (Rebar)=	60 ksi	*(A706 Steel)*
Welds:	D =	5/16 in	
	Lwy	13.5 in	
	Lwx	4 in	



BOLT: SINGLE ROW



WELDED



BOLT: DOUBLE ROW

Beam Size:

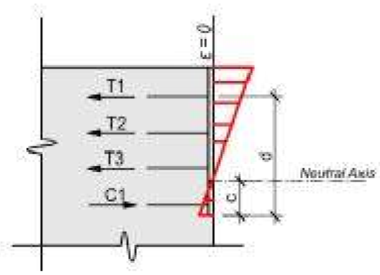
W18X50
tf = 0.57
tw = 0.36
bf = 7.5
d = 18
Ag = 14.7

APPLIED LOADS

Drag Load =	**60.2** kips	*(ASD)*
Gravity Load =	**72.3** kips	*(LRFD)*

$$\text{Drag Tensile Load} = \frac{\text{Drag Load}}{0.7*(\#\ \text{Of Bars})}$$

Drag Tensile Load =	10.8 kips	*(LRFD/Per Anchor)*
Gravity Tensile Load =	5.7 kips	*(LRFD/Per Anchor)*
Total Tensile Load =	16.4 kips	*(LRFD/Per Anchor)*
Gravity Shear Load =	9.0 kips	*(LRFD/Per Anchor)*



RESULTANT LOAD

$$R_U = \sqrt{(V_U^2 + N_U^2)}$$

R_U = 112.35 kips

Θ = 49.95

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REBAR DESIGN

TENSILE CAPACITY OF SINGLE BAR

A_s = 0.6 in^2

Φ = 0.75 *(ACI 318-19 17.5.3a)*

ΦN_N = 27.0 kips

SHEAR CAPACITY OF SINGLE BAR

A_s = 0.6 in^2

Φ = 0.65 *(ACI 318-19 17.5.3a)*

ΦV_N = 23.4 kips

TENSION & SHEAR INTERACTION

N_{UA} = 16.4 kips V_{UA} = 9.0 kips

$N_{UA}/\Phi N_N$ = 0.61 $V_{UA}/\Phi V_N$ = 0.39

$(N_{UA}/\Phi N_N) + (V_{UA}/\Phi V_N)$ = **0.99** <1.2

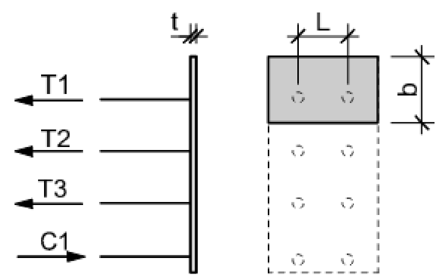
EMBED PLATE DESIGN

EMBED PLATE THICKNESS

L = 2 in

b = 5.25 in

T1 = 32.8 kips *(LRFD)*



$M_{pl} = \frac{T1 \cdot L}{8}$ *(AISC 15th Ed. 3-23.16)*

M_{pl} = 8.21 K-in

Flexure Yield: $Z = \frac{b \cdot t^2}{4}$

Z = 0.33 in^3

$\phi M_n = \phi F_y Z$ *(AISC 15th Ed. F11-1)*

Φ = 0.9

ΦM_N = 10.63 K-in

DCR = **0.77** <1

Shear Yield: A_{gv} = 7.5 in^2

$\phi R_n = \phi\, 0.6 F_y A_{gv}$ *(AISC 15th Ed. J4-3)*

Φ = 0.75

ΦR_n = 121.5 kips

DCR = **0.6** <1

STRENGTH OF WELD

$\mu = 1.0 + 0.5 \sin^{1.5}\theta$ *(AISC 15th Ed. J2-5)*

Θ = 49.95

μ = 1.33

$R_n = (1.392\ \text{kip/in})\, Dl\, \mu\, (2\ sides)$ *(AISC 15th Ed. 8-2a)*

$R_n =$ 150.51 kips

DCR = **0.75** <1

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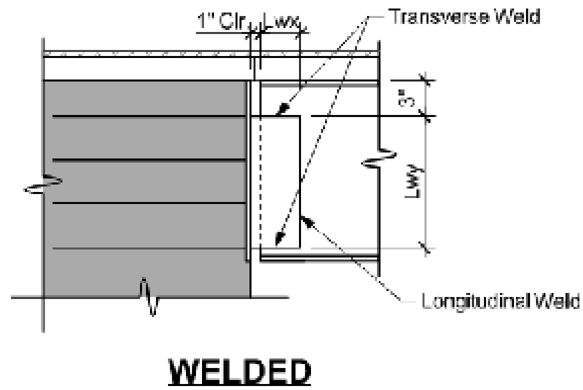
STRENGTH OF WELDED CONN.

RESULTANT LOAD

$$R_U = \sqrt{(V_U^2 + N_U^2)}$$

R_U = 112.35 kips
Θ = 49.95



WELD STRENGTH

$$\frac{kl}{l} = \frac{4}{13.5}$$ = k = 0.3

x = 0.06
xl = 0.74 in
e_x = 4.26 in

$$\frac{e_x}{l} =$$ a = 0.32

C = 3.27

$$\phi R_n = \phi C C_1 D l$$ (AISC 15th Ed. 8-21)

Φ = 0.75
C_1 = 1 (AISC 15th Ed. T.8-3)

Gravity Load: $\phi R_n =$ 165.51 kips

DCR = **0.44** <1

Drag Load: $\phi R_n =$ 98.08

DCR = **0.88** <1

BEAM CHECKS

Shear Rupture of Beam Web: $$t_{min} = \frac{3.09 D}{F_U}$$ (AISC 15th Ed. 8-21)

t_{MIN} = 0.01 in

DCR = **0.04** <1

Shear Yielding: A_{gv} = 6.39 in^2

$$\phi R_n = \phi 0.6 F_y A_{gv}$$ (AISC 15th Ed. J4-3)

Φ = 1
Φ R_n = 191.7 kips

DCR = **0.38** <1

Tensile Yielding: A_g = 14.7 in^2

$$\phi R_n = \phi F_y A_g$$ (AISC 15th Ed. J4-1)

Φ = 0.9
Φ R_n = 661.5 kips

DCR = **0.13** <1

Tensile Rupture: A_n = 14.7 in^2

U = 1 - (x-bar/l)

$$\bar{x} = \frac{2b_f^2 t_f + t_w^2 (d - 2t_f)}{8b_f t_f + 4t_w (d - 2t_f)}$$

x_bar = 1.14 in

U = 0.72

$\phi R_n = \phi F_u A_e$ (AISC 15th Ed. J4-2)

Φ = 0.75

ΦR_n = 512.48 kips

DCR = **0.17** <1

Block Shear Rupture: A_{gv} = 2.84 in^2

A_{nv} = 2.84 in^2

A_{nt} = 4.79 in^2

U_{bs} = 1

Φ = 0.75

$\phi R_n = \phi 0.6 F_u A_{nv} + U_{bs} F_u A_{nt} \leq \phi 0.6 F_y A_{gv} + U_{bs} F_u A_{nt}$ (AISC 15th Ed. J4-5)

$\phi 0.6 F_y A_{gv} + U_{bs} F_u A_{nt} =$ 375.41 kips

$\phi 0.6 F_u A_{nv} + U_{bs} F_u A_{nt} =$ 394.58 kips

$\phi R_n =$ 375.41 kips

DCR = **0.16** <1

PLATE CHECKS

Shear Rupture of Plate: $t_{min} = \frac{3.09 D}{F_U}$ (AISC 15th Ed. 8-21)

t_{MIN} = 0.02 in

DCR = **0.03** <1

Flexure Yield: $\phi M_n = \phi F_y Z$ (AISC 15th Ed. F11-1)

