STEEL TO CONCRETE CONNECTIONS

Senior Project:
Spring 2021

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California Polytechnic State University: Architectural Engineering 1/103
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.

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,
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.

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
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Dylan Thompson

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 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.

Sketch 1: Concept Hand-drawn Sketch

<table>
<thead>
<tr>
<th>Nominal Beam Size</th>
<th>D</th>
<th># of Bolts</th>
<th>Bolt Size, Grade, &amp; Conn. Type</th>
<th>Required Plate Thickness</th>
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<tr>
<td>2&quot; x 2&quot;</td>
<td>2</td>
<td>2</td>
<td>5/8 A325 Type X</td>
<td>1/4</td>
</tr>
<tr>
<td>4&quot; x 4&quot;</td>
<td>3</td>
<td>3</td>
<td>7/8 A325 Type X</td>
<td>1/4</td>
</tr>
<tr>
<td>6&quot; x 6&quot;</td>
<td>3</td>
<td>3</td>
<td>7/8 A325 Type X</td>
<td>1/4</td>
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<tr>
<td>10&quot; x 10&quot;</td>
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<td>4</td>
<td>7/8 A325 Type X</td>
<td>3/8</td>
</tr>
<tr>
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<td>6</td>
<td>6</td>
<td>7/8 A325 Type X</td>
<td>3/8</td>
</tr>
<tr>
<td>20&quot; x 20&quot;</td>
<td>9</td>
<td>9</td>
<td>1/2 A325 Type X</td>
<td>1/2</td>
</tr>
</tbody>
</table>

NOTE: Steel Beam and Plate, Table Size Refers to End of Beam in Connection
NOTE: All Notes and Dimensions to Be IN
NOTE: Steel Beams May Have EGP Size - Coordinate w/Arch.
NOTE: Use 1/16" Thick Plate Ref. 7-14
NOTE: Bolt Spacing Based on Depth, "D", 3" Min.

TYP STEEL BEAM CONNECTIONS
**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.

<table>
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<tr>
<th>Redundancy Factor (ρ)</th>
<th>1.0</th>
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</thead>
<tbody>
<tr>
<td>Seismic Design Category</td>
<td>E</td>
</tr>
<tr>
<td>Spectral Response Coefficients (S₀ₛ, S₀₁)</td>
<td>1.566, 0.728</td>
</tr>
<tr>
<td>Design Wind Pressure</td>
<td>52.5 psf</td>
</tr>
<tr>
<td>Response Modification Coefficient (R)</td>
<td>5 (Special Reinforced Conc. S.W.) 6.5 (Light Framed Wd. S.W. w/ Plywood)</td>
</tr>
<tr>
<td>Design Base Shear (1.0E)</td>
<td>633.1 kips</td>
</tr>
</tbody>
</table>

**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 submission. 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.

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.

**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
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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

<table>
<thead>
<tr>
<th>Anchor</th>
<th>Tension load, ( F_t ) (lb)</th>
<th>Shear load, ( V_{x,\text{in}} ) (lb)</th>
<th>Shear load, ( V_{y,\text{in}} ) (lb)</th>
<th>Shear load combined, ( (V_{x,\text{in}})^2 + (V_{y,\text{in}})^2 ) (lb)</th>
</tr>
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<tbody>
<tr>
<td>1</td>
<td>4663.4</td>
<td>0.0</td>
<td>12287.5</td>
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</tr>
<tr>
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<td>3</td>
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Maximum concrete compression strain (\( \epsilon_{\text{c}} \)) 0.07
Maximum concrete compression stress (psi) 2100
Resultant tension force (lb): 0
Resultant compression force (lb) 1500
Ecocentricity of resultant tension forces in x-axis, \( \epsilon_{\text{x,x}} \): 0.00
Ecocentricity of resultant tension forces in y-axis, \( \epsilon_{\text{y,y}} \): 0.00
Ecocentricity of resultant shear forces in x-axis, \( \epsilon_{\text{x,s}} \): 0.00
Ecocentricity of resultant shear forces in y-axis, \( \epsilon_{\text{y,s}} \): 0.00

**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.
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
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

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.

**Site Visit:**

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

**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
Steel Gravity Bm. Key Plan w/ Bm. Member ID's and Locations of Max Loads for Calcs

Max W16 Load
Max W18 Load
Max W21 Load
Max W24 Load
W14 Max Load
# 1. Project Information

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<th>Project Description:</th>
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<table>
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<table>
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# 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
- **Reinforcement provided at corners:** Yes
- **Ignore concrete breakout in tension:** Yes
- **Ignore concrete breakout in shear:** Yes
- **Ignore 6d 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

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

---

Input data and results must be checked for agreement with the existing circumstances, the standards and guidelines must be checked for plausibility.

Simpson Strong-Tie Company Inc.  5956 W. Las Positas Boulevard  Pleasanton, CA 94588  Phone: 925.560.9000  Fax: 925.847.3871  www.strongtie.com
**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:

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

*<Figure 1>*
Input data and results must be checked for agreement with the existing circumstances, the standards and guidelines must be checked for plausibility.
### 3. Resulting Anchor Forces

<table>
<thead>
<tr>
<th>Anchor</th>
<th>Tension load, ( N_a ) (lb)</th>
<th>Shear load ( V_{ax} ), ( V_{ay} ) (lb)</th>
<th>Shear load combined, ( \sqrt{(V_{ax})^2+(V_{ay})^2} ) (lb)</th>
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<tbody>
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<td>1</td>
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<tr>
<td>Sum</td>
<td>12718.8</td>
<td>0.0</td>
<td>46890.0</td>
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</table>

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, \( e'_{N_x} \) (inch): 0.00
Eccentricity of resultant tension forces in y-axis, \( e'_{N_y} \) (inch): 0.00
Eccentricity of resultant shear forces in x-axis, \( e'_{V_x} \) (inch): 0.00
Eccentricity of resultant shear forces in y-axis, \( e'_{V_y} \) (inch): 0.00

![Figure 3]

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

\[
N_{sa} = \phi N_a
\]

<table>
<thead>
<tr>
<th>( N_{sa} ) (lb)</th>
<th>( \phi )</th>
<th>( \phi N_{sa} ) (lb)</th>
</tr>
</thead>
<tbody>
<tr>
<td>26950</td>
<td>0.75</td>
<td>20213</td>
</tr>
</tbody>
</table>

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

\[
\phi N_{pn} = \phi V_{sc} N_p = \phi V_{sc} A_{brg} f'_{c} \quad (Sec. 17.3.1, Eq. 17.4.3.1 & 17.4.3.4)
\]

\[
\begin{align*}
\phi & = 0.70 \\
A_{brg} & = 4000 \\
f'_{c} & = 24618
\end{align*}
\]

Input data and results must be checked for agreement with the existing circumstances, the standards and guidelines must be checked for plausibility.

Simpson Strong-Tie Company Inc.  
5956 W. Las Positas Boulevard  Pleasanton, CA 94588  Phone: 925.560.9000  Fax: 925.847.3871  www.strongtie.com
8. Steel Strength of Anchor in Shear (Sec. 17.5.1)

<table>
<thead>
<tr>
<th>$V_{as}$ (lb)</th>
<th>$\phi_{out}$</th>
<th>$\phi$</th>
<th>$\phi_{out}V_{as}$ (lb)</th>
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<tbody>
<tr>
<td>26950</td>
<td>1.0</td>
<td>0.65</td>
<td>17518</td>
</tr>
</tbody>
</table>

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

$$\phi_{cpg} = \frac{k_{cp}}{A_{Nc}}$$

<table>
<thead>
<tr>
<th>$k_{cp}$</th>
<th>$A_{Nc}$ (in$^2$)</th>
<th>$A_{Nco}$ (in$^2$)</th>
<th>$\psi_{c,N}$</th>
<th>$\psi_{ed,N}$</th>
<th>$\psi_{c,N}$</th>
<th>$\psi_{ed,N}$</th>
<th>$N_{c}$ (lb)</th>
<th>$\phi$</th>
<th>$\phi_{cpg}$ (lb)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.0</td>
<td>367.25</td>
<td>153.14</td>
<td>1.000</td>
<td>0.894</td>
<td>1.250</td>
<td>1.000</td>
<td>12717</td>
<td>0.70</td>
<td>47709</td>
</tr>
</tbody>
</table>

11. Results

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

<table>
<thead>
<tr>
<th></th>
<th>Factored Load, $N_{ua}$ (lb)</th>
<th>Design Strength, $\phi N_{c}$ (lb)</th>
<th>Ratio</th>
<th>Status</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tension</td>
<td></td>
<td></td>
<td></td>
<td>Pass (Governs)</td>
</tr>
<tr>
<td>Steel</td>
<td>4382</td>
<td>20213</td>
<td>0.22</td>
<td>Pass</td>
</tr>
<tr>
<td>Pullout</td>
<td>4382</td>
<td>24618</td>
<td>0.18</td>
<td></td>
</tr>
<tr>
<td>Shear</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Steel</td>
<td>7815</td>
<td>17518</td>
<td>0.45</td>
<td>Pass</td>
</tr>
<tr>
<td>Pryout</td>
<td>46890</td>
<td>47709</td>
<td>0.98</td>
<td>Pass (Governs)</td>
</tr>
</tbody>
</table>

Interaction check

<table>
<thead>
<tr>
<th>Sec. 17.6..3</th>
<th>$N_{ua}/\phi N_{c}$</th>
<th>$V_{as}/\phi V_{a}$</th>
<th>Combined Ratio</th>
<th>Permissible</th>
<th>Status</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.22</td>
<td>0.98</td>
<td>120.0%</td>
<td>1.2</td>
<td>Pass</td>
</tr>
</tbody>
</table>

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.
1. Project Information

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

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_eff (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
- **Ψ_{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 6d 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:

\[ N_u \ [\text{lb}] : 0 \]
\[ V_{uax} \ [\text{lb}] : 0 \]
\[ V_{uay} \ [\text{lb}] : 59350 \]
\[ M_{ux} \ [\text{ft-lb}] : -14830 \]
\[ M_{uy} \ [\text{ft-lb}] : 0 \]
\[ M_{uz} \ [\text{ft-lb}] : 0 \]

<Figure 1>
Input data and results must be checked for agreement with the existing circumstances, the standards and guidelines must be checked for plausibility.
3. Resulting Anchor Forces

<table>
<thead>
<tr>
<th>Anchor</th>
<th>Tension load, $N_{sa}$ (lb)</th>
<th>Shear load x, $V_{uax}$ (lb)</th>
<th>Shear load y, $V_{uay}$ (lb)</th>
<th>Shear load combined, $\sqrt{(V_{uax})^2 + (V_{uay})^2}$ (lb)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>4754.8</td>
<td>0.0</td>
<td>9891.7</td>
<td>9891.7</td>
</tr>
<tr>
<td>2</td>
<td>2221.9</td>
<td>0.0</td>
<td>9891.7</td>
<td>9891.7</td>
</tr>
<tr>
<td>3</td>
<td>0.0</td>
<td>0.0</td>
<td>9891.7</td>
<td>9891.7</td>
</tr>
<tr>
<td>4</td>
<td>0.0</td>
<td>0.0</td>
<td>9891.7</td>
<td>9891.7</td>
</tr>
<tr>
<td>5</td>
<td>2221.9</td>
<td>0.0</td>
<td>9891.7</td>
<td>9891.7</td>
</tr>
<tr>
<td>6</td>
<td>4754.8</td>
<td>0.0</td>
<td>9891.7</td>
<td>9891.7</td>
</tr>
<tr>
<td>Sum</td>
<td>13953.4</td>
<td>0.0</td>
<td>59350.0</td>
<td>59350.0</td>
</tr>
</tbody>
</table>

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

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

$$N_{sa} \begin{array}{c} \phi \\ \phi N_{sa} \end{array} \begin{array}{c} \phi N_{sa} \end{array}$$

26950 0.75 20213

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

$$\phi N_{p} = \phi V_{c} \cdot c_{p} = \phi V_{c} \cdot A_{brg} f'_{c} \quad (Sec. 17.3.1, Eq. 17.4.3.1 & 17.4.3.4)$$

$$\begin{array}{c}
\phi \\
A_{brg} \quad f'_{c} \quad \phi \quad \phi N_{p}
\end{array}$$

1.4 0.79 4000 0.70 24618

Input data and results must be checked for agreement with the existing circumstances, the standards and guidelines must be checked for plausibility.
8. Steel Strength of Anchor in Shear (Sec. 17.5.1)

\[
V_{sa} (lb) \quad \phi_{grout} \quad \phi \quad \phi_{grout}V_{sa} (lb)
\]

<p>| | | | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>26950</td>
<td>1.0</td>
<td>0.65</td>
<td>17518</td>
</tr>
</tbody>
</table>

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

\[
\phi N_{cp} = \phi k_p N_{cp} = \phi k_p (A_{nc} / A_{nco}) \frac{\psi}{\psi_{ec,N}} \left( \psi_{ed,N} N_c, N_p, N_e \right) (Sec. 17.3.1 & Eq. 17.5.3.1b)
\]

<table>
<thead>
<tr>
<th>(k_p)</th>
<th>(A_{nc}) (in²)</th>
<th>(A_{nco}) (in²)</th>
<th>(\psi_{ec,N})</th>
<th>(\psi_{ed,N})</th>
<th>(\psi_{N_c})</th>
<th>(\psi_{N_p})</th>
<th>(\psi_{N_e})</th>
<th>(N_c) (lb)</th>
<th>(\phi)</th>
<th>(\phi N_{cp}) (lb)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.0</td>
<td>473.69</td>
<td>153.14</td>
<td>1.000</td>
<td>0.894</td>
<td>1.250</td>
<td>1.000</td>
<td>12717</td>
<td>0.70</td>
<td>61536</td>
<td></td>
</tr>
</tbody>
</table>

11. Results

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

<table>
<thead>
<tr>
<th></th>
<th>Factored Load, (N_{ua}) (lb)</th>
<th>Design Strength, (\phi N_c) (lb)</th>
<th>Ratio</th>
<th>Status</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel</td>
<td>4755</td>
<td>20213</td>
<td>0.24</td>
<td>Pass (Governs)</td>
</tr>
<tr>
<td>Pullout</td>
<td>4755</td>
<td>24618</td>
<td>0.19</td>
<td>Pass</td>
</tr>
<tr>
<td>Steel</td>
<td>9892</td>
<td>17518</td>
<td>0.56</td>
<td>Pass</td>
</tr>
<tr>
<td>Pryout</td>
<td>59350</td>
<td>61536</td>
<td>0.96</td>
<td>Pass (Governs)</td>
</tr>
</tbody>
</table>

Interaction check

- \(N_{ua}/\phi N_c\) = 0.24
- \(V_{ua}/\phi V_c\) = 0.96

Combined Ratio = 120.0%

Status = 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.
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:

\[ N_u [\text{lb}]: 0 \]
\[ V_u [\text{lb}]: 0 \]
\[ V_{uax} [\text{lb}]: 78688 \]
\[ M_u [\text{ft}-\text{lb}]: -19672 \]
\[ M_{ux} [\text{ft}-\text{lb}]: 0 \]
\[ M_{uy} [\text{ft}-\text{lb}]: 0 \]

<Figure 1>
Input data and results must be checked for agreement with the existing circumstances, the standards and guidelines must be checked for plausibility.
3. Resulting Anchor Forces

<table>
<thead>
<tr>
<th>Anchor</th>
<th>Tension load, $N_{sa}$ (lb)</th>
<th>Shear load $V_{ux}$, $V_{uy}$ (lb)</th>
<th>Shear load combined, $\sqrt{(V_{ux})^2+(V_{uy})^2}$ (lb)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>4788.1</td>
<td>0.0</td>
<td>13114.7</td>
</tr>
<tr>
<td>2</td>
<td>2221.4</td>
<td>0.0</td>
<td>13114.7</td>
</tr>
<tr>
<td>3</td>
<td>0.0</td>
<td>0.0</td>
<td>13114.7</td>
</tr>
<tr>
<td>4</td>
<td>0.0</td>
<td>0.0</td>
<td>13114.7</td>
</tr>
<tr>
<td>5</td>
<td>2221.4</td>
<td>0.0</td>
<td>13114.7</td>
</tr>
<tr>
<td>6</td>
<td>4788.1</td>
<td>0.0</td>
<td>13114.7</td>
</tr>
<tr>
<td>Sum</td>
<td>14019.1</td>
<td>0.0</td>
<td>78688.0</td>
</tr>
</tbody>
</table>

Maximum concrete compression strain ($\varepsilon_{con}$): 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

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

\[ N_{sa} (lb) \quad \phi \quad \phi N_{sa} (lb) \]

\[
\begin{array}{c|ccc}
26950 & 0.75 & 20213 \\
\end{array}
\]

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

\[ \phi N_{pl} = \phi V_{pl} N_{pl} = \phi V_{pl} A_{pl} f'_{c} \quad (Sec. 17.4.4.1, Eq. 17.4.3.1 & 17.4.3.4) \]

\[
\begin{array}{c|cccc}
\phi_{pl} & A_{pl} (in^2) & f'_{c} (psi) & \phi & \phi N_{pl} (lb) \\
1.4 & 0.79 & 4000 & 0.70 & 24618 \\
\end{array}
\]

Input data and results must be checked for agreement with the existing circumstances, the standards and guidelines must be checked for plausibility.
8. Steel Strength of Anchor in Shear (Sec. 17.5.1)

<table>
<thead>
<tr>
<th>$V_{sa}$ (lb)</th>
<th>$\phi_{grout}$</th>
<th>$\phi$</th>
<th>$\phi_{grout}V_{sa}$ (lb)</th>
</tr>
</thead>
<tbody>
<tr>
<td>26950</td>
<td>1.0</td>
<td>0.65</td>
<td>17518</td>
</tr>
</tbody>
</table>

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

\[
\phi V_{cpg} = \phi N_{cp} = \phi \left( A_{nc} / A_{nc0} \right) \gamma_{c,n} \gamma_{c,N} N_{c,N} N_{b} \text{ (Sec. 17.3.1 & Eq. 17.5.3.1b)}
\]

<table>
<thead>
<tr>
<th>$k_{cp}$</th>
<th>$A_{nc}$ (in$^2$)</th>
<th>$A_{nc0}$ (in$^2$)</th>
<th>$\gamma_{c,N}$</th>
<th>$\gamma_{c,n}$</th>
<th>$N_{c,N}$</th>
<th>$N_{b}$ (lb)</th>
<th>$\phi$</th>
<th>$\phi V_{cpg}$ (lb)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.0</td>
<td>629.00</td>
<td>153.14</td>
<td>1.00</td>
<td>0.894</td>
<td>1.250</td>
<td>1000</td>
<td>0.70</td>
<td>81712</td>
</tr>
</tbody>
</table>

11. Results

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

<table>
<thead>
<tr>
<th>Tension</th>
<th>Factored Load, $N_{ua}$ (lb)</th>
<th>Design Strength, $\phi N_{u}$ (lb)</th>
<th>Ratio</th>
<th>Status</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel</td>
<td>4788</td>
<td>20213</td>
<td>0.24</td>
<td>Pass (Governs)</td>
</tr>
<tr>
<td>Pullout</td>
<td>4788</td>
<td>24618</td>
<td>0.19</td>
<td>Pass</td>
</tr>
<tr>
<td>Shear</td>
<td>Factored Load, $V_{sa}$ (lb)</td>
<td>Design Strength, $\phi V_{s}$ (lb)</td>
<td>Ratio</td>
<td>Status</td>
</tr>
<tr>
<td>Steel</td>
<td>13115</td>
<td>17518</td>
<td>0.75</td>
<td>Pass</td>
</tr>
<tr>
<td>Pryout</td>
<td>78688</td>
<td>81712</td>
<td>0.96</td>
<td>Pass (Governs)</td>
</tr>
</tbody>
</table>

Interaction check

<table>
<thead>
<tr>
<th>$N_{ua}/\phi N_{u}$</th>
<th>$V_{sa}/\phi V_{s}$</th>
<th>Combined Ratio</th>
<th>Permissible</th>
<th>Status</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.24</td>
<td>0.96</td>
<td>120.0%</td>
<td>1.2</td>
<td>Pass</td>
</tr>
</tbody>
</table>

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.
1. Project Information

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

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 6d 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

---

Input data and results must be checked for agreement with the existing circumstances, the standards and guidelines must be checked for plausibility.
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
Ω₀ factor: not set
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: No

Strength level loads:
Nₜ [lb]: 0
Vᵥₓ [lb]: 0
Vᵥᵧ [lb]: 71000
Mᵥₓ [ft-lb]: 17750
Mᵥᵧ [ft-lb]: 0
Mᵥz [ft-lb]: 0

<Figure 1>
Input data and results must be checked for agreement with the existing circumstances, the standards and guidelines must be checked for plausibility.
3. Resulting Anchor Forces

<table>
<thead>
<tr>
<th>Anchor</th>
<th>Tension load, $N_{as}$ (lb)</th>
<th>Shear load $x$, $V_{ux}$ (lb)</th>
<th>Shear load $y$, $V_{uy}$ (lb)</th>
<th>Shear load combined, $\sqrt{(V_{ux})^2+(V_{uy})^2}$ (lb)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.0</td>
<td>11833.3</td>
<td>11833.3</td>
<td>11833.3</td>
</tr>
<tr>
<td>2</td>
<td>1980.1</td>
<td>0.0</td>
<td>11833.3</td>
<td>11833.3</td>
</tr>
<tr>
<td>3</td>
<td>4369.8</td>
<td>0.0</td>
<td>11833.3</td>
<td>11833.3</td>
</tr>
<tr>
<td>4</td>
<td>4369.8</td>
<td>0.0</td>
<td>11833.3</td>
<td>11833.3</td>
</tr>
<tr>
<td>5</td>
<td>1980.1</td>
<td>0.0</td>
<td>11833.3</td>
<td>11833.3</td>
</tr>
<tr>
<td>6</td>
<td>0.0</td>
<td>0.0</td>
<td>11833.3</td>
<td>11833.3</td>
</tr>
<tr>
<td>Sum</td>
<td>12699.9</td>
<td>0.0</td>
<td>71000.0</td>
<td>71000.0</td>
</tr>
</tbody>
</table>

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

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

<table>
<thead>
<tr>
<th>$N_{as}$ (lb)</th>
<th>$\phi$</th>
<th>$\phi N_{as}$ (lb)</th>
</tr>
</thead>
<tbody>
<tr>
<td>26950</td>
<td>0.75</td>
<td>20213</td>
</tr>
</tbody>
</table>

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

<table>
<thead>
<tr>
<th>$q_{f,\gamma}$</th>
<th>$A_{brg}$ (in$^2$)</th>
<th>$f'_c$ (psi)</th>
<th>$\phi$</th>
<th>$0.75 q_{f,\gamma} N_{as}$ (lb)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.4</td>
<td>0.79</td>
<td>4000</td>
<td>0.70</td>
<td>18463</td>
</tr>
</tbody>
</table>
8. Steel Strength of Anchor in Shear (Sec. 17.5.1)

<table>
<thead>
<tr>
<th>$V_{sa}$ (lb)</th>
<th>$\phi_{prout}$</th>
<th>$\phi$</th>
<th>$\phi_{prout}V_{sa}$ (lb)</th>
</tr>
</thead>
<tbody>
<tr>
<td>26950</td>
<td>1.0</td>
<td>0.65</td>
<td>17518</td>
</tr>
</tbody>
</table>

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

$$\phi V_{cpg} = \phi k_{cp} N_{cpg} = \phi k_{cp} \left( A_{nc} / A_{nco} \right) \psi_{nc} \psi_{nco} N_{cpg} N_{cpg} \psi_{nc} \psi_{nco} N_{cpg} \psi_{nc} \psi_{nco} N_{cpg}$$ (Sec. 17.3.1 & Eq. 17.5.3.1b)

<table>
<thead>
<tr>
<th>$k_{cp}$</th>
<th>$A_{nc}$ (in$^2$)</th>
<th>$A_{nco}$ (in$^2$)</th>
<th>$\psi_{nc}$</th>
<th>$\psi_{nco}$</th>
<th>$\psi_{nc}$</th>
<th>$\psi_{nco}$</th>
<th>$N_c$ (lb)</th>
<th>$\phi$</th>
<th>$\phi V_{cpg}$ (lb)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.0</td>
<td>569.63</td>
<td>153.14</td>
<td>1.000</td>
<td>0.894</td>
<td>1.250</td>
<td>1.000</td>
<td>12717</td>
<td>0.70</td>
<td>73999</td>
</tr>
</tbody>
</table>