$Z = \frac{t^* D^2}{4}$

Z = 22.78 in^3

Φ = 0.9

$\phi M_n =$ 738.11 k-in

$\phi R_n =$ 173.37 kips

DCR = **0.42** <1

Lateral-Torsional Buckling: $\frac{0.08E}{F_y}$ = 64.44

$\frac{1.9E}{F_y}$ = 1530.56

$\frac{L_b d}{t^2}$ = 216

$\frac{0.08E}{F_y} < \frac{L_b d}{t^2} < \frac{1.9E}{F_y}$ So use AISC 15th Ed. F11-2b

$F_y \quad E \quad F_y$

$$M_N = C_b[1.52 - 0.274(\frac{L_b d}{t^2})\frac{F_y}{E}]M_y$$ (AISC 15th Ed. F11-2)

$$C_b = \frac{12.5M_{max}}{2.5M_{max} + 3M_A + 4M_B + 3M_C}$$ (AISC 15th Ed. F1-1)

M_y = 820.13 kip-in

C_b = 1.67

Φ = 0.9

$\phi M_n =$ 1779.5 kip-in

$\phi R_n =$ 417.97 kips

DCR = **0.17** <1

Shear Yielding:

A_{gv} = 6.75 in^2

$\phi R_n = \phi 0.6 F_y A_{gv}$ (AISC 15th Ed. J4-3)

Φ = 1

$\phi R_n =$ 145.8 kips

DCR = **0.5** <1

Tensile Yielding:

A_g = 6.75 in^2

$\phi R_n = \phi F_y A_g$ (AISC 15th Ed. J4-1)

Φ = 0.9

$\phi R_n =$ 218.7 kips

DCR = **0.39** <1

Interaction of Axial, Flexure and Shear Yielding in Plate:

N_U = 86.0 kips

V_U = 72.3 kips

$\phi R_{np} =$ 218.7 kips

$\phi R_{nv} =$ 145.8 kips

$\phi M_n =$ 738.11 kip-in

$\frac{N_u}{\Phi R_{np}} =$ 0.39 >0.2 So use AISC 15th Ed. H1-1a

$$[\frac{N_u}{\Phi R_{np}} + \frac{8}{9}(\frac{V_u a}{\Phi M_n})]^2 + [\frac{V_u}{\Phi R_{nv}}]^2 =$$ **0.83** <1 (AISC 15th Ed. H1-1a)

$$\left(\frac{N_u}{2\phi R_{np}} + \frac{V_u a}{\phi M_n}\right)^2 + \left(\frac{V_u}{\phi R_{nv}}\right)^2 =$$ **N/A** >1 (AISC 15th Ed. H1-1b)

Flexure Rupture:

$Z = \frac{t^*D^\wedge 2}{4}$

Z_{net} = 22.78 in^3

Φ = 0.75

$\phi M_n =$ 990.98 k-in

$\phi R_n =$ 232.76

DCR = **0.31** <1

Shear Rupture:

A_{nv} = 6.75 in^2

$\phi R_n = \phi 0.6 F_u A_n$ (AISC 15th Ed. J4-4)

Φ = 0.75

$\phi R_n =$ 176.18 kips

DCR = **0.41** <1

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Tensile Rupture:

A_n = 6.75 in^2

U = 1

$\phi R_n = \phi F_u A_e$ *(AISC 15th Ed. J4-2)*

Φ = 0.75

$\phi R_n =$ 293.63 kips

DCR = **0.29** <1

Interaction of Axial, Flexure and Shear Rupture in Plate:

N_U = 86.0 kips

V_U = 72.3 kips

$\phi R_{np} =$ 293.63 kips

$\phi R_{nv} =$ 176.18 kips

$\phi M_n =$ 990.98 kip-in

$\frac{N_u}{\Phi R_{np}} =$ 0.29 >0.2 So use AISC 15th Ed. H1-1a

$[\frac{N_u}{\Phi R_{np}} + \frac{8}{9}(\frac{V_u a}{\Phi M_n})]^2 + [\frac{V_u}{\Phi R_{nv}}]^2 =$ **0.49** <1 *(AISC 15th Ed. H1-1a)*

$\left(\frac{N_u}{2\phi R_{np}} + \frac{V_u a}{\phi M_n}\right)^2 + \left(\frac{V_u}{\phi R_{nv}}\right)^2 =$ **N/A** >1 *(AISC 15th Ed. H1-1b)*

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ED-4D

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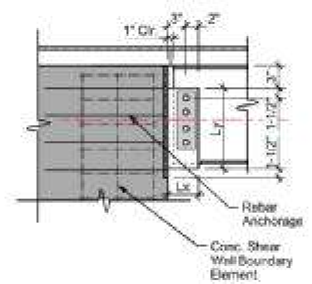
STEEL DRAG BEAM CONNECTION CALCULATION

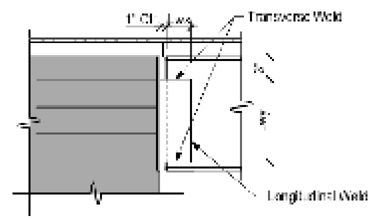
WHAT TYPE OF CONNECTION IS BEING USED? **BOLT:DOUBLE ROW** *EXTENDED CONFIGURATION*

MATERIAL AND CONFIGURATION

Embed Plate:	Height =	21 in	
	Width =	8 in	
	t =	1/2 in	
	F_y =	36 ksi	(A36 Steel)
	F_U =	58 ksi	(A36 Steel)
Shear Plate:	L_x =	5 in	
	L_Y =	16.5 in	
	t =	3/8 in	
	Weld to Embed Plate =	3/16 in	
	F_y =	36 ksi	(A36 Steel)
	F_U =	58 ksi	(A36 Steel)
Rebar:	Size =	#7	
	# of Bars per Row =	2	
	# of Rows =	4	
	Vertical Spacing =	5.5 in	
	F_y (Rebar)=	60 ksi	(A706 Steel)
Bolts:	Ø =	1 in	
	# of Rows =	5	
	Vert. Spacing =	3.15 in	
	Horiz. Spacing =	3 in	
	Fnt (Bolts)=	45 ksi	(A307 Steel)
	Fnv (Bolts)=	27 ksi	(A307 Steel)
Beam Size:	W21X93		
	tf = 0.93		
	tw = 0.58		
	bf = 8.42		
	d = 21.6		
	Ag = 27.3		



BOLT: SINGLE ROW



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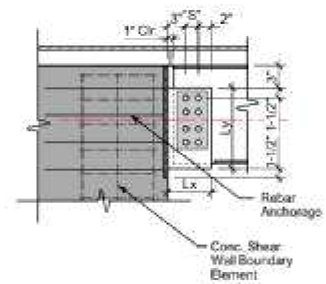


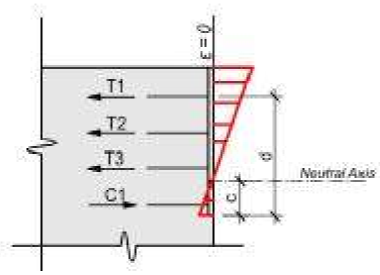
BOLT: DOUBLE ROW

APPLIED LOADS

Drag Load =	**45.0** kips	(ASD)
Gravity Load =	**70.0** kips	(LRFD)

$$\text{Drag Tensile Load} = \frac{\text{Drag Load}}{0.7\,(\#\ \text{Of Bars})}$$

Drag Tensile Load =	8.0 kips	(LRFD/Per Anchor)
Gravity Tensile Load =	6.8 kips	(LRFD/Per Anchor)
Total Tensile Load =	14.8 kips	(LRFD/Per Anchor)
Gravity Shear Load =	8.8 kips	(LRFD/Per Anchor)