11. Results

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

<table>
<thead>
<tr>
<th></th>
<th>Factored Load, $N_{ua}$ (lb)</th>
<th>Design Strength, $\varepsilon N_{u}$ (lb)</th>
<th>Ratio</th>
<th>Status</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tension</td>
<td>Steel</td>
<td>4370</td>
<td>20213</td>
<td>0.22</td>
</tr>
<tr>
<td></td>
<td>Pullout</td>
<td>4370</td>
<td>18463</td>
<td>0.24</td>
</tr>
<tr>
<td>Shear</td>
<td>Steel</td>
<td>11833</td>
<td>17518</td>
<td>0.68</td>
</tr>
<tr>
<td></td>
<td>Pryout</td>
<td>71000</td>
<td>73999</td>
<td>0.96</td>
</tr>
</tbody>
</table>

Interaction check

<table>
<thead>
<tr>
<th></th>
<th>$N_{ua}/\varepsilon N_{u}$</th>
<th>$V_{ua}/\varepsilon V_{u}$</th>
<th>Combined Ratio</th>
<th>Permissible</th>
<th>Status</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sec. 17.6.3</td>
<td>0.24</td>
<td>0.96</td>
<td>119.6%</td>
<td>1.2</td>
<td>Pass</td>
</tr>
</tbody>
</table>

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.
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 6d 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
 Q0 factor: not set
 Apply entire shear load at front row: No
 Anchors only resisting wind and/or seismic loads: No

Strength level loads:

\[ N_{uw} \ [\text{lb}] : 0 \]
\[ V_{ua} \ [\text{lb}] : 0 \]
\[ V_{uy} \ [\text{lb}] : 101000 \]
\[ M_{ux} \ [\text{ft-lb}] : 25250 \]
\[ M_{uy} \ [\text{ft-lb}] : 0 \]
\[ M_{uz} \ [\text{ft-lb}] : 0 \]

<Figure 1>
Input data and results must be checked for agreement with the existing circumstances, the standards and guidelines must be checked for plausibility.
3. Resulting Anchor Forces

<table>
<thead>
<tr>
<th>Anchor</th>
<th>Tension load, $N_a$ (lb)</th>
<th>Shear load $x$, $V_{ux}$ (lb)</th>
<th>Shear load $y$, $V_{uy}$ (lb)</th>
<th>Shear load combined, $\sqrt{V_{ux}^2+V_{uy}^2}$ (lb)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.0</td>
<td>0.0</td>
<td>12625.0</td>
<td>12625.0</td>
</tr>
<tr>
<td>2</td>
<td>589.2</td>
<td>0.0</td>
<td>12625.0</td>
<td>12625.0</td>
</tr>
<tr>
<td>3</td>
<td>1969.1</td>
<td>0.0</td>
<td>12625.0</td>
<td>12625.0</td>
</tr>
<tr>
<td>4</td>
<td>3349.1</td>
<td>0.0</td>
<td>12625.0</td>
<td>12625.0</td>
</tr>
<tr>
<td>5</td>
<td>3349.1</td>
<td>0.0</td>
<td>12625.0</td>
<td>12625.0</td>
</tr>
<tr>
<td>6</td>
<td>1969.1</td>
<td>0.0</td>
<td>12625.0</td>
<td>12625.0</td>
</tr>
<tr>
<td>7</td>
<td>589.2</td>
<td>0.0</td>
<td>12625.0</td>
<td>12625.0</td>
</tr>
<tr>
<td>8</td>
<td>0.0</td>
<td>0.0</td>
<td>12625.0</td>
<td>12625.0</td>
</tr>
</tbody>
</table>

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, $e'_{N_x}$ (inch): 0.00
Eccentricity of resultant tension forces in y-axis, $e'_{N_y}$ (inch): 0.00
Eccentricity of resultant shear forces in x-axis, $e'_{V_x}$ (inch): 0.00
Eccentricity of resultant shear forces in y-axis, $e'_{V_y}$ (inch): 0.00

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

$$N_a (lb) \quad \phi \quad \phi N_a (lb)$$

<table>
<thead>
<tr>
<th>N_a (lb)</th>
<th>$\phi$</th>
<th>$\phi N_a$ (lb)</th>
</tr>
</thead>
<tbody>
<tr>
<td>47910</td>
<td>0.75</td>
<td>35933</td>
</tr>
</tbody>
</table>

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

$$0.75\phi N_{pin} = 0.75\phi \psi_{A} \psi_{f} N_{a} = 0.75\phi \psi_{A} \psi_{f} A_{brg} f'_{c} (Sec. 17.3.1, Eq. 17.4.3.1 & 17.4.3.4)$$

$$\psi_{A} \quad A_{brg} (in^2) \quad f'_{c} (psi) \quad \phi \quad 0.75\phi N_{pin} (lb)$$

<table>
<thead>
<tr>
<th>$\psi_{A}$</th>
<th>$A_{brg}$ (in$^2$)</th>
<th>$f'_{c}$ (psi)</th>
<th>$\phi$</th>
<th>$0.75\phi N_{pin}$ (lb)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.4</td>
<td>1.29</td>
<td>4000</td>
<td>0.70</td>
<td>30317</td>
</tr>
</tbody>
</table>
8. Steel Strength of Anchor in Shear (Sec. 17.5.1)

<table>
<thead>
<tr>
<th>$V_{sa}$ (lb)</th>
<th>$\phi_{out}$</th>
<th>$\phi$</th>
<th>$\phi_{out}V_{sa}$ (lb)</th>
</tr>
</thead>
<tbody>
<tr>
<td>47910</td>
<td>1.0</td>
<td>0.65</td>
<td>31142</td>
</tr>
</tbody>
</table>

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

$$\phi_{cp} = \frac{\phi_{cp}}{\phi_{cp}} = \frac{k_{cp}A_{nc}}{k_{cp}A_{nc}N_{pc}}\phi_{nc}N_{pc}\phi_{ca}N_{ca} (Sec. 17.3.1 & Eq. 17.5.3.1b)$$

<table>
<thead>
<tr>
<th>$k_{cp}$</th>
<th>$A_{nc}$ (in$^2$)</th>
<th>$A_{no}$ (in$^2$)</th>
<th>$\phi_{nc}$N_{pc}</th>
<th>$\phi_{ca}$N_{ca}</th>
<th>$N_{pc}$ (lb)</th>
<th>$\phi$</th>
<th>$\phi_{cp}$ (lb)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.0</td>
<td>951.15</td>
<td>272.25</td>
<td>1.00</td>
<td>1.250</td>
<td>19579</td>
<td>0.70</td>
<td>101203</td>
</tr>
</tbody>
</table>

11. Results

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

<table>
<thead>
<tr>
<th></th>
<th>Factored Load, $N_{ua}$ (lb)</th>
<th>Design Strength, $\phi N_{u}$ (lb)</th>
<th>Ratio</th>
<th>Status</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel</td>
<td>3349</td>
<td>35933</td>
<td>0.09</td>
<td>Pass</td>
</tr>
<tr>
<td>Pullout</td>
<td>3349</td>
<td>30317</td>
<td>0.11</td>
<td>Pass</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th></th>
<th>Factored Load, $V_{ua}$ (lb)</th>
<th>Design Strength, $\phi V_{u}$ (lb)</th>
<th>Ratio</th>
<th>Status</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel</td>
<td>12625</td>
<td>31142</td>
<td>0.41</td>
<td>Pass</td>
</tr>
<tr>
<td>Pryout</td>
<td>101000</td>
<td>101203</td>
<td>1.00</td>
<td>Pass</td>
</tr>
</tbody>
</table>

Interaction check

<table>
<thead>
<tr>
<th>Sec. 17.6.2</th>
<th>$N_{ua}/\phi N_{u}$</th>
<th>$V_{ua}/\phi V_{u}$</th>
<th>Combined Ratio</th>
<th>Permissible</th>
<th>Status</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.00</td>
<td>1.00</td>
<td>99.8%</td>
<td>1.00</td>
<td>Pass</td>
</tr>
</tbody>
</table>

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
Appendix B: Steel Drag Beam Calculations
## STEEL DRAG BEAM CONNECTION DESIGNATION

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

**Notes:**
- Max Load for Calculation of Conn.
STEEL DRAG BEAM CONNECTION CALCULATION

WHAT TYPE OF CONNECTION IS BEING USED?

BOLT: SINGLE ROW

MATERIAL AND CONFIGURATION

Embed Plate:  
- Height = 14 in  
- Width = 8 in  
- t = 1/2 in

Shear Plate:  
- L = 5 in  
- t = 1/4 in

Weld to Embed Plate = 3/16 in

Rebar:  
- Size = #4
- # of Bars per Row = 2
- # of Rows = 3
- Vertical Spacing = 4.75 in
- F_y (Rebar) = 60 ksi

Bolts:  
- Ø = 1 in  
- # of Rows = 3  
- Vert. Spacing = 2.45 in  
- Horiz. Spacing = 0 in  
- Fnt (Bolts) = 45 ksi  
- Fnv (Bolts) = 27 ksi

Beam Size: W14X26
- t_f = 0.42  
- t_w = 0.26  
- b_f = 5.03  
- d = 13.9
- A_g = 7.69

APPLIED LOADS

Drag Load = 15.1 kips  
Gravity Load = 21.7 kips

Drag Tensile Load = 3.6 kips  
Gravity Tensile Load = 2.4 kips  
Total Tensile Load = 6.0 kips  
Gravity Shear Load = 3.6 kips

RESULTANT LOAD

\[ R_u = \sqrt{V_u^2 + N_u^2} \]
\[ \theta = 44.83^\circ \]
REBAR DESIGN

TENSILE CAPACITY OF SINGLE BAR

\[ A_t = 0.2 \text{ in}^2 \]
\[ \phi = 0.75 \]  
\[ \Phi N_t = 9.0 \text{ kips} \]  

(SACI 318-19 17.5.3a)

SHEAR CAPACITY OF SINGLE BAR

\[ A_s = 0.2 \text{ in}^2 \]
\[ \phi = 0.65 \]  
\[ \Phi V_s = 7.8 \text{ kips} \]  

(SACI 318-19 17.5.3a)

TENSION & SHEAR INTERACTION

\[ N_s = 6.0 \text{ kips} \]
\[ V_s = 3.6 \text{ kips} \]
\[ \frac{N_s}{\phi N_t} = 0.67 \]
\[ \frac{V_s}{\phi V_s} = 0.46 \]

\[ \left(\frac{N_s}{\phi N_t}\right) + \left(\frac{V_s}{\phi V_s}\right) = 1.13 < 1.2 \]

EMBED PLATE DESIGN

EMBED PLATE THICKNESS

\[ L = 2 \text{ in} \]
\[ b = 5.38 \text{ in} \]
\[ T1 = 12.0 \text{ kips} \]  

\[ M_n = \frac{T1 \cdot t}{8} \]  
\[ M_n = 3.01 \text{ K-in} \]  

Flexure Yield:

\[ Z = \frac{b \cdot t}{4} \]  
\[ Z = 0.34 \text{ in}^2 \]  

\[ \phi M_n = \phi F_y Z \]  
\[ \phi = 0.9 \]
\[ \Phi M_n = 10.88 \text{ K-in} \]

DCR = 0.28 < 1

Shear Yield:

\[ A_w = 5.5 \text{ in}^2 \]
\[ \phi R_n = \phi 0.6 F_y A_w \]  
\[ \phi = 0.75 \]
\[ \Phi R_n = 89.1 \text{ kips} \]

DCR = 0.24 < 1

STRENGTH OF WELD

\[ \mu = 1.0 + 0.5 \sin^{1.5} \theta \]  
\[ \theta = 44.83 \]
\[ \mu = 1.3 \]

\[ R_n = (1.392 \text{ kip/in}) DL \mu \{2 \text{ sides}\} \]  
\[ R_n = 102.83 \text{ kips} \]  

DCR = 0.3 < 1
STRENGTH OF BOLTED CONN.

RESULTANT LOAD

\[ R_U = \sqrt{\left( F_U^2 + N_U^2 \right)} \]

- \( R_U = 30.6 \text{ kips} \)
- \( \Theta = 44.83 \)
- \( e = 1.5 \)
- \( C = 2.45 \)  
  \( \text{(AISC 15th Ed. T.10-9)} \)

BEAM WEB STRENGTH

- **Bolt Shear:**
  \[ \phi r_n = \phi F_u A_s \]  
  \( \Phi = 0.75 \)
  \( \Phi r_n = 15.9 \text{ kips/bolt} \)

- **Bolt Bearing Strength:**
  \[ \phi r_n = \phi 3.0 d t F_u \]  
  \( \Phi = 0.75 \)
  \( \Phi r_n = 37.29 \text{ kips/bolt} \)

- **Bolt Tearout Strength:**
  \[ \phi r_n = \phi 1.5 l t F_u \]  
  \( \Phi = 0.75 \)
  \( \Phi r_n = 26.8 \text{ kips/bolt} \)

  **Governing**
  \[ \phi R_n = 15.9 \text{ kips} \]
  \[ \Phi R_n = 38.89 \text{ kips} \]

- **DCR:** \( 0.79 < 1 \)

SHEAR PLATE STRENGTH

- **Bolt Bearing Strength:**
  \[ \phi r_n = \phi 3.0 d t F_u \]  
  \( \Phi = 0.75 \)
  \( \Phi r_n = 32.63 \text{ kips/bolt} \)

- **Bolt Tearout Strength:**
  \[ \phi r_n = \phi 1.5 l t F_u \]  
  \( \Phi = 0.75 \)
  \( \Phi r_n = 15.29 \text{ kips/bolt} \)

  **Governing**
  \[ \phi R_n = 15.29 \text{ kips} \]
  \[ \Phi R_n = 37.39 \text{ kips} \]

- **DCR:** \( 0.82 < 1 \)

PLATE CHECKS

- **Maximum Plate Thick:**
  \( t_{MAX} = \left( D_{BOLT}/2 \right) + \left( 1/16 \right) \)  
  \( t = 0.56 \text{ in} \)  
  \( \text{(AISC 15th Ed. T.10-9)} \)
STEEL DRAG BEAM CONNECTION CALCULATION

WHAT TYPE OF CONNECTION IS BEING USED?

BOLT: DOUBLE ROW  EXTENDED CONFIGURATION

MATERIAL AND CONFIGURATION

<table>
<thead>
<tr>
<th>Embed Plate:</th>
<th>Height = 14 in</th>
<th>Width = 8 in</th>
<th>t = 1/2 in</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear Plate:</td>
<td>L_y = 5 in</td>
<td>t = 1/4 in</td>
<td></td>
</tr>
<tr>
<td>Weld to Embed Plate = 3/16 in</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Rebar:</td>
<td>Size = #4</td>
<td></td>
<td></td>
</tr>
<tr>
<td># of Bars per Row = 2</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Vertical Spacing = 3.17 in</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Weld (Bolts) = 3/16 in</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>F, (Rebar) = 60 ksi</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td># of Rows = 3</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Vert. Spacing = 2.4 in</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Horiz. Spacing = 3 in</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Fnt (Bolts) = 45 ksi</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bolts: Ø = 7/8 in</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td># of Rows = 3</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Fnv (Bolts) = 27 ksi</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Beam Size: W14X30

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

APPLIED LOADS

Drag Load = 44.1 kips
Gravity Load = 6.0 kips

Drag Tensile Load = 7.9 kips (LRFD)
Gravity Tensile Load = 0.8 kips (LRFD)
Total Tensile Load = 8.7 kips (LRFD)
Gravity Shear Load = 0.8 kips (LRFD)

RESULTANT LOAD

\[
R_u = \sqrt{V_u^2 + N_u^2}
\]

\[
R_u = 63.29 \text{ kips}
\]

\[
\Theta = 84.56
\]

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REBAR DESIGN

TENSILE CAPACITY OF SINGLE BAR

\[ \phi \sigma_{t} = 0.75 \quad (ACI 318-19 17.5.3a) \]

\[ \Phi N_t = 9.0 \text{ kips} \]

SHEAR CAPACITY OF SINGLE BAR

\[ \phi \sigma_{s} = 0.65 \quad (ACI 318-19 17.5.3a) \]

\[ \Phi V_s = 7.8 \text{ kips} \]

TENSION & SHEAR INTERACTION

\[ \frac{N_t}{\Phi N_t} + \frac{V_s}{\Phi V_s} = 1.06 < 1.2 \]

EMBED PLATE DESIGN

EMBED PLATE THICKNESS

\[ L = 2 \text{ in} \]

\[ b = 4.58 \text{ in} \]

\[ T1 = 17.4 \text{ kips} \quad \text{(LRFD)} \]

\[ M_{n} = \frac{T1 L}{8} = 4.34 \text{ k-in} \quad \text{(AISC 15th Ed. 3-23.16)} \]

Flexure Yield:

\[ Z = \frac{b t^2}{4} = 0.29 \text{ in}^2 \quad \text{(AISC 15th Ed. F11-1)} \]

\[ \phi M_{n} = \phi F_y Z = 0.9 \]

\[ \Phi M_{n} = 9.28 \text{ k-in} \]

\[ \text{DCR} = 0.47 < 1 \]

Shear Yield:

\[ A_{n} = 5.5 \text{ in}^2 \]

\[ \phi \sigma_{s} = 0.6 F_y A_{n} \quad \text{(AISC 15th Ed. J4-3)} \]

\[ \phi = 0.75 \]

\[ \Phi R_{s} = 89.1 \text{ kips} \]

\[ \text{DCR} = 0.07 < 1 \]

STRENGTH OF WELD

\[ \mu = 1.0 + 0.5 \sin^{1.5} \theta \quad \text{(AISC 15th Ed. J2-9)} \]

\[ \theta = 84.56 \]

\[ \mu = 1.5 \]

\[ R_w = (1.392 \text{ kip/in}) D (2 \text{ sides}) \quad \text{(AISC 15th Ed. 8-2a)} \]

\[ R_w = 118.75 \text{ kips} \]

\[ \text{DCR} = 0.53 < 1 \]
STRENGTH OF BOLTED CONN.

RESULTANT LOAD

\[ R_0 = \sqrt{\left( V_U + N_U^2 \right)} \]

- \( R_0 = 63.29 \text{ kips} \)
- \( \Theta = 84.56 \)
- \( e = 4.5 \text{ in} \)
- \( C = 5.28 \text{ in} \)  
  
  \[ (\text{AISC 15th Ed. T.10-9}) \]

BEAM WEB STRENGTH

Bolt Shear:  
\[ \Phi \, r_n = \Phi \, F_u \, A_s \]  
- \( \Phi = 0.75 \)
- \( \Phi \, r_n = 12.18 \text{ kips/bolt} \)

Bolt Bearing Strength:  
\[ \Phi \, r_n = \Phi \, 3.0 \cdot d \cdot t \cdot F_u \]  
- \( \Phi = 0.75 \)
- \( \Phi \, r_n = 34.55 \text{ kips/bolt} \)

Bolt Tearout Strength:  
\[ \Phi \, r_n = \Phi \, 1.5 \cdot l \cdot t \cdot F_u \]  
- \( \Phi = 0.75 \)
- \( \Phi \, r_n = 29.62 \text{ kips/bolt} \)

Governing  
\[ \Phi \, r_n = 12.18 \text{ kips} \]
\[ \Phi \, R_n = C \, \Phi \, r_n \]  
- \( \Phi \, R_n = 64.23 \text{ kips} \)

DCR = \( 0.99 < 1 \)

SHEAR PLATE STRENGTH

Bolt Bearing Strength:  
\[ \Phi \, r_n = \Phi \, 3.0 \cdot d \cdot t \cdot F_u \]  
- \( \Phi = 0.75 \)
- \( \Phi \, r_n = 28.55 \text{ kips/bolt} \)

Bolt Tearout Strength:  
\[ \Phi \, r_n = \Phi \, 1.5 \cdot l \cdot t \cdot F_u \]  
- \( \Phi = 0.75 \)
- \( \Phi \, r_n = 24.47 \text{ kips/bolt} \)

Governing  
\[ \Phi \, r_n = 12.18 \text{ kips} \]
\[ \Phi \, R_n = C \, \Phi \, r_n \]  
- \( \Phi \, R_n = 64.23 \text{ kips} \)

DCR = \( 0.99 < 1 \)

BEAM CHECKS

Shear Yielding:  
\[ A_{py} = 3.73 \text{ in}^2 \]
\[ \Phi \, R_{py} = \Phi \, 0.6 \cdot F_y \cdot A_{py} \]  
  
  \[ (\text{AISC 15th Ed. J4-3}) \]
Tensile Yielding: \( A_y = 8.85 \text{ in}^2 \)

\[ \phi R_y = \phi F_y A_y \]

\[ \Phi = 0.9 \]

\[ \Phi R_y = 398.25 \text{ kips} \]

\[ \text{DCR} = 0.16 < 1 \]

Tensile Rupture: \( A_y = 8.09 \text{ in}^2 \)

\[ U = 1 - (x_{\bar{b}}/l) \]

\[ \bar{x} = \frac{2b_y^2 l_f + l_n^2 (d - 2t_f)}{8b_y l_f - 4l_n (d - 2t_f)} \]

\[ x_{\bar{b}} = 1.03 \text{ in} \]

\[ U = 0.66 \]

\[ \phi R_n = \phi F_y A_y \]

\[ \Phi = 0.75 \]

\[ \Phi R_n = 259.07 \text{ kips} \]

\[ \text{DCR} = 0.24 < 1 \]

Block Shear Rupture: \( A_y = 2.7 \text{ in}^2 \)

\[ A_n = 2.32 \text{ in}^2 \]

\[ A_v = 1.28 \text{ in}^2 \]

\[ U_n = 1 \]

\[ \Phi = 0.75 \]

\[ \phi R_n = \phi 0.6 F_y A_v + U_{n_{\bar{b}}l_{n_{\bar{b}}}F_u A_{n_{\bar{b}}} + U_{n_{v}} F_u A_{v} = \phi 0.6F_y A_v + U_{n_{\bar{b}}} F_u A_{n_{\bar{b}}} = 144.11 \text{ kips} \]

\[ \phi 0.6 F_u A_v + U_{n_{v}} F_u A_{v} = 151.23 \text{ kips} \]

\[ \phi R_n = 144.11 \text{ kips} \]

\[ \text{DCR} = 0.31 < 1 \]

\section*{PLATE CHECKS}

\textbf{Maximum Plate Thick:}

\[ t_{\text{max}} = \frac{E' M_{\text{max}}}{F_y t} \]

\[ M_{\text{max}} = \frac{F_{\text{max}}}{0.5} (d_0 \cdot C') \]

\[ C' = 15.8 \]

\[ M_{\text{max}} = 285.03 \text{ k-in} \]

\[ t = 0.3 \text{ in} \]

\textbf{Flexure Yield:}

\[ \phi M_y = \phi F_y Z \]

\[ Z = \frac{rD^2}{4} \]

\[ Z = 5.64 \text{ in}^4 \]

\[ \phi M_y = 182.76 \text{ k-in} \]

\[ \phi R_n = 40.61 \text{ kips} \]
Lateral-Torsional Buckling:

\[
\frac{0.08E}{F_y} = 64.44
\]
\[
\frac{1.9E}{F_y} = 1530.56
\]
\[
\frac{L_d}{t} = 456
\]

\[
\frac{0.08E}{F_y} < \frac{L_d}{t} < \frac{1.9E}{F_y} \quad \text{So use AISC 15th Ed. F11-2b}
\]

\[
M_u = C_b \left[ 1.52 \cdot 0.274 \left( \frac{L_d}{t} \right) \frac{F_y}{E} \right] M_y
\]

\[
C_b = \frac{12.5M_{res}}{2.5 M_{res} + 3M_a + 4M_b + 3M_c}
\]

\[
M_i = 135.38 \text{ kip-in}
\]
\[
C_i = 1.67
\]
\[
\Phi = 0.9
\]
\[
\phi M_y = 277.16 \text{ kip-in}
\]
\[
\phi R_y = 61.59 \text{ kips}
\]

DCR = 0.15 < 1

Shear Yielding:

\[
A_y = 2.38 \text{ in}^2
\]
\[
\phi R_y = \phi F_y A_y, \frac{1}{1} = \phi
\]
\[
\phi R_y = 51.3 \text{ kips}
\]

DCR = 0.12 < 1

Tensile Yielding:

\[
A_t = 2.38 \text{ in}^2
\]
\[
\phi R_y = \phi F_y A_y, \frac{1}{1} = \phi
\]
\[
\phi R_y = 76.95 \text{ kips}
\]