SAN LUIS OBISPO | PASADENA | SANTA ROSA

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RESULTANT LOAD

$$R_U = \sqrt{(V_U^2 + N_U^2)}$$

R_U = 95.04 kips

Θ = 42.56

REBAR DESIGN

TENSILE CAPACITY OF SINGLE BAR

A_s = 0.6 in^2

Φ = 0.75 *(ACI 318-19 17.5.3a)*

$ΦN_N$ = 27.0 kips

SHEAR CAPACITY OF SINGLE BAR

A_s = 0.6 in^2

Φ = 0.65 *(ACI 318-19 17.5.3a)*

$ΦV_N$ = 23.4 kips

TENSION & SHEAR INTERACTION

N_{UA} = 14.8 kips V_{UA} = 8.8 kips

$N_{UA}/ΦN_N$ = 0.55 $V_{UA}/ΦV_N$ = 0.37

$(N_{UA}/ΦN_N) + (V_{UA}/ΦV_N)$ = **0.92** <1.2

EMBED PLATE DESIGN

EMBED PLATE THICKNESS

L = 2 in

b = 5.75 in

T1 = 29.6 kips *(LRFD)*

$$M_{pl} = \frac{T1 \cdot L}{8}$$ *(AISC 15th Ed. 3-23.16)*

M_{pl} = 7.41 K-in

Flexure Yield:

$$Z = \frac{b \cdot t^2}{4}$$

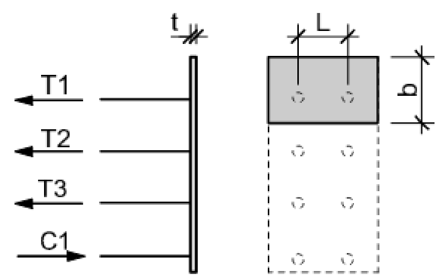
Z = 0.36 in^3

$$\phi M_n = \phi F_y Z$$ *(AISC 15th Ed. F11-1)*

Φ = 0.9

$ΦM_N$ = 11.64 K-in

DCR = **0.64** <1



Shear Yield:

A_{gv} = 9 in^2

$$\phi R_n = \phi\, 0.6 F_y A_{gv}$$ *(AISC 15th Ed. J4-3)*

Φ = 0.75

$Φ R_n$ = 145.8 kips

DCR = **0.48** <1

STRENGTH OF WELD

$$\mu = 1.0 + 0.5 \sin^{1.5}\theta$$ *(AISC 15th Ed. J2-5)*

Θ = 42.56

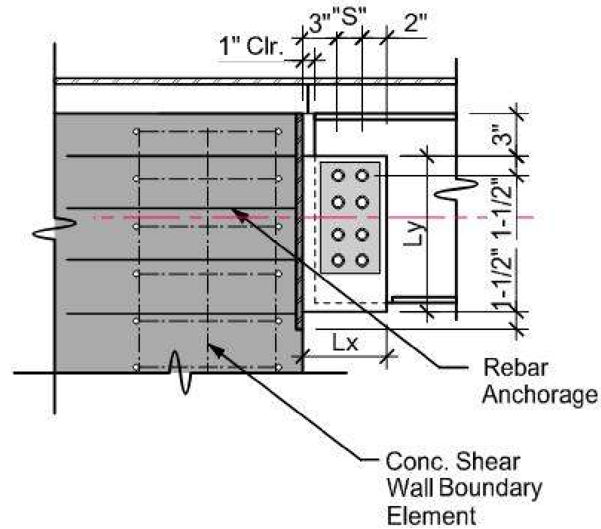
μ = 1.28

$$R_n = (1.392\ \text{kip/in})\, Dl\, \mu\, (2\ sides)$$ *(AISC 15th Ed. 8-2a)*

R_n = 176.14 kips

DCR = **0.54** <1

STRENGTH OF BOLTED CONN.



BOLT: DOUBLE ROW

RESULTANT LOAD

$$R_U = \sqrt{(V_U^2 + N_U^2)}$$

R_U = 95.04 kips

Θ = 42.56

e = 4.5 *(AISC 15th Ed. T.10-9)*

C = 6.92

BEAM WEB STRENGTH

Bolt Shear: $\phi r_n = \phi F_n A_b$ *(AISC 15th Ed. J3-1)*

Φ = 0.75

Φr_n = 15.9 kips/bolt

Bolt Bearing Strength: $\phi r_n = \phi 3.0 dt F_U$ *(AISC 15th Ed. J3-6b)*

Φ = 0.75

Φr_n = 84.83 kips/bolt

Bolt Tearout Strength: $\phi r_n = \phi 1.5 l_c t F_U$ *(AISC 15th Ed. J3-6d)*

Φ = 0.75

Φr_n = 60.97 kips/bolt

$Governing\ \phi r_n =$ 15.9 kips

$\phi R_n = C \phi r_n$

ΦR_n = 110.07 kips

DCR = **0.86** <1

SHEAR PLATE STRENGTH

Bolt Bearing Strength: $\phi r_n = \phi 3.0 dt F_U$ *(AISC 15th Ed. J3-6b)*

Φ = 0.75

Φr_n = 48.94 kips/bolt

Bolt Tearout Strength: $\phi r_n = \phi 1.5 l_c t F_U$ *(AISC 15th Ed. J3-6d)*

Φ = 0.75

Φr_n = 35.17 kips/bolt

$Governing\ \phi r_n =$ 15.9 kips

$\phi R_n = C \phi r_n$

ΦR_n = 110.07 kips

DCR = **0.86** <1

BEAM CHECKS

Shear Yielding: A_{gv} = 12.53 in^2

$\phi R_n = \phi 0.6 F_y A_{gv}$ *(AISC 15th Ed. J4-3)*

$\phi R_n = \phi 0.6 F_y A_{gv}$

Φ = 1

Φ R_n = 375.84 kips

DCR = **0.19** <1

Tensile Yielding: A_g = 27.3 in²

$\phi R_n = \phi F_y A_g$ *(AISC 15th Ed. J4-1)*

Φ = 0.9

Φ R_n = 1228.5 kips

DCR = **0.05** <1

Tensile Rupture: A_n = 24.22 in²

U = 1 - (x-bar/l)

$$\bar{x} = \frac{2b_f^2 t_f + t_w^2 (d - 2t_f)}{8b_f t_f + 4t_w (d - 2t_f)}$$

x_bar = 1.28 in

U = 0.57

$\phi R_n = \phi F_u A_e$ *(AISC 15th Ed. J4-2)*

Φ = 0.75

Φ R_n = 677.99 kips

DCR = **0.09** <1

Block Shear Rupture: A_{gv} = 5.8 in²

A_{nv} = 4.88 in²

A_{nt} = 5.51 in²

U_{bs} = 1

Φ = 0.75

$\phi R_n = \phi 0.6 F_u A_{nv} + U_{bs} F_u A_{nt} \leqslant \phi 0.6 F_y A_{gv} + U_{bs} F_u A_{nt}$ *(AISC 15th Ed. J4-5)*

$\phi 0.6 F_y A_{gv} + U_{bs} F_u A_{nt} =$ 488.65 kips

$\phi 0.6 F_u A_{nv} + U_{bs} F_u A_{nt} =$ 500.76 kips

$\phi R_n =$ 488.65 kips

DCR = **0.09** <1

PLATE CHECKS

Maximum Plate Thick: $t\ max. = \frac{6 * M_{max}}{F_y * Y}$ *(AISC 15th Ed. 10-3)*

$M_{max} = \frac{F_{nv}}{0.9} (A_b * C')$ *(AISC 15th Ed. 10-4)*

C' = 38.7 *(AISC 15th Ed. 10-4)*

M_{MAX} = 911.85 k-in

t = 0.4 in

Flexure Yield: $\phi M_n = \phi F_y Z$ *(AISC 15th Ed. F11-1)*

$Z = \frac{t * D^2}{4}$

Z = 25.52 in³

Φ = 0.9

$\phi M_n =$ 826.96 k-in

$\phi R_n =$ 183.77 kips

DCR = **0.38** <1

Lateral-Torsional Buckling:

$$\frac{0.08E}{F_y} = 64.44$$

$$\frac{1.9E}{F_y} = 1530.56$$

$$\frac{L_b d}{t^2} = 352$$

$$\frac{0.08E}{F_y} < \frac{L_b d}{t^2} < \frac{1.9E}{F_y}$$ So use AISC 15th Ed. F11-2b

$$M_N = C_b[1.52 - 0.274(\frac{L_b d}{t^2})\frac{F_y}{E}]M_y$$ *(AISC 15th Ed. F11-2)*

$$C_b = \frac{12.5M_{max}}{2.5M_{max} + 3M_A + 4M_B + 3M_C}$$ *(AISC 15th Ed. F1-1)*