DCR = 0.82 < 1

Interaction of Axial, Flexure and Shear Yielding in Plate:

\[
N_u = 63.0 \text{ kips}
\]
\[
V_u = 6.0 \text{ kips}
\]
\[
\phi R_y = 76.95 \text{ kips}
\]
\[
\phi M_y = 182.76 \text{ kip-in}
\]

\[
\left[ \frac{N}{\phi R_y} \right] + \left[ \frac{B}{9} \frac{V_u A_y}{\phi M_y} \right]^2 \geq 0.92 < 1
\]

\[
N_{res} = \frac{N}{\phi R_y} \geq 0.82 > 0.2 \quad \text{So use AISC 15th Ed. H1-1a}
\]

Flexure Rupture:

\[
Z_{net} = \frac{d^2}{4} - \frac{1}{4} \left( d_h + \frac{V_u}{A_y} \sin \left( \frac{\theta}{\sin \theta} \right) \left( n^2 - 1 \right) + \left( \frac{d_h}{2} + \frac{1}{n} \sin \theta \right) \left( 2d_h + \frac{1}{n} \sin \theta \right) \right)
\]
\[
\phi M_n = \phi F_y Z_{net}
\]
\[
\phi M_n = 4.46 \text{ in}^3
\]
\[
\Phi = 0.75
\]
\[
\phi M_n =
\]
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\[
\phi M_n = 194.04 \, \text{k-in} \\
\phi R_n = 43.12 \\
\text{DCR} = 0.14 < 1
\]

**Shear Rupture:**
\[
A_n = 1.67 \, \text{in}^2 \\
\phi R_n = \phi \frac{0.6 F_u A_g}{\Phi} \\
= \frac{43.64 \, \text{kips}}{0.75} \\
= 58.22 \, \text{kips} \\
\text{DCR} = 0.14 < 1
\]

**Tensile Rupture:**
\[
U_n = 1 \\
\phi R_n = \phi \frac{F_u A_g}{\Phi} \\
= \frac{72.73 \, \text{kips}}{0.75} \\
= 96.97 \, \text{kips} \\
\text{DCR} = 0.08 < 1
\]

**Interaction of Axial, Flexure and Shear Rupture in Plate:**
\[
\frac{N_u}{\phi R_{uw}} = 0.37 > 0.2 \quad \text{So use AISC 15th Ed. H1-1a}
\]

\[
\left( \frac{N_u}{\phi R_{uw}} \right) + \frac{8}{9} \left( \frac{V_{ud}}{\phi M_n} \right)^2 + \left( \frac{V_{ud}}{\phi R_{uw}} \right)^2 = 1 < 1 \quad \text{(AISC 15th Ed. H1-1a)}
\]

\[
\left( \frac{N_u}{\phi R_{uw}} \right) + \frac{V_{ud}^2}{\phi M_n} + \left( \frac{V_{ud}}{\phi R_{uw}} \right)^2 = \frac{N/A}{1} \quad \text{(AISC 15th Ed. H1-1b)}
\]

**Block Shear Rupture (Beam Shear Direction):**
\[
A_w = 2 \, \text{in}^2 \\
A_n = 1.45 \, \text{in}^2 \\
A_{nw} = 0.8 \, \text{in}^2 \\
U_{nw} = 0.5 \\
\Phi = 0.75 \\
\phi R_n = \phi \frac{0.6 F_u A_g + U_{nw} F_u A_{gw} + \phi 0.6 F_u A_{gw}}{\phi R_{nw}} \\
= \frac{55.51 \, \text{kips}}{105.08 \, \text{kips}} \\
\phi R_n = 0.53 \quad \text{DCR} = 0.11 < 1 \quad \text{(AISC 15th Ed. J4-5)}
\]

**Block Shear Rupture (Beam Axial Direction L Shape):**
\[
A_w = 1.13 \, \text{in}^2 \\
A_n = 0.8 \, \text{in}^2 \\
A_{nw} = 1.45 \, \text{in}^2 \\
U_{nw} = 1 \\
\Phi = 0.75 \\
\phi R_n = \phi \frac{0.6 F_u A_g + U_{nw} F_u A_{gw} + \phi 0.6 F_u A_{gw}}{\phi R_{nw}} \\
= \frac{102.51 \, \text{kips}}{105.08 \, \text{kips}} \\
\phi R_n = 0.98 \quad \text{DCR} = 0.43 < 1 \quad \text{(AISC 15th Ed. J4-5)}
\]

**Block Shear Rupture (Beam Axial Direction U Shape):**
\[
A_w = 2.25 \, \text{in}^2 \\
A_n = 1.59 \, \text{in}^2 \\
A_{nw} = 1.19 \, \text{in}^2 \\
U_{nw} = 1 \\
\phi R_n = \phi \frac{0.6 F_u A_g + U_{nw} F_u A_{gw} + \phi 0.6 F_u A_{gw}}{\phi R_{nw}} \\
= \frac{102.51 \, \text{kips}}{105.08 \, \text{kips}} \\
\phi R_n = 0.98 \quad \text{DCR} = 0.43 < 1 \quad \text{(AISC 15th Ed. J4-5)}
\]
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\( \Phi = 0.75 \)

\[ \Phi R_n = \Phi 0.6 F_u A_{sw} + U_{bw} F_u A_{sw} \leq \Phi 0.6 F_u A_{sw} + U_{bw} F_u A_{sw} \]

\[ 0.6 F_u A_{sw} + U_{bw} F_u A_{sw} = 105.33 \text{ kips} \]

\[ 0.6 F_u A_{sw} + U_{bw} F_u A_{sw} = 110.47 \text{ kips} \]

\[ 0.6 F_u A_{sw} + U_{bw} F_u A_{sw} = 105.33 \text{ kips} \]

DCR = 0.42 < 1

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

\( V_u = 6.0 \text{ kips} \)

\( N_u = 44.1 \text{ kips} \)

\( \Phi R_{bu} = 55.51 \text{ kips} \)

\( \Phi R_{bn} = 102.51 \text{ kips} \)

\[ \left( \frac{V_u}{\Phi R_{bu}} \right)^2 + \left( \frac{N_u}{\Phi R_{bn}} \right)^2 = 0.2 < 1 \]
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
- $F_u = 58$ ksi

Shear Plate:
- $L_y = 5$ in
- $L_x = 9.5$ in
- t = 5/8 in

Weld to Embed Plate:
- $F_y = 36$ ksi
- $F_u = 58$ ksi

Rebar:
- Size = #7
- # of Bars per Row = 2
- # of Rows = 4
- Vertical Spacing = 3.17 in
- $F_y$ (Rebar) = 60 ksi

Welds:
- D = 5/16 in
- Lwy = 9.5 in
- Lwx = 4 in

Beam Size:

<table>
<thead>
<tr>
<th>Beam Size</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>W14X30</td>
<td></td>
</tr>
<tr>
<td>tf = 0.39</td>
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<td>bf = 6.73</td>
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<td>d = 13.8</td>
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<tr>
<td>Ag = 8.85</td>
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</table>

APPLIED LOADS

<table>
<thead>
<tr>
<th>Load Type</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Drag Load (ASD)</td>
<td>59.1 kips</td>
</tr>
<tr>
<td>Gravity Load (LRFD)</td>
<td>45.2 kips</td>
</tr>
<tr>
<td>Drag Tensile Load</td>
<td>10.6 kips</td>
</tr>
<tr>
<td>Gravity Tensile Load</td>
<td>5.2 kips</td>
</tr>
<tr>
<td>Total Tensile Load</td>
<td>15.8 kips</td>
</tr>
<tr>
<td>Gravity Shear Load</td>
<td>5.7 kips</td>
</tr>
</tbody>
</table>

RESULTANT LOAD

$$R = \sqrt{V^2 + N^2}$$

- $R_y = 95.77$ kips
- $\theta = 61.84$
REBAR DESIGN

TENSILE CAPACITY OF SINGLE BAR

\[ A_t = 0.6 \text{ in}^2 \]
\[ \phi = 0.75 \]
\[ \Phi N_t = 27.0 \text{ kips} \]  

(ACI 318-19 17.5.3a)

SHEAR CAPACITY OF SINGLE BAR

\[ A_t = 0.6 \text{ in}^2 \]
\[ \phi = 0.65 \]
\[ \Phi V_t = 23.4 \text{ kips} \]

(ACI 318-19 17.5.3a)

TENSION & SHEAR INTERACTION

\[ N_{in} = 15.8 \text{ kips} \]
\[ V_{in} = 5.7 \text{ kips} \]
\[ N_{in}/\Phi N_t = 0.59 \quad V_{in}/\Phi V_t = 0.24 \]

\[ (N_{in}/\Phi N_t) + (V_{in}/\Phi V_t) = 0.83 < 1.2 \]

EMBED PLATE DESIGN

EMBED PLATE THICKNESS

\[ L = 2 \text{ in} \]
\[ b = 4.58 \text{ in} \]
\[ T_1 = 31.6 \text{ kips} \]  

(LRFD)

\[ M_n = \frac{T_1 t}{8} \]  

(AISC 15th Ed. 3-23.16)

Flexure Yield:

\[ Z = \frac{0.45}{4} \text{ in}^2 \]
\[ \phi M_n = \phi F_y Z \]  

(AISC 15th Ed. F11-1)

\[ \Phi M_n = 7.9 \text{ k-in} \]
\[ \Phi F_y = 14.5 \text{ k-in} \]

DCR = 0.54 < 1

Shear Yield:

\[ A_w = 6.88 \text{ in}^2 \]
\[ \phi R_w = \phi 0.6 F_y A_w \]  

(AISC 15th Ed. J4-3)

\[ \phi = 0.75 \]
\[ \Phi R_w = 111.38 \text{ kips} \]

DCR = 0.41 < 1

STRENGTH OF WELD

\[ \mu = 1.0 + 0.5 \sin^{1.5} \theta \]  

(AISC 15th Ed. J2-5)

\[ \Theta = 61.84 \]
\[ \mu = 1.41 \]

\[ R_n = (1.392 \text{ kip/in}) D \mu (2 \text{ sides}) \]  

(AISC 15th Ed. B-2a)

\[ R_n = 112.18 \text{ kips} \]

DCR = 0.85 < 1
STRENGTH OF WELDED CONN.

RESULTANT LOAD

\[ R_u = \sqrt{V_u^2 + N_u^2} \]

\[ \Theta = 95.77 \text{ kips} \]

\[ \Theta = 61.84 \]

WELD STRENGTH

\[ \frac{kI}{l} = \frac{4}{9.5} = k = 0.42 \]

\[ x = 0.10 \]

\[ xl = 0.92 \text{ in} \]

\[ e_x = 4.08 \text{ in} \]

\[ \frac{e_x}{I} = a = 0.43 \]

\[ C = 3.13 \]

\[ \phi R_u = \phi CC_1 Dl \]

\[ \phi = 0.75 \]

\[ C_1 = 1 \]

Gravity Load: \[ \phi R_u = 111.68 \text{ kips} \]

DCR = \[ 0.4 < 1 \]

Drag Load: \[ 94.04 \]

DCR = \[ 0.9 < 1 \]

BEAM CHECKS

Shear Rupture of Beam Web:

\[ t_{min} = \frac{3.09 D}{F_U} \]

\[ t_{min} = 0.01 \text{ in} \]

DCR = \[ 0.06 < 1 \]

Shear Yielding:

\[ A_s = 3.73 \text{ in}^2 \]

\[ \phi R_y = \phi 0.6 F_y A_s \]

\[ \phi = 0.75 \]

\[ R_y = 111.78 \text{ kips} \]

DCR = \[ 0.4 < 1 \]

Tensile Yielding:

\[ A_s = 8.85 \text{ in}^2 \]

\[ \phi R_y = \phi F_y A_s \]

\[ \phi = 0.9 \]

\[ R_y = 398.25 \text{ kips} \]
DCR = 0.21 <1

Tensile Rupture:
\[ A_s = 8.85 \text{ in}^2 \]
\[ U = 1 - \frac{\text{x_bar}}{\text{l}} \]
\[ F = \frac{2h_f^2 + t_m^2 (d - 2f_t)}{8ht_f + 4t_m (d - 2f_t)} \]
\[ \text{x_bar} = 1.03 \text{ in} \]
\[ U = 0.74 \]
\[ \phi R = \phi F_a A_s \quad (AISC 15^{th} Ed. J4-2) \]
\[ \phi = 0.75 \]
\[ \phi R = 320.4 \text{ kips} \]

DCR = 0.26 <1

Block Shear Rupture:
\[ A_{sv} = 2.16 \text{ in}^2 \]
\[ A_{sv} = 2.16 \text{ in}^2 \]
\[ A_{sn} = 2.57 \text{ in}^2 \]
\[ U_n = 1 \]
\[ \phi = 0.75 \]
\[ \frac{\phi}{R_n} = 215.33 \text{ kips} \]