M_y = 612.56 kip-in
C_b = 1.67
Φ = 0.9
$\phi M_n =$ 1286.63 kip-in
$\phi R_n =$ 285.92 kips

DCR = **0.24** <1

Shear Yielding: A_{gv} = 6.19 in²

$$\phi R_n = \phi 0.6 F_y A_{gv}$$ *(AISC 15th Ed. J4-3)*

Φ = 1
$\phi R_n =$ 133.65 kips

DCR = **0.52** <1

Tensile Yielding: A_g = 6.19 in²

$$\phi R_n = \phi F_y A_g$$ *(AISC 15th Ed. J4-1)*

Φ = 0.9
$\phi R_n =$ 200.48 kips

DCR = **0.32** <1

Interaction of Axial, Flexure and Shear Yielding in Plate:

N_U = 64.3 kips
V_U = 70.0 kips
$\phi R_{np} =$ 200.48 kips
$\phi R_{nv} =$ 133.65 kips
$\phi M_n =$ 826.96 kip-in

$$\frac{N_u}{\Phi R_{np}} = 0.32 > 0.2$$ So use AISC 15th Ed. H1-1a

$$[\frac{N_u}{\Phi R_{np}} + \frac{8}{9}(\frac{V_u a}{\Phi M_n})]^2 + [\frac{V_u}{\Phi R_{nv}}]^2 =$$ **0.71** <1 *(AISC 15th Ed. H1-1a)*

$$\left(\frac{N_u}{2\phi R_{np}} + \frac{V_u a}{\phi M_n}\right)^2 + \left(\frac{V_u}{\phi R_{nv}}\right)^2 =$$ **N/A** >1 *(AISC 15th Ed. H1-1b)*

Flexure Rupture:

$$Z_{net} = \frac{tl^2}{4} - \frac{t}{4}\left[(d_h + 1/16 \text{ in.})(s)(n^2 - 1) + (d_h + 1/16 \text{ in.})^2\right]$$ *(AISC 15th Ed. 9-4)*

$$\phi M_n = \phi F_u Z_{net}$$

Z_{net} = 17.89 in³
Φ = 0.75

$\phi M_n =$ 778.09 k-in

$\phi R_n =$ 172.91

DCR = **0.4** <1

Shear Rupture:

A_{nv} = 4.2 in^2

$\phi R_n = \phi 0.6 F_u A_{nv}$ *(AISC 15th Ed. J4-4)*

Φ = 0.75

$\phi R_n =$ 109.5 kips

DCR = **0.64** <1

Tensile Rupture:

A_{nt} = 4.2 in^2

U = 1

$\phi R_n = \phi F_u A_e$ *(AISC 15th Ed. J4-2)*

Φ = 0.75

$\phi R_n =$ 182.5

DCR = **0.38** <1

Interaction of Axial, Flexure and Shear Rupture in Plate:

N_U = 64.3 kips

V_U = 70.0 kips

$\phi R_{np} =$ 182.5 kips

$\phi R_{nv} =$ 109.5 kips

$\phi M_n =$ 778.09 kip-in

$\dfrac{N_u}{\Phi R_{np}} =$ 0.35 >0.2 So use AISC 15th Ed. H1-1a

$$\left[\frac{N_u}{\Phi R_{np}} + \frac{8}{9}\left(\frac{V_u a}{\Phi M_n}\right)\right]^2 + \left[\frac{V_u}{\Phi R_{nv}}\right]^2 = \textbf{0.92} < 1$$ *(AISC 15th Ed. H1-1a)*

$$\left(\frac{N_u}{2\phi R_{np}} + \frac{V_u a}{\phi M_n}\right)^2 + \left(\frac{V_u}{\phi R_{nv}}\right)^2 = \textbf{N/A} > 1$$ *(AISC 15th Ed. H1-1b)*

Block Shear Rupture (Beam Shear Direction):

A_{gv} = 5.63 in^2

A_{nv} = 3.94 in^2

A_{nt} = 1.13 in^2

U_{bs} = 0.5

Φ = 0.75

$\phi R_n = \phi 0.6 F_u A_{nv} + U_{bs} F_u A_{nt} \leqslant \phi 0.6 F_y A_{gv} + U_{bs} F_u A_{nt}$ *(AISC 15th Ed. J4-5)*

$\phi 0.6 F_y A_{gv} + U_{bs} F_u A_{nt} =$ 123.75 kips

$\phi 0.6 F_u A_{nv} + U_{bs} F_u A_{nt} =$ 135.39 kips

$\phi R_n =$ 123.75 kips

DCR = **0.57** <1

Block Shear Rupture (Beam Axial Direction L Shape):

A_{gv} = 1.69 in^2

A_{nv} = 1.13 in^2

A_{nt} = 3.94 in^2

U_{bs} = 1

Φ = 0.75

$\phi R_n = \phi 0.6 F_u A_{nv} + U_{bs} F_u A_{nt} \leqslant \phi 0.6 F_y A_{gv} + U_{bs} F_u A_{nt}$ *(AISC 15th Ed. J4-5)*

$\phi 0.6 F_y A_{gv} + U_{bs} F_u A_{nt} =$ 255.71 kips

$\phi 0.6 F_u A_{nv} + U_{bs} F_u A_{nt} =$ 257.74 kips

$\phi R_n =$ 255.71 kips

DCR = **0.18** <1

Block Shear Rupture (Beam Axial Direction U Shape):

A_{gv} = 3.38 in^2

A_{nv} = 2.25 in^2

A_{nt} = 3.56 in^2

U_{bs} = 1

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Φ = 0.75

$$\phi R_n = \phi 0.6 F_u A_{nv} + U_{bs} F_u A_{nt} \leq \phi 0.6 F_y A_{gv} + U_{bs} F_u A_{nt}$$ *(AISC 15th Ed. J4-5)*

$\phi 0.6 F_y A_{gv} + U_{bs} F_u A_{nt} =$ 261.3 kips

$\phi 0.6 F_u A_{nv} + U_{bs} F_u A_{nt} =$ 265.35 kips

$\phi R_n =$ 261.3 kips

DCR = **0.17** <1

Block Shear Rupture (Comb. Axial & Shear U Shape):

V_U = 70.0 kips

N_U = 45.0 kips

ΦR_{bsv} = 123.75 kips

ΦR_{bsn} = 255.71 kips

$$\left(\frac{V_u}{\phi R_{bsv}}\right)^2 + \left(\frac{N_u}{\phi R_{bsn}}\right)^2 =$$ **0.35** <1

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ED-5D

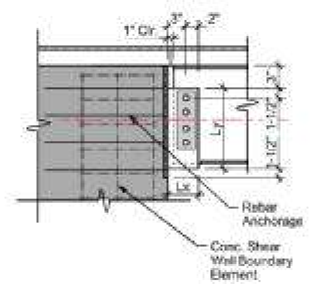
STEEL DRAG BEAM CONNECTION CALCULATION

(2016 CBC Section 16__)

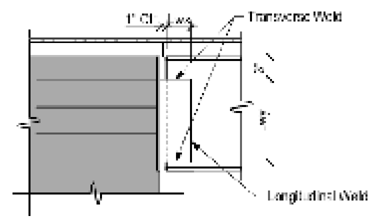
WHAT TYPE OF CONNECTION IS BEING USED? **BOLT:DOUBLE ROW** *EXTENDED CONFIGURATION*

MATERIAL AND CONFIGURATION

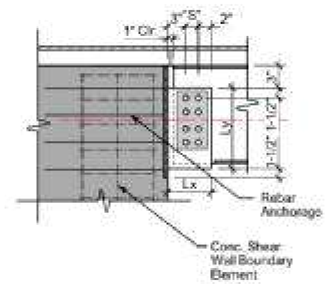
Embed Plate:	Height =	24 in	
	Width =	8 in	
	t =	3/8 in	
	F_y =	36 ksi	(A36 Steel)
	F_U =	58 ksi	(A36 Steel)
Shear Plate:	L_X =	5 in	
	L_Y =	19.5 in	
	t =	1/4 in	
	Weld to Embed Plate =	3/16 in	
	F_y =	36 ksi	(A36 Steel)
	F_U =	58 ksi	(A36 Steel)
Rebar:	Size =	#6	
	# of Bars per Row =	2	
	# of Rows =	4	
	Vertical Spacing =	6.5 in	
	F_y (Rebar)=	60 ksi	(A706 Steel)
Bolts:	Ø =	1 in	
	# of Rows =	5	
	Vert. Spacing =	3.78 in	
	Horiz. Spacing =	3 in	
	Fnt (Bolts)=	45 ksi	(A307 Steel)
	Fnv (Bolts)=	27 ksi	(A307 Steel)
Beam Size:	W24X84		
	tf =	0.77	
	tw =	0.47	
	bf =	9.02	
	d =	24.1	
	Ag =	24.7	



BOLT: SINGLE ROW



WELDED



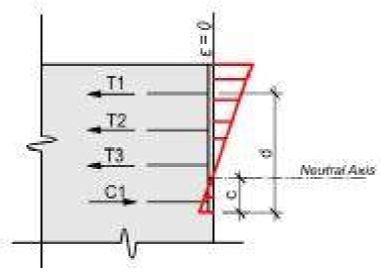
BOLT: DOUBLE ROW

APPLIED LOADS

Drag Load =	**43.6** kips	(ASD)
Gravity Load =	**52.8** kips	(LRFD)

$$\text{Drag Tensile Load} = \frac{\text{Drag Load}}{0.7*(\#\text{ Of Bars})}$$

Drag Tensile Load =	7.8 kips	(LRFD/Per Anchor)
Gravity Tensile Load =	4.0 kips	(LRFD/Per Anchor)
Total Tensile Load =	11.8 kips	(LRFD/Per Anchor)
Gravity Shear Load =	6.6 kips	(LRFD/Per Anchor)



RESULTANT LOAD

$$R_U = \sqrt{(V_U^2 + N_U^2)}$$

R_U = 81.65 kips

Θ = 49.71

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REBAR DESIGN

TENSILE CAPACITY OF SINGLE BAR

A_s = 0.44 in^2

Φ = 0.75   *(ACI 318-19 17.5.3a)*

$ΦN_N$ = 19.8 kips

SHEAR CAPACITY OF SINGLE BAR

A_s = 0.44 in^2

Φ = 0.65   *(ACI 318-19 17.5.3a)*

$ΦV_N$ = 17.2 kips

TENSION & SHEAR INTERACTION

N_{UA} = 11.8 kips V_{UA} = 6.6 kips

$N_{UA}/ΦN_N$ = 0.59 $V_{UA}/ΦV_N$ = 0.38

$(N_{UA}/ΦN_N) + (V_{UA}/ΦV_N)$ = **0.98** <1.2

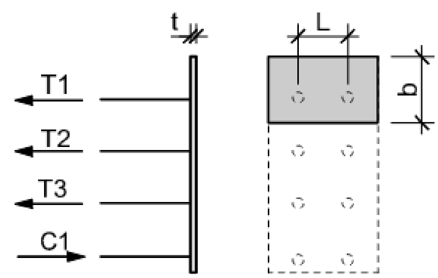
EMBED PLATE DESIGN

EMBED PLATE THICKNESS

L = 2 in

b = 6.25 in

T1 = 23.5 kips   *(LRFD)*



$M_{pl} = \frac{T1*L}{8}$ *(AISC 15th Ed. 3-23.16)*

M_{pl} = 5.88 K-in

Flexure Yield:

$Z = \frac{b*t^2}{4}$

Z = 0.22 in^3

$\phi M_n = \phi F_y Z$ *(AISC 15th Ed. F11-1)*

Φ = 0.9

$ΦM_N$ = 7.12 K-in

DCR = **0.83** <1

Shear Yield:

A_{gv} = 7.88 in^2

$\phi R_n = \phi\, 0.6 F_y A_{gv}$ *(AISC 15th Ed. J4-3)*

Φ = 0.75

$Φ R_n$ = 127.58 kips

DCR = **0.41** <1

STRENGTH OF WELD

$\mu = 1.0 + 0.5 \sin^{1.5}\theta$ *(AISC 15th Ed. J2-5)*

Θ = 49.71

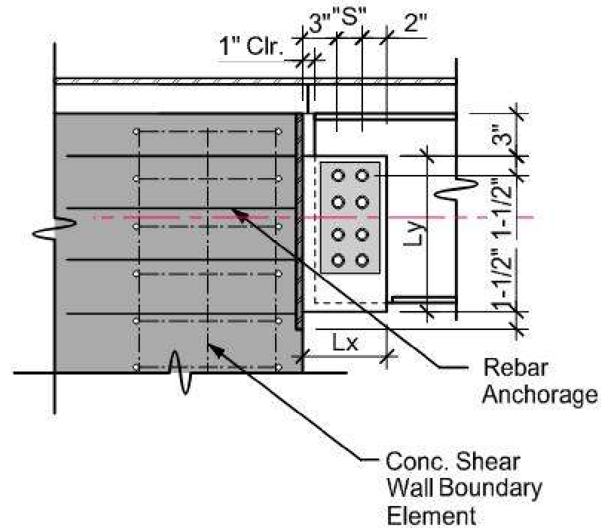
μ = 1.33

$R_n = (1.392\,\text{kip/in})\, Dl\, \mu\,(2\,sides)$ *(AISC 15th Ed. 8-2a)*

$R_n =$ 217.12 kips

DCR = **0.38** <1

STRENGTH OF BOLTED CONN.



BOLT: DOUBLE ROW

RESULTANT LOAD

$$R_U = \sqrt{(V_U^2 + N_U^2)}$$

R_U = 81.65 kips

Θ = 49.71

e = 4.5 *(AISC 15th Ed. T.10-9)*

C = 7.24

BEAM WEB STRENGTH

Bolt Shear: $\phi r_n = \phi F_n A_b$ *(AISC 15th Ed. J3-1)*

Φ = 0.75

Φ r_n = 15.9 kips/bolt

Bolt Bearing Strength: $\phi r_n = \phi 3.0\, dt\, F_U$ *(AISC 15th Ed. J3-6b)*

Φ = 0.75

Φ r_n = 68.74 kips/bolt

Bolt Tearout Strength: $\phi r_n = \phi\, 1.5 l_c\, t\, F_U$ *(AISC 15th Ed. J3-6d)*

Φ = 0.75

Φ r_n = 49.41 kips/bolt

$Governing\ \phi r_n =$ 15.9 kips

$\phi R_n = C \phi r_n$

Φ R_n = 115.07 kips

DCR = **0.71** <1

SHEAR PLATE STRENGTH

Bolt Bearing Strength: $\phi r_n = \phi 3.0\, dt\, F_U$ *(AISC 15th Ed. J3-6b)*

Φ = 0.75

Φ r_n = 32.63 kips/bolt

Bolt Tearout Strength: $\phi r_n = \phi\, 1.5 l_c\, t\, F_U$ *(AISC 15th Ed. J3-6d)*