DCR = 0.27 <1

**PLATE CHECKS**

Shear Rupture of Plate:
\[ t_{\min} = \frac{3.09 D}{F_u} \]
\[ t_{\min} = 0.02 \text{ in} \]

DCR = 0.03 <1

Flexure Yield:
\[ \phi M_n = \phi F_u Z \quad (AISC 15^{th} Ed. F11-1) \]
\[ Z = \frac{r e_x z}{t} \]
\[ Z = 14.1 \text{ in}^3 \]
\[ \phi = 0.9 \]
\[ \phi M_n = 456.89 \text{ k-in} \]
\[ \phi R_n = 111.91 \text{ kips} \]

DCR = 0.4 <1

Lateral-Torsional Buckling:
\[ \frac{0.08E}{F_t} = 64.44 \]
\[ \frac{1.9E}{F_t} = 1530.56 \]
\[ \frac{L_d}{F_t} = 97.28 \]
\[ \frac{0.08E}{F_t} < \frac{L_d}{F_t} < \frac{1.9E}{F_t} \quad \text{So use AISC 15^{th} Ed. F11-2b} \]
\[ M_n = C_s \left[ 1.5 - 0.274 \left( \frac{L d}{f} \right) \frac{F}{E} \right] M_s \]  
\[ C_s = \frac{12.5 M_{max}}{2.5 M_t + 3 M_s + 4 M_n + 3 M_c} \]  
\[ M_t = 507.66\ \text{kip-in} \]  
\[ C_s = 1.67 \]  
\[ \Phi = 0.9 \]  
\[ \phi M_s = 1132.26\ \text{kip-in} \]  
\[ \phi R_n = 277.34\ \text{kips} \]  
\[ \text{DCR} = 0.16 < 1 \]

**Shear Yielding:**
\[ A_s = \frac{5.94}{\phi} \]  
\[ \phi R_n = \phi 0.6 F_y A_s \]  
\[ \Phi = \frac{1}{1} \]  
\[ \phi R_n = 128.25\ \text{kips} \]  
\[ \text{DCR} = 0.35 < 1 \]

**Tensile Yielding:**
\[ A_s = \frac{5.94}{\phi} \]  
\[ \phi R_n = \phi F_y A_s \]  
\[ \Phi = 0.9 \]  
\[ \phi R_n = 192.38\ \text{kips} \]  
\[ \text{DCR} = 0.44 < 1 \]

**Interaction of Axial, Flexure and Shear Yielding in Plate:**
\[ \frac{N_u}{\phi R_{nv}} = \frac{84.4}{45.2} = 0.44 \]  
\[ \text{So use AISC 15th Ed. H1-1a} \]

\[ I \left[ \frac{N_u}{\phi R_{nv}} + \frac{8}{9} \left( \frac{V_u}{\phi M_u} \right)^2 + \left( \frac{V_u}{\phi R_{nv}} \right)^2 \right] = 0.8 < 1 \]  
\[ \left( \frac{N_u}{2\phi R_{nv}} + \frac{V_u A_{yv}}{\phi M_u} \right)^2 + \left( \frac{V_u}{\phi R_{nv}} \right)^2 = N/A > 1 \]  
\[ \text{AISC 15th Ed. H1-1b} \]

**Flexure Rupture:**
\[ Z = \frac{rD^2}{2} \]  
\[ Z_{vn} = 14.1\ \text{in}^4 \]  
\[ \Phi = 0.75 \]  
\[ \phi M_n = 613.42\ \text{k-in} \]  
\[ \phi R_n = 150.26 \]  
\[ \text{DCR} = 0.3 < 1 \]

**Shear Rupture:**
\[ A_{sv} = \frac{5.94}{\phi} \]  
\[ \phi R_n = \phi 0.6 F_y A_s \]  
\[ \Phi = 0.75 \]  
\[ \phi R_n = 154.97\ \text{kips} \]  
\[ \text{DCR} = 0.29 < 1 \]
Tensile Rupture: \( A_n = 5.94 \text{ in}^2 \)
\( U = 1 \)
\( \Phi R_n = \Phi F_p A_n \)  \( \text{(AISC 15th Ed. H4-2)} \)
\( \Phi = 0.75 \)
\( \Phi R_n = 258.28 \)

DCR = \( 0.18 < 1 \)

Interaction of Axial, Flexure and Shear Rupture in Plate:
\( N_u = 84.4 \text{ kips} \)
\( V_u = 45.2 \text{ kips} \)
\( \Phi R_n = 258.28 \text{ kips} \)
\( \Phi R_{\nu} = 150.26 \text{ kips} \)
\( \Phi M_{\nu} = 613.42 \text{ kip-in} \)

\[
\frac{N_u}{\Phi R_{\nu}} = 0.33 > 0.2 \quad \text{So use AISC 15th Ed. H1-1a}
\]

\[
\left[ \frac{N_u}{\Phi R_{\nu}} + \frac{8}{9} \left( \frac{V_{a\nu}}{\Phi M_{\nu}} \right)^2 + \left( \frac{V_u}{\Phi R_{\nu}} \right)^2 \right] = 0.44 < 1
\]  \( \text{(AISC 15th Ed. H1-1a)} \)

\[
\left\{ \frac{N_{\nu}}{2 \Phi R_{\nu}} + \frac{V_{a\nu}}{\Phi M_{\nu}} \right\}^2 + \left( \frac{V_u}{\Phi R_{\nu}} \right)^2 = \frac{N/A}{1} \quad \text{(AISC 15th Ed. H1-1b)}
\]
**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
- \( t = \frac{1}{2} \) in
- \( F_y = 36 \text{ ksi} \)
- \( F_u = 58 \text{ ksi} \)

Shear Plate:
- \( L_x = 5 \) in
- \( t = \frac{3}{8} \) in
- \( F_y = 36 \text{ ksi} \)
- \( F_u = 58 \text{ ksi} \)

Weld to Embed Plate:
- \( t = \frac{3}{16} \) in
- \( F_y = 36 \text{ ksi} \)
- \( F_u = 58 \text{ ksi} \)

Rebar:
- Size = #5
- \# of Bars per Row = 2
- \# of Rows = 4
- Vertical Spacing = 4.5 in
- \( F_y = 60 \text{ ksi} \)

Bolts:
- \( \varnothing = 1 \) in
- \# of Rows = 4
- Vert. Spacing = 3 in
- Horiz. Spacing = 0 in
- \( F_{nt} = 45 \text{ ksi} \)
- \( F_{nv} = 27 \text{ ksi} \)

Beam Size:

- W18x50
- \( tf = 0.57 \)
- \( tw = 0.36 \)
- \( bf = 7.5 \)
- \( d = 18 \)
- \( Ag = 14.7 \)

**APPLIED LOADS**

- Drag Load = 19.7 kips
- Gravity Load = 44.7 kips

Drag Tensile Load = \( \frac{19.7}{0.7 \cdot \# \text{ of Bars}} \)

- Drag Tensile Load = 3.5 kips
- Gravity Tensile Load = 3.1 kips
- Total Tensile Load = 6.6 kips
- Gravity Shear Load = 5.6 kips

**WELDED**

**BOLT: DOUBLE ROW**
RESULTANT LOAD

\[ R_u = \sqrt{N_u^2 + V_u^2} \]

\[ R_u = 52.82 \text{ kips} \]

\[ \Theta = 32.19 \]

REBAR DESIGN

TENSILE CAPACITY OF SINGLE BAR

\[ A_s = 0.31 \text{ in}^2 \]

\[ \Phi = 0.75 \]

\[ \Phi N_s = 14.0 \text{ kips} \]

SHEAR CAPACITY OF SINGLE BAR

\[ A_s = 0.31 \text{ in}^2 \]

\[ \Phi = 0.65 \]

\[ \Phi V_u = 12.1 \text{ kips} \]

TENSION & SHEAR INTERACTION

\[ N_u = 6.6 \text{ kips} \]

\[ V_u = 5.6 \text{ kips} \]

\[ N_u / \Phi N_s = 0.47 \]

\[ V_u / \Phi V_u = 0.46 \]

\[ (N_u / \Phi N_s) + (V_u / \Phi V_u) = 0.94 < 1.2 \]

EMBED PLATE DESIGN

EMBED PLATE THICKNESS

\[ L = 2 \text{ in} \]

\[ b = 5.25 \text{ in} \]

\[ T1 = 13.2 \text{ kips (LRFD)} \]

\[ M_u = \frac{T1 \cdot L}{8} \]

\[ M_u = 3.3 \text{ K-in} \]

\[ Z = \frac{\Phi T \cdot L}{4} \]

\[ Z = 0.33 \text{ in}^3 \]

\[ \phi R_y = \phi F_y Z \]

\[ \phi R_y = 0.9 \]

\[ \Phi M_u = 10.63 \text{ K-in} \]

\[ DCR = 0.31 < 1 \]

Shear Yield:

\[ A_s = 7.5 \text{ in}^2 \]

\[ \phi R_y = 0.6 F_y A_s \]

\[ \phi R_y = 121.5 \text{ kips} \]

\[ DCR = 0.37 < 1 \]

STRENGTH OF WELD

\[ \mu = 1.0 + 0.5 \sin^{1.5} \theta \]

\[ \Theta = 32.19 \]

\[ \mu = 1.19 \]

\[ R_u = \frac{(1.392 \text{ kip/in}) D \mu}{(2 \text{ sides})} \]

\[ R_u = 134.58 \text{ kips} \]

\[ DCR = 0.39 < 1 \]
**STRENGTH OF BOLTED CONN.**

**RESULTANT LOAD**

\[ R_u = \sqrt{V_u^2 + N_u^2} \]

- \( R_u = 52.82 \) kips
- \( \Theta = 32.19 \)
- \( e = 1.5 \)
- \( C = 3.47 \)  

(AISC 15th Ed. T.10-9)

**BEAM WEB STRENGTH**

Bolt Shear:

\[ \phi R_e = \phi F_u A_e \]

- \( \phi = 0.75 \)
- \( \phi R_e = 15.9 \) kips/bolt

Bolt Bearing Strength:

\[ \phi R_b = \phi 3.0 \cdot d_t F_u \]

- \( \phi = 0.75 \)
- \( \phi R_b = 51.92 \) kips/bolt

Bolt Tearout Strength:

\[ \phi R_t = \phi 1.5 \cdot l_c t F_u \]

- \( \phi = 0.75 \)
- \( \phi R_t = 37.32 \) kips/bolt

**Governing**

\[ \phi R_e = 15.9 \text{ kips} \]

- \( \Theta \phi R_e = 55.19 \) kips

DCR = 0.96 <1

**SHEAR PLATE STRENGTH**

Bolt Bearing Strength:

\[ \phi R_b = \phi 3.0 \cdot d_t F_u \]

- \( \phi = 0.75 \)
- \( \phi R_b = 48.94 \) kips/bolt

Bolt Tearout Strength:

\[ \phi R_t = \phi 1.5 \cdot l_c t F_u \]

- \( \phi = 0.75 \)
- \( \phi R_t = 22.94 \) kips/bolt

**Governing**

\[ \phi R_e = 15.9 \text{ kips} \]

- \( \Theta \phi R_e = 55.19 \) kips

DCR = 0.96 <1

**PLATE CHECKS**

Maximum Plate Thick:

\[ t_{\text{MAX}} = \left( \frac{D_{\text{BOLT}}}{2} \right) \cdot \left( \frac{1}{16} \right) \]

- \( t = 0.56 \) in

(AISC 15th Ed. T.10-9)
STEEL DRAG BEAM CONNECTION CALCULATION

WHAT TYPE OF CONNECTION IS BEING USED?