Φ = 0.75

Φ r_n = 23.45 kips/bolt

$Governing\ \phi r_n =$ 15.9 kips

$\phi R_n = C \phi r_n$

Φ R_n = 115.07 kips

DCR = **0.71** <1

BEAM CHECKS

Shear Yielding: A_{gv} = 11.33 in^2

$\phi R_n = \phi 0.6 F_y A_{gv}$ *(AISC 15th Ed. J4-3)*

$\phi R_n = \phi 0.6 F_y A_{gv}$

Φ = 1

Φ R_n = 339.81 kips

DCR = **0.16** <1

Tensile Yielding: A_g = 24.7 in^2

$\phi R_n = \phi F_y A_g$ *(AISC 15th Ed. J4-1)*

Φ = 0.9

Φ R_n = 1111.5 kips

DCR = **0.06** <1

Tensile Rupture: A_n = 22.2 in^2

U = 1 - (x-bar/l)

$$\bar{x} = \frac{2b_f^2 t_f + t_w^2 (d - 2t_f)}{8b_f t_f + 4t_w (d - 2t_f)}$$

x_bar = 1.33 in

U = 0.56

$\phi R_n = \phi F_u A_e$ *(AISC 15th Ed. J4-2)*

Φ = 0.75

Φ R_n = 602.65 kips

DCR = **0.1** <1

Block Shear Rupture: A_{gv} = 4.7 in^2

A_{nv} = 3.95 in^2

A_{nt} = 5.88 in^2

U_{bs} = 1

Φ = 0.75

$\phi R_n = \phi 0.6 F_u A_{nv} + U_{bs} F_u A_{nt} \leqslant \phi 0.6 F_y A_{gv} + U_{bs} F_u A_{nt}$ *(AISC 15th Ed. J4-5)*

$\phi 0.6 F_y A_{gv} + U_{bs} F_u A_{nt} =$ 487.63 kips

$\phi 0.6 F_u A_{nv} + U_{bs} F_u A_{nt} =$ 497.44 kips

$\phi R_n =$ 487.63 kips

DCR = **0.09** <1

PLATE CHECKS

Maximum Plate Thick: $t\ max. = \frac{6 M_{MAX}}{F_y Y}$ *(AISC 15th Ed. 10-3)*

$M_{MAX} = \frac{F_{nv}}{0.9} (A_b C')$ *(AISC 15th Ed. 10-4)*

C' = 38.7 *(AISC 15th Ed. 10-4)*

M_{MAX} = 911.85 k-in

t = 0.3 in

Flexure Yield: $\phi M_n = \phi F_y Z$ *(AISC 15th Ed. F11-1)*

$Z = \frac{t D^2}{4}$

Z = 23.77 in^3

Φ = 0.9

$\phi M_n =$ 770.01 k-in

$\phi R_n =$ 171.11 kips

DCR = **0.31** <1

Lateral-Torsional Buckling:

$$\frac{0.08E}{F_y} = 64.44$$

$$\frac{1.9E}{F_y} = 1530.56$$

$$\frac{L_b d}{t^2} = 936$$

$$\frac{0.08E}{F_y} < \frac{L_b d}{t^2} < \frac{1.9E}{F_y}$$ So use AISC 15th Ed. F11-2b

$$M_N = C_b[1.52 - 0.274(\frac{L_b d}{t^2})\frac{F_y}{E}]M_y$$ *(AISC 15th Ed. F11-2)*

$$C_b = \frac{12.5M_{max}}{2.5M_{max} + 3M_A + 4M_B + 3M_C}$$ *(AISC 15th Ed. F1-1)*

M_y = 570.38 kip-in

C_b = 1.67

Φ = 0.9

$\phi M_n =$ 1028.07 kip-in

$\phi R_n =$ 228.46 kips

DCR = **0.23** <1

Shear Yielding:

A_{gv} = 4.88 in^2

$$\phi R_n = \phi 0.6 F_y A_{gv}$$ *(AISC 15th Ed. J4-3)*

Φ = 1

$\phi R_n =$ 105.3 kips

DCR = **0.5** <1

Tensile Yielding:

A_g = 4.88 in^2

$$\phi R_n = \phi F_y A_g$$ *(AISC 15th Ed. J4-1)*

Φ = 0.9

$\phi R_n =$ 157.95 kips

DCR = **0.39** <1

Interaction of Axial, Flexure and Shear Yielding in Plate:

N_U = 62.3 kips

V_U = 52.8 kips

$\phi R_{np} =$ 157.95 kips

$\phi R_{nv} =$ 105.3 kips

$\phi M_n =$ 770.01 kip-in

$$\frac{N_u}{\Phi R_{np}} = 0.39 > 0.2$$ So use AISC 15th Ed. H1-1a

$$[\frac{N_u}{\Phi R_{np}} + \frac{8}{9}(\frac{V_u a}{\Phi M_n})]^2 + [\frac{V_u}{\Phi R_{nv}}]^2 =$$ **0.7** <1 *(AISC 15th Ed. H1-1a)*

$$\left(\frac{N_u}{2\phi R_{np}} + \frac{V_u a}{\phi M_n}\right)^2 + \left(\frac{V_u}{\phi R_{nv}}\right)^2 =$$ **N/A** >1 *(AISC 15th Ed. H1-1b)*

Flexure Rupture:

$$Z_{net} = \frac{tl^2}{4} - \frac{t}{4}\left[(d_h + 1/16\text{ in.})(s)(n^2 - 1) + (d_h + 1/16\text{ in.})^2\right]$$ *(AISC 15th Ed. 9-4)*

$$\phi M_n = \phi F_u Z_{net}$$

Z_{net} = 17.68 in^3

Φ = 0.75

$\phi M_n =$ 769.02 k-in
$\phi R_n =$ 170.89

DCR = **0.31** <1

Shear Rupture:
A_{nv} = 3.55 in²
$\phi R_n = \phi 0.6 F_u A_{nv}$ *(AISC 15th Ed. J4-4)*
Φ = 0.75
$\phi R_n =$ 92.57 kips

DCR = **0.57** <1

Tensile Rupture:
A_{nt} = 3.55 in²
U = 1
$\phi R_n = \phi F_u A_e$ *(AISC 15th Ed. J4-2)*
Φ = 0.75
$\phi R_n =$ 154.29

DCR = **0.34** <1

Interaction of Axial, Flexure and Shear Rupture in Plate:
N_U = 62.3 kips
V_U = 52.8 kips
$\phi R_{np} =$ 154.29 kips
$\phi R_{nv} =$ 92.57 kips
$\phi M_n =$ 769.02 kip-in
$\frac{N_u}{\Phi R_{np}}$ = 0.4 >0.2 So use AISC 15th Ed. H1-1a

$$\left[\frac{N_u}{\Phi R_{np}} + \frac{8}{9}\left(\frac{V_u a}{\Phi M_n}\right)\right]^2 + \left[\frac{V_u}{\Phi R_{nv}}\right]^2 =$$ **0.79** <1 *(AISC 15th Ed. H1-1a)*

$$\left(\frac{N_u}{2\phi R_{np}} + \frac{V_u a}{\phi M_n}\right)^2 + \left(\frac{V_u}{\phi R_{nv}}\right)^2 =$$ **N/A** >1 *(AISC 15th Ed. H1-1b)*

Block Shear Rupture (Beam Shear Direction):
A_{gv} = 4.5 in²
A_{nv} = 3.38 in²
A_{nt} = 0.75 in²
U_{bs} = 0.5
Φ = 0.75
$\phi R_n = \phi 0.6 F_u A_{nv} + U_{bs} F_u A_{nt} \leq \phi 0.6 F_y A_{gv} + U_{bs} F_u A_{nt}$ *(AISC 15th Ed. J4-5)*
$\phi 0.6 F_y A_{gv} + U_{bs} F_u A_{nt} =$ 94.65 kips
$\phi 0.6 F_u A_{nv} + U_{bs} F_u A_{nt} =$ 109.84 kips
$\phi R_n =$ 94.65 kips