BOLT: DOUBLE ROW

MATERIAL AND CONFIGURATION

Embed Plate:
- Height = 16 in
- Width = 8 in
- \(F_y = 36\) ksi
- \(F_u = 58\) ksi
- \(t = 1/2\) in
- \(L_y = 5\) in
- \(L_x = 11.5\) in

Shear Plate:
- \(F_y = 36\) ksi
- \(F_u = 58\) ksi
- \(t = 3/8\) in
- Weld to Embed Plate = 3/16 in

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

Bolts:
- \(\phi = 1\) in
- \# of Rows = 3
- Vert. Spacing = 3.45 in
- Horiz. Spacing = 5.5 in
- \(F_{nt} \) (Bolts) = 45 ksi
- \(F_{nv} \) (Bolts) = 27 ksi

Beam Size:
- W16x36
- \(t_f = 0.43\)
- \(b_w = 0.3\)
- \(b_f = 6.99\)
- \(d = 15.9\)
- \(A_g = 10.6\)

APPLIED LOADS

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

**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

\[ R_1 = \sqrt{R_{u1}^2 + N_{u1}^2} \]

\[ R_0 = 60.49 \text{ kips} \]

\[ \Theta = 62.32 \]
**REBAR DESIGN**

**TENSILE CAPACITY OF SINGLE BAR**

\[ A_t = 0.31 \text{ in}^2 \]

\[ \phi = 0.75 \]

\[ \Phi N_t = 14.0 \text{ kips} \]

**SHEAR CAPACITY OF SINGLE BAR**

\[ A_s = 0.31 \text{ in}^2 \]

\[ \phi = 0.65 \]

\[ \Phi V_s = 12.1 \text{ kips} \]

**TENSION & SHEAR INTERACTION**

\[ N_{in} = 11.0 \text{ kips} \]

\[ V_{in} = 3.5 \text{ kips} \]

\[ \frac{N_{in}}{\Phi N_t} = 0.79 \]

\[ \frac{V_{in}}{\Phi V_s} = 0.29 \]

\[ \left( \frac{N_{in}}{\Phi N_t} \right) \left( \frac{V_{in}}{\Phi V_s} \right) = 1.08 < 1.2 \]

**EMBED PLATE DESIGN**

**EMBED PLATE THICKNESS**

\[ L = 2 \text{ in} \]

\[ b = 4.92 \text{ in} \]

\[ T1 = 22.1 \text{ kips} \]

\[ M_n = \frac{T1 \cdot T}{8} \] (LRFD)

\[ M_n = 5.51 \text{ k-in} \]

**Flexure Yield**

\[ Z = \frac{b^2 h}{4} \]

\[ \phi M_{fl} = 0.31 \text{ in}^2 \]

\[ \phi = 0.9 \]

\[ \Phi M_n = 9.96 \text{ k-in} \]

\[ DCR = 0.55 < 1 \]

**Shear Yield**

\[ A_w = 6.5 \text{ in}^2 \]

\[ \phi R_{yw} = 0.6 \phi F_y A_w \] (AISC 15th Ed. J4-3)

\[ \phi = 0.75 \]

\[ \Phi R_n = 105.3 \text{ kips} \]

\[ DCR = 0.27 < 1 \]

**STRENGTH OF WELD**

\[ \mu = 1.0 + 0.5 \sin^{1/5} \theta \] (AISC 15th Ed. J2-5)

\[ \theta = 62.32 \]

\[ \mu = 1.42 \]

\[ R_n = (1.392 \text{ kip/in}) Dl \mu \] (2 sides)

\[ R_n = 136.07 \text{ kips} \] (AISC 15th Ed. 8-2a)

\[ DCR = 0.44 < 1 \]
STRENGTH OF BOLTED CONN.

RESULTANT LOAD

\[ R_U = \sqrt{V_U + N_U^2} \]

\[ R_U = 60.49 \text{ kips} \]
\[ \Theta = 62.32 \]
\[ \Theta = 5.75 \text{ kips} \]
\[ C = 4.08 \text{ kips} \]

BEAM WEB STRENGTH

Bolt Shear:

\[ \phi r_n = \phi F_n A_s \]
\[ \phi = 0.75 \]
\[ \phi r_n = 15.9 \text{ kips/bolt} \]

Bolt Bearing Strength:

\[ \phi r_n = \phi 3.0 d F_U \]
\[ \phi = 0.75 \]
\[ \phi r_n = 43.14 \text{ kips/bolt} \]

Bolt Tearout Strength:

\[ \phi r_n = \phi 1.5 F_U \]
\[ \phi = 0.75 \]
\[ \phi r_n = 31.01 \text{ kips/bolt} \]

Governing \( \phi r_n = 15.9 \text{ kips} \)

\[ \phi R_n = C \phi r_n \]
\[ \phi R_n = 64.83 \text{ kips} \]

DCR = 0.93 < 1

SHEAR PLATE STRENGTH

Bolt Bearing Strength:

\[ \phi r_n = \phi 3.0 d t F_U \]
\[ \phi = 0.75 \]
\[ \phi r_n = 48.94 \text{ kips/bolt} \]

Bolt Tearout Strength:

\[ \phi r_n = \phi 1.5 t F_U \]
\[ \phi = 0.75 \]
\[ \phi r_n = 35.17 \text{ kips/bolt} \]

Governing \( \phi r_n = 15.9 \text{ kips} \)

\[ \phi R_n = C \phi r_n \]
\[ \phi R_n = 64.83 \text{ kips} \]

DCR = 0.93 < 1

BEAM CHECKS

Shear Yielding:

\[ A_v = 4.69 \text{ in}^2 \]
\[ \phi R_n = \phi 0.6 F_y A_v \]

(AISC 15th Ed. J4-3)
Project: 18412 – Angelo View Revisions – Uberion

Tensile Yielding:

\[ \Phi = 0.6 \frac{F_y}{A_g} \]

\[ \Phi = \frac{140.72 \text{ kips}}{1} \]

DCR = 0.2 < 1

Tensile Rupture:

\[ \Phi = 0.9 \]

\[ \Phi = 477 \text{ kips} \]

DCR = 0.11 < 1

Block Shear Rupture:

\[ \Phi = 0.75 \]

\[ \Phi = 382.14 \text{ kips} \]

DCR = 0.14 < 1

PLATE CHECKS

Maximum Plate Thick:

\[ t_{\text{max}} = \frac{E' M_{\text{ax}}}{F_y Z} \]

(AISC 15th Ed. 10-3)

\[ M_{\text{ax}} = \frac{F_M}{0.5} (I_{22} + C') \]

(AISC 15th Ed. 10-4)

\[ C' = 21.2 \]

(AISC 15th Ed. 10-4)

\[ M_{\text{ax}} = 499.51 \text{ k-in} \]

\[ t = 0.4 \text{ in} \]

Flexure Yield:

\[ \phi M_y = \phi F_y Z \]

(AISC 15th Ed. F11-1)

\[ Z = \frac{et^{3/2}}{2} \]

\[ Z = 12.4 \text{ in}^4 \]

\[ \Phi = 0.9 \]

\[ \phi M_y = 401.71 \text{ k-in} \]

\[ \phi R_n = 69.86 \text{ kips} \]
DCR = \[ 0.4 < 1 \]

Lateral-Torsional Buckling:

\[ \frac{0.08E}{F_y} = 64.44 \]
\[ \frac{1.9E}{F_y} = 1530.56 \]
\[ \frac{L_d}{f} = 245.33 \]

\[ \frac{0.08E}{F_y} < \frac{L_d}{f} < \frac{1.9E}{F_y} \]

So use AISC 15th Ed. F11-2b

\[ M_u = C_f[1.52 - 0.274\left(\frac{L_d}{f}\right)\frac{F_y}{E}]M_y \]

(AISC 15th Ed. F11-2)

\[ C_f = \frac{12.5M_{plas}}{2.5M_y + 3M_x + 4M_w + 3M_c} \]

(AISC 15th Ed. F1-1)

\[ M_y = 297.56 \text{ kip-in} \]
\[ C_f = 1.67 \]
\[ \phi = 0.9 \]
\[ \phi M_w = 641.2 \text{ kip-in} \]
\[ \phi R_y = 111.51 \text{ kips} \]

DCR = \[ 0.25 < 1 \]

Shear Yielding:

\[ A_{pl} = 4.31 \text{ in}^2 \]
\[ \phi R_y \equiv \phi F_y A_{pl} \]
\[ \phi = 0.9 \]
\[ \phi R_y = 93.15 \text{ kips} \]

DCR = \[ 0.3 < 1 \]

Tensile Yielding:

\[ A_{pl} = 4.31 \text{ in}^2 \]
\[ \phi R_y \equiv \phi F_y A_{pl} \]
\[ \phi = 0.9 \]
\[ \phi R_y = 139.73 \text{ kips} \]

DCR = \[ 0.38 < 1 \]

Interaction of Axial, Flexure and Shear Yielding in Plate:

\[ N_u = 53.6 \text{ kips} \]
\[ V_u = 28.1 \text{ kips} \]
\[ \phi R_w = 93.15 \text{ kips} \]
\[ \phi M_w = 401.71 \text{ kip-in} \]

\[ \frac{N_u}{\phi R_w} = 0.38 > 0.2 \]

So use AISC 15th Ed. H1-1a

\[ \left[ \frac{N_u}{\phi R_w} + \frac{V_u}{\phi M_w} \right]^2 + \left[ \frac{V_u}{\phi M_w} \right]^2 = 0.64 < 1 \]

(AISC 15th Ed. H1-1a)

\[ \left[ \frac{N_u}{2\phi R_w} \right]^2 + \left[ \frac{V_u}{\phi M_w} \right]^2 = \frac{N_u}{2\phi R_w} \]

(AISC 15th Ed. H1-1b)

Flexure Rupture:

\[ M_{plas} = \frac{d^2}{4} \left( \frac{d_x + \sqrt{d_x^2 + 8V_u}}{2} \right) \]
\[ \phi M_{plas} = \phi F_y Z_{plas} \]
\[ Z_{plas} = 9.54 \text{ in}^3 \]
\[ \phi = 0.75 \]
\[ \phi M_{plas} = \]
\[ \phi M' = 415.14 \text{ k-in} \]
\[ \phi R_u = 72.2 \]
\[ \text{DCR} = 0.39 < 1 \]

Shear Rupture:
\[ A_v = 3.12 \text{ in}^2 \]
\[ \phi R_v = \frac{0.6 F_u A_v}{\Phi} = 81.36 \text{ kips} \]
\[ \phi R_u = \text{N/A} > 1 \]
\[ \text{DCR} = 0.35 < 1 \]

Tensile Rupture:
\[ A_v = 3.12 \text{ in}^2 \]
\[ U = 1 \]
\[ \phi R_v = \frac{0.6 F_u A_v}{\Phi} = 135.6 \text{ kips} \]
\[ \phi R_u = \text{N/A} > 1 \]
\[ \text{DCR} = 0.21 < 1 \]

Interaction of Axial, Flexure and Shear Rupture in Plate:
\[ N_u = 53.6 \text{ kips} \]
\[ V_u = 28.1 \text{ kips} \]
\[ \phi R_v = 135.6 \text{ kips} \]
\[ \phi R_u = 81.36 \text{ kips} \]
\[ \phi M_v = 415.14 \text{ kip-in} \]
\[ \frac{N_u}{\phi R_u} = 0.4 > 0.2 \text{ So use AISC 15th Ed. H1-1a} \]

\[ \left( \frac{N_u}{\phi R_u} \right) + \frac{8}{9} \frac{V_u A_v}{(\Phi M_v)^{\frac{3}{2}}} + \left( \frac{V_u}{\Phi R_v} \right)^{\frac{1}{2}} = 0.67 < 1 \]  \hspace{1cm} \text{(AISC 15th Ed. H1-1a)}

\[ \left( \frac{N_u}{2\phi R_v} + \frac{V_u A_v}{\phi M_v} \right)^{\frac{1}{2}} + \left( \frac{V_u}{\phi R_v} \right)^{\frac{1}{2}} = \text{N/A} > 1 \]  \hspace{1cm} \text{(AISC 15th Ed. H1-1b)}

Block Shear Rupture (Beam)
Shear Direction:
\[ A_v = 3.75 \text{ in}^2 \]
\[ A_v = 2.81 \text{ in}^2 \]
\[ A_v = 2.06 \text{ in}^2 \]
\[ U_v = 0.5 \]
\[ \phi = 0.75 \]
\[ \phi R_v = \frac{0.6 F_u A_v + U_v F_a A_v}{\phi R_v} = 120.56 \text{ kips} \]
\[ \phi R_v = \frac{0.6 F_u A_v + U_v F_a A_v}{\phi R_v} = 133.22 \text{ kips} \]
\[ \phi R_v = \frac{0.6 F_u A_v + U_v F_a A_v}{\phi R_v} = 120.56 \text{ kips} \]
\[ \text{DCR} = 0.23 < 1 \]