DCR = **0.56** <1

Block Shear Rupture (Beam Axial Direction L Shape):
A_{gv} = 1.13 in²
A_{nv} = 0.75 in²
A_{nt} = 3.38 in²
U_{bs} = 1
Φ = 0.75
$\phi R_n = \phi 0.6 F_u A_{nv} + U_{bs} F_u A_{nt} \leq \phi 0.6 F_y A_{gv} + U_{bs} F_u A_{nt}$ *(AISC 15th Ed. J4-5)*
$\phi 0.6 F_y A_{gv} + U_{bs} F_u A_{nt} =$ 213.98 kips
$\phi 0.6 F_u A_{nv} + U_{bs} F_u A_{nt} =$ 215.33 kips
$\phi R_n =$ 213.98 kips

DCR = **0.2** <1

Block Shear Rupture (Beam Axial Direction U Shape):
A_{gv} = 2.25 in²
A_{nv} = 1.5 in²
A_{nt} = 3.13 in²
U_{bs} = 1

Φ = 0.75

$$\phi R_n = \phi 0.6 F_u A_{nv} + U_{bs} F_u A_{nt} \leq \phi 0.6 F_y A_{gv} + U_{bs} F_u A_{nt}$$ *(AISC 15th Ed. J4-5)*

$\phi 0.6 F_y A_{gv} + U_{bs} F_u A_{nt} =$ 217.7 kips

$\phi 0.6 F_u A_{nv} + U_{bs} F_u A_{nt} =$ 220.4 kips

$\phi R_n =$ 217.7 kips

DCR = **0.2** <1

Block Shear Rupture (Comb. Axial & Shear U Shape):

V_U = 52.8 kips

N_U = 43.6 kips

ΦR_{bsv} = 94.65 kips

ΦR_{bsn} = 213.98 kips

$$\left(\frac{V_u}{\phi R_{bsv}}\right)^2 + \left(\frac{N_u}{\phi R_{bsn}}\right)^2 =$$ **0.35** <1

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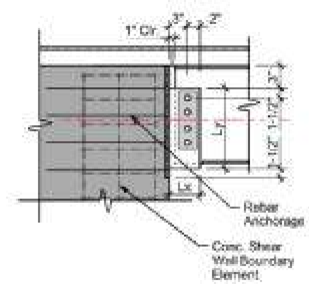
ED-5W

STEEL DRAG BEAM CONNECTION CALCULATION

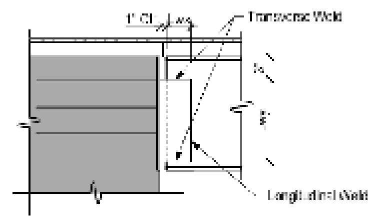
WHAT TYPE OF CONNECTION IS BEING USED? **WELDED**

MATERIAL AND CONFIGURATION

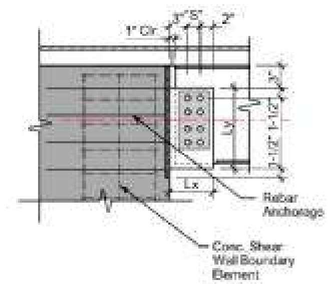
Embed Plate:	Height =	24 in	
	Width =	8 in	
	t =	1/2 in	
	F_y =	36 ksi	*(A36 Steel)*
	F_U =	58 ksi	*(A36 Steel)*
Shear Plate:	L_X =	5 in	
	L_Y =	19.5 in	
	t =	3/8 in	
	Weld to Embed Plate =	3/16 in	
	F_y =	36 ksi	*(A36 Steel)*
	F_U =	58 ksi	*(A36 Steel)*
Rebar:	Size =	#7	
	# of Bars per Row =	2	
	# of Rows =	4	
	Vertical Spacing =	6.5 in	
	F_y (Rebar)=	60 ksi	*(A706 Steel)*
Welds:	D =	5/16 in	
	Lwy	19.5 in	
	Lwx	8 in	



BOLT: SINGLE ROW



WELDED



BOLT: DOUBLE ROW

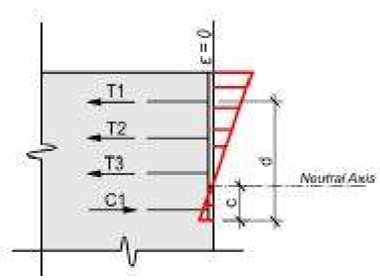
Beam Size:	W24X84
	tf = 0.77
	tw = 0.47
	bf = 9.02
	d = 24.1
	Ag = 24.7

APPLIED LOADS

Drag Load =	**125.7** kips	*(ASD)*
Gravity Load =	**27.9** kips	*(LRFD)*

$$\text{Drag Tensile Load} = \frac{\text{Drag Load}}{0.7*(\#\text{ Of Bars})}$$

Drag Tensile Load =	22.4 kips	*(LRFD/Per Anchor)*
Gravity Tensile Load =	2.5 kips	*(LRFD/Per Anchor)*
Total Tensile Load =	24.9 kips	*(LRFD/Per Anchor)*
Gravity Shear Load =	3.5 kips	*(LRFD/Per Anchor)*



RESULTANT LOAD

$$R_U = \sqrt{(V_U^2 + N_U^2)}$$

R_U = 181.73 kips

Θ = 81.17

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REBAR DESIGN

TENSILE CAPACITY OF SINGLE BAR

A_s = 0.6 in^2

Φ = 0.75 *(ACI 318-19 17.5.3a)*

$ΦN_N$ = 27.0 kips

SHEAR CAPACITY OF SINGLE BAR

A_s = 0.6 in^2

Φ = 0.65 *(ACI 318-19 17.5.3a)*

$ΦV_N$ = 23.4 kips

TENSION & SHEAR INTERACTION

N_{UA} = 24.9 kips

$N_{UA}/ΦN_N$ = 0.92

V_{UA} = 3.5 kips

$V_{UA}/ΦV_N$ = 0.15

$(N_{UA}/ΦN_N) + (V_{UA}/ΦV_N)$ = **1.07** <1.2

EMBED PLATE DESIGN

EMBED PLATE THICKNESS

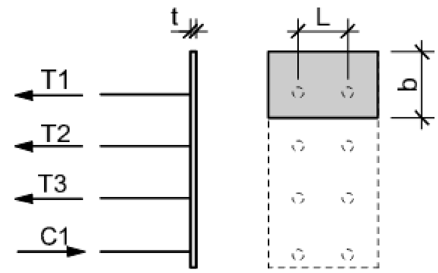
L = 2 in

b = 6.25 in

T1 = 49.8 kips *(LRFD)*

$M_{pl} = \frac{T1 \cdot L}{8}$ *(AISC 15th Ed. 3-23.16)*

M_{pl} = 12.45 K-in



Flexure Yield:

$Z = \frac{b \cdot t^2}{4}$

Z = 0.39 in^3

$\phi M_n = \phi F_y Z$ *(AISC 15th Ed. F11-1)*

Φ = 0.9

$ΦM_N$ = 12.66 K-in

DCR = **0.98** <1

Shear Yield:

A_{gv} = 10.5 in^2

$\phi R_n = \phi\, 0.6 F_y A_{gv}$ *(AISC 15th Ed. J4-3)*

Φ = 0.75

$Φ R_n$ = 170.1 kips

DCR = **0.16** <1

STRENGTH OF WELD

$\mu = 1.0 + 0.5 \sin^{1.5}\theta$ *(AISC 15th Ed. J2-5)*

Θ = 81.17

μ = 1.49

$R_n = (1.392\,\text{kip/in})\, D l\, \mu\, (2\, sides)$ *(AISC 15th Ed. 8-2a)*

$R_n =$ 242.85 kips

DCR = **0.75** <1

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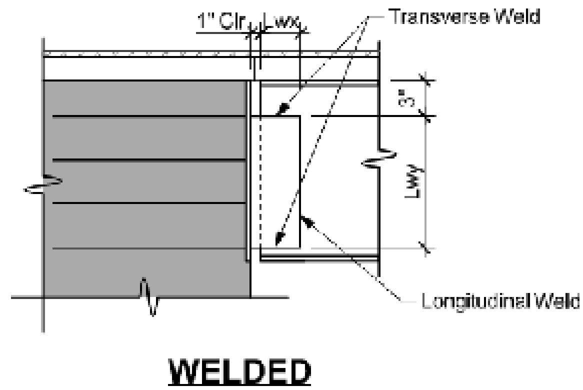
STRENGTH OF WELDED CONN.



RESULTANT LOAD

$$R_U = \sqrt{(V_U^2 + N_U^2)}$$

R_U = 181.73 kips

Θ = 81.17

WELD STRENGTH

$\frac{kl}{l}$ = $\frac{8}{19.5}$ = k = 0.41

x = 0.09

xl = 1.81 in

e_x = 7.19 in

$\frac{e_x}{l}$ = a = 0.37

C = 3.62

$$\phi R_n = \phi C C_1 Dl$$ *(AISC 15th Ed. 8-21)*

Φ = 0.75

C_1 = 1 *(AISC 15th Ed. T.8-3)*

Gravity Load: $\phi R_n =$ 264.53 kips

DCR = **0.11** <1

Drag Load: $\phi R_n =$ 217.05

DCR = **0.83** <1

BEAM CHECKS

Shear Rupture of Beam Web: $t_{min} = \frac{3.09\,D}{F_U}$ *(AISC 15th Ed. 8-21)*

t_{MIN} = 0.01 in

DCR = **0.03** <1

Shear Yielding: A_{gv} = 11.33 in²

$$\phi R_n = \phi 0.6 F_y A_{gv}$$ *(AISC 15th Ed. J4-3)*

Φ = 1

Φ R_n = 339.81 kips

DCR = **0.08** <1

Tensile Yielding: A_g = 24.7 in²

$$\phi R_n = \phi F_y A_g$$ *(AISC 15th Ed. J4-1)*

Φ = 0.9

Φ R_n = 1111.5 kips

DCR = **0.16** <1

Tensile Rupture:

A_n = 24.7 in^2

U = 1 - (x-bar/l)

$$\bar{x} = \frac{2b_f^2 t_f + t_w^2 (d - 2t_f)}{8b_f t_f + 4t_w (d - 2t_f)}$$

x_bar = 1.33 in

U = 0.83

$\phi R_n = \phi F_u A_e$ (AISC 15th Ed. J4-2)

Φ = 0.75

ΦR_n = 1003.98 kips

DCR = **0.18** <1

Block Shear Rupture:

A_{gv} = 7.52 in^2

A_{nv} = 7.52 in^2

A_{nt} = 9.17 in^2

U_{bs} = 1

Φ = 0.75

$\phi R_n = \phi 0.6 F_u A_{nv} + U_{bs} F_u A_{nt} \leq \phi 0.6 F_y A_{gv} + U_{bs} F_u A_{nt}$ (AISC 15th Ed. J4-5)

$\phi 0.6 F_y A_{gv} + U_{bs} F_u A_{nt} =$ 764.93 kips

$\phi 0.6 F_u A_{nv} + U_{bs} F_u A_{nt} =$ 815.69 kips

$\phi R_n =$ 764.93 kips

DCR = **0.16** <1

PLATE CHECKS

Shear Rupture of Plate:

$$t_{min} = \frac{3.09 D}{F_U}$$ (AISC 15th Ed. 8-21)

t_{MIN} = 0.02 in

DCR = **0.04** <1

Flexure Yield: $\phi M_n = \phi F_y Z$ (AISC 15th Ed. F11-1)

$$Z = \frac{t*D^2}{4}$$

Z = 35.65 in^3

Φ = 0.9

$\phi M_n =$ 1155.01 k-in

$\phi R_n =$ 160.59 kips

DCR = **0.17** <1

Lateral-Torsional Buckling:

$\frac{0.08E}{F_y}$ = 64.44

$\frac{1.9E}{F_y}$ = 1530.56

$\frac{L_b d}{t^2}$ = 1109.33

$\frac{0.08E}{F} < \frac{L_b d}{t^2} < \frac{1.9E}{F}$ So use AISC 15th Ed. F11-2b

F_y t^2 F_y

$$M_N = C_b[1.52 - 0.274(\frac{L_b d}{t^2})\frac{F_y}{E}]M_y$$ *(AISC 15th Ed. F11-2)*

$$C_b = \frac{12.5M_{max}}{2.5M_{max} + 3M_A + 4M_B + 3M_C}$$ *(AISC 15th Ed. F1-1)*

M_y = 1283.34 kip-in
C_b = 1.67
Φ = 0.9
ϕM_n = 2199.66 kip-in
ϕR_n = 305.83 kips

DCR = **0.09** <1

Shear Yielding:

A_{gv} = 7.31 in^2
$\phi R_n = \phi 0.6 F_y A_{gv}$ *(AISC 15th Ed. J4-3)*
Φ = 1
ϕR_n = 157.95 kips

DCR = **0.18** <1

Tensile Yielding:

A_g = 7.31 in^2
$\phi R_n = \phi F_y A_g$ *(AISC 15th Ed. J4-1)*
Φ = 0.9
ϕR_n = 236.93 kips

DCR = **0.76** <1

Interaction of Axial, Flexure and Shear Yielding in Plate:

N_U = 179.6 kips
V_U = 27.9 kips
ϕR_{np} = 236.93 kips
ϕR_{nv} = 157.95 kips
ϕM_n = 1155.01 kip-in

$\frac{N_u}{\Phi R_{np}}$ = 0.76 >0.2 So use AISC 15th Ed. H1-1a

$$[\frac{N_u}{\Phi R_{np}} + \frac{8}{9}(\frac{V_u a}{\Phi M_n})]^2 + [\frac{V_u}{\Phi R_{nv}}]^2 =$$ **0.86** <1 *(AISC 15th Ed. H1-1a)*

$$\left(\frac{N_u}{2\phi R_{np}} + \frac{V_u a}{\phi M_n}\right)^2 + \left(\frac{V_u}{\phi R_{nv}}\right)^2 =$$ **N/A** >1 *(AISC 15th Ed. H1-1b)*

Flexure Rupture:

$$Z = \frac{t*D^2}{4}$$

Z_{net} = 35.65 in^3
Φ = 0.75
ϕM_n = 1550.71 k-in
ϕR_n = 215.6

DCR = **0.13** <1

Shear Rupture:

A_{nv} = 7.31 in^2
$\phi R_n = \phi 0.6 F_u A_n$ *(AISC 15th Ed. J4-4)*
Φ = 0.75
ϕR_n = 190.86 kips

DCR = **0.15** <1

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Tensile Rupture:

A_n = 7.31 in^2

U = 1

$\phi R_n = \phi F_u A_e$ *(AISC 15th Ed. J4-2)*

Φ = 0.75

$\phi R_n =$ 318.09 kips

DCR = **0.56** <1

Interaction of Axial, Flexure and Shear Rupture in Plate:

N_U = 179.6 kips

V_U = 27.9 kips

$\phi R_{np} =$ 318.09 kips

$\phi R_{nv} =$ 190.86 kips

$\phi M_n =$ 1550.71 kip-in

$\frac{N_u}{\Phi R_{np}} =$ 0.56 >0.2 So use AISC 15th Ed. H1-1a

$[\frac{N_u}{\Phi R_{np}} + \frac{8}{9}(\frac{V_u a}{\Phi M_n})]^2 + [\frac{V_u}{\Phi R_{nv}}]^2 =$ **0.48** <1 *(AISC 15th Ed. H1-1a)*

$\left(\frac{N_u}{2\phi R_{np}} + \frac{V_u a}{\phi M_n}\right)^2 + \left(\frac{V_u}{\phi R_{nv}}\right)^2 =$ **N/A** >1 *(AISC 15th Ed. H1-1b)*