Block Shear Rupture (Beam)
Axial Direction L Shape:
\[ A_v = 2.63 \text{ in}^2 \]
\[ A_v = 2.06 \text{ in}^2 \]
\[ A_v = 2.81 \text{ in}^2 \]
\[ U_v = 1 \]
\[ \phi = 0.75 \]
\[ \phi R_v = \frac{0.6 F_u A_v + U_v F_a A_v}{\phi R_v} = 205.65 \text{ kips} \]
\[ \phi R_v = \frac{0.6 F_u A_v + U_v F_a A_v}{\phi R_v} = 216.96 \text{ kips} \]
\[ \phi R_v = \frac{0.6 F_u A_v + U_v F_a A_v}{\phi R_v} = 205.65 \text{ kips} \]
\[ \text{DCR} = 0.18 < 1 \]

Block Shear Rupture (Beam)
Axial Direction U Shape:
\[ A_v = 5.25 \text{ in}^2 \]
\[ A_v = 4.13 \text{ in}^2 \]
\[ A_v = 2.44 \text{ in}^2 \]
\[ U_v = 1 \]
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\[ \Phi = 0.75 \]

\[ \phi R_u = \phi 0.6 F_u A_n + U_{bu} F_u A_n \leq \phi 0.6 F_u A_n + U_{bu} F_u A_n \]

\[ \phi 0.6 F_u A_n + U_{bu} F_u A_n = 226.43 \text{ kips} \]

\[ \phi 0.6 F_u A_n + U_{bu} F_u A_n = 249.04 \text{ kips} \]

\[ \frac{V_u}{\phi R_{v1v}} + \frac{N_u}{\phi R_{v01v}} = 0.09 < 1 \]

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

\[ V_u = 28.1 \text{ kips} \]
\[ N_u = 37.5 \text{ kips} \]
\[ \phi R_{v1v} = 120.56 \text{ kips} \]
\[ \phi R_{v01v} = 205.65 \text{ kips} \]
STEEL DRAG BEAM CONNECTION CALCULATION

WHAT TYPE OF CONNECTION IS BEING USED? WELDED

MATERIAL AND CONFIGURATION

Embed Plate: Height = 16 in
Width = 8 in
\( t = \frac{5}{8} \) in
\( F_r = 36 \) ksi
\( F_u = 58 \) ksi
Shear Plate: \( t = 11.5 \) in
\( F_r = 11.5 \) ksi
Weld to Embed Plate = \( 5/16 \) in
\( F_r = 36 \) ksi
\( F_u = 58 \) ksi
Rebar: Size = #7
\# of Bars per Row = 2
\# of Rows = 4
Vertical Spacing = 3.83 in
\( F_y = 60 \) ksi
Welds: \( D = 5/16 \) in
\( L_{wy} = 11.5 \) in
\( L_{wx} = 8 \) in

Beam Size:
<table>
<thead>
<tr>
<th>W16x36</th>
</tr>
</thead>
<tbody>
<tr>
<td>( t_f = 0.43 )</td>
</tr>
<tr>
<td>( t_w = 0.3 )</td>
</tr>
<tr>
<td>( b_f = 6.99 )</td>
</tr>
<tr>
<td>( d = 15.9 )</td>
</tr>
<tr>
<td>( A_g = 10.6 )</td>
</tr>
</tbody>
</table>

APPLIED LOADS

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

Drag Tensile Load = \( \frac{0.7}{4} \times \text{Drag Load} \) (LRFD/Per Anchor)

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

\[
R = \sqrt{R_U^2 + R_D^2}
\]

\( R_U = 176.29 \) kips
\( \theta = 90 \)
REBAR DESIGN

TENSILE CAPACITY OF SINGLE BAR

\[ A_t = 0.6 \text{ in}^2 \]
\[ \phi = 0.75 \]  
\[ \phi N_t = 27.0 \text{ kips} \]  
\[ \text{(ACI 318-19 17.5.3a)} \]

SHEAR CAPACITY OF SINGLE BAR

\[ A_s = 0.6 \text{ in}^2 \]
\[ \phi = 0.65 \]  
\[ \phi V_s = 23.4 \text{ kips} \]  
\[ \text{(ACI 318-19 17.5.3a)} \]

TENSION & SHEAR INTERACTION

\[ N_t = 22.0 \text{ kips} \]  
\[ V_t = 0.0 \text{ kips} \]  
\[ \phi N_t = 0.82 \]  
\[ \phi V_t = 0 \]  
\[ \left( \frac{N_t}{\phi N_t} \right) + \left( \frac{V_t}{\phi V_t} \right) = 0.82 < 1.2 \]

EMBED PLATE DESIGN

EMBED PLATE THICKNESS

\[ L = 2 \text{ in} \]
\[ b = 4.92 \text{ in} \]
\[ T1 = 44.1 \text{ kips} \]  
\[ M_n = 14.38 \text{ K-in} \]  
\[ (AISC 15th Ed. J4-3) \]
\[ Z = \frac{b T1}{4} \]  
\[ Z = 0.48 \text{ in}^2 \]  
\[ \phi M_n = \phi F_y Z \]  
\[ \phi = 0.9 \]  
\[ \phi M_n = 15.56 \text{ K-in} \]  
\[ 
\text{DCR} = 0.71 < 1 
\]

Shear Yield:

\[ A_w = 8.13 \text{ in}^2 \]
\[ \phi R_n = \phi 0.6 F_y A_w \]  
\[ \phi = 0.75 \]  
\[ \phi R_n = 131.63 \text{ kips} \]  
\[ 
\text{DCR} = 0 < 1 
\]

STRENGTH OF WELD

\[ \mu = 1.0 + 0.5 \sin^{1/3} \theta \]  
\[ \text{(AISC 15th Ed. J2-5)} \]
\[ \Theta = 90 \]  
\[ \mu = 1.5 \]  
\[ R_n (1.392 \text{ kip/in}) D (\mu(2 \text{ sides})) \]  
\[ R_n = 240.12 \text{ kips} \]  
\[ \text{(AISC 15th Ed. B-2a)} \]
\[ 
\text{DCR} = 0.73 < 1 
\]
STRENGTH OF WELDED CONN.

RESULTANT LOAD

\[ R_n = \sqrt{N^2_U + V^2_n} \]
\[ \theta = 90 \]
\[ R_n = 176.29 \text{ kips} \]

WELD STRENGTH

\[ k = \frac{8}{11.5} = 0.7 \]
\[ d = 0.20 \text{ in} \]
\[ e = 6.67 \text{ in} \]
\[ e_x = 2.33 \text{ in} \]
\[ e_y = 0.58 \]
\[ C = 4.49 \]
\[ \phi R_n = \phi C C \| D I \]  
\[ \Phi = 0.75 \]
\[ C_1 = 1 \]  
\[ (AISC 15^{th} Ed. T.8-3) \]

Gravity Load: \[ \phi R_n = 193.52 \text{ kips} \]
\[ \text{DCR} = 0.05 < 1 \]

Drag Load: \[ 269.24 \text{ kips} \]
\[ \text{DCR} = 0.65 < 1 \]

BEAM CHECKS

Shear Rupture of Beam Web:
\[ t_{min} = \frac{3.09 D}{F_U} \]  
\[ t_{min} = 0.01 \text{ in} \]
\[ \text{DCR} = 0.05 < 1 \]

Shear Yielding:
\[ A_s = 4.69 \text{ in}^2 \]
\[ \phi R_y = \phi 0.6 F_y A_{s} \]  
\[ \Phi = 0.65 \]
\[ 140.72 \text{ kips} \]
\[ \text{DCR} = 0 < 1 \]

Tensile Yielding:
\[ A_s = 10.6 \text{ in}^2 \]
\[ \phi R_y = \phi F_y A_{s} \]  
\[ \Phi = 0.9 \]
\[ 477 \text{ kips} \]

\[ (AISC 15^{th} Ed. J4-1) \]
DCR = 0.37 <1

Tensile Rupture:

\[ A_s = 10.6 \text{ in}^2 \]

\[ U = 1 - (x_{bar}/l) \]

\[ \tau = \frac{2(h - 2t_e)}{8h} \frac{t_e^2}{t_e^2 + \phi_t \cdot (d - 2t_e)} \]

\[ x_{\text{bar}} = 1.04 \text{ in} \]

\[ U = 0.87 \]

\[ \phi R_n = \phi F_u A_s \]

\[ \Phi = 0.75 \]

\[ \Phi R_n = 449.78 \text{ kips} \]

DCR = 0.39 <1

Block Shear Rupture:

\[ A_s = 4.72 \text{ in}^2 \]

\[ A_{se} = 4.72 \text{ in}^2 \]

\[ A_{mn} = 3.39 \text{ in}^2 \]

\[ U_{mn} = 1 \]

\[ \Phi = 0.75 \]

\[ \phi R_n = \phi 0.6 F_u A_{se} + U_{bn} F_u A_{mn} \leq \phi 0.6 F_u A_{se} + U_{bn} F_u A_{mn} \]

\[ \phi 0.6 F_u A_{se} + U_{bn} F_u A_{mn} = \]

326.71 kips

358.57 kips

DCR = 0.38 <1

**PLATE CHECKS**

Shear Rupture of Plate:

\[ t_{\text{min}} = \frac{3.09 D}{F_u} \]

\[ t_{\text{min}} = 0.02 \text{ in} \]

DCR = 0.03 <1

Flexure Yield:

\[ \phi M_n = \phi F_u Z \]

\[ Z = \frac{t e^2}{t} \]

\[ Z = 16.53 \text{ in}^4 \]

\[ \Phi = 0.9 \]

\[ \phi M_n = 535.61 \text{ k-in} \]

\[ \phi R_n = 80.25 \text{ kips} \]

DCR = 0 <1

Lateral-Torsional Buckling:

\[ \frac{0.08E}{F_{cr}} = 64.44 \]

\[ \frac{1.9E}{F_{cr}} = 1530.56 \]

\[ \frac{Ld}{F} = 368 \]

\[ \frac{0.08E}{F} < \frac{Ld}{F} < \frac{1.9E}{F} \]

So use AISC 15th Ed. F11-2b
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\[
F_r = \frac{t}{f} \cdot F_y
\]

\[
M_n = C_n \left[ 1.52 - 0.274 \left( \frac{L_d}{t} \right) \frac{F_r}{E} \right] M_y
\]

\[
C_n = \frac{12.5 M_{ex}}{2.5 M_y + 3 M_n + 4 M_e + 3 M_c}
\]

\[
M_y = 595.13 \text{ kip-in}
\]

\[
C_n = 1.67
\]

\[
\Phi = 0.9
\]

\[
\phi, M_n = 1245.15 \text{ kip-in}
\]

\[
\phi, R_n = 186.57 \text{ kips}
\]

\[
\text{DCR} = 0 < 1
\]

Shear Yielding:

\[
A_v = 5.75 \text{ in}^2
\]

\[
\phi R_v = \phi 0.6 F_y A_v
\]

\[
\Phi = 1
\]

\[
\phi, R_v = 124.2 \text{ kips}
\]

\[
\text{DCR} = 0 < 1
\]

Tensile Yielding:

\[
A = 5.75 \text{ in}^2
\]

\[
\phi R_s = \phi F_y A_g
\]

\[
\Phi = 0.9
\]

\[
\phi, R_s = 186.3 \text{ kips}
\]

\[
\text{DCR} = 0.95 < 1
\]

Interaction of Axial, Flexure, and Shear Yielding in Plate:

\[
N_s = 176.3 \text{ kips}
\]

\[
V_u = 0.0 \text{ kips}
\]

\[
\phi R_{nu} = 186.3 \text{ kips}
\]

\[
\phi, R_{nv} = 80.25 \text{ kips}
\]

\[
\phi, M_{nv} = 535.61 \text{ kip-in}
\]

\[
\frac{N_s}{\phi R_{nv}} = 0.95 > 0.2 \quad \text{So use AISC 15th Ed. H1-1a}
\]

\[
\left[ \frac{N_s}{\phi R_{nv}} + \frac{8}{9} \left( \frac{V_u}{\phi M_{nv}} \right)^2 + \left[ \frac{V_u}{\phi R_{nv}} \right]^2 \right] = 0.9 < 1
\]

\[
\left[ \frac{N_s}{2 \phi R_{nv}} + \frac{V_u}{\phi M_{nv}} \right]^2 + \left[ \frac{V_u}{\phi R_{nv}} \right]^2 = N/A > 1
\]

Flexure Rupture:

\[
Z = \frac{rD^2}{2}
\]

\[
Z_{av} = 16.53 \text{ in}^4
\]

\[
\Phi = 0.75
\]

\[
\phi, M_{av} = 719.11 \text{ k-in}
\]

\[
\phi, R_{av} = 107.75 \text{ kips}
\]

\[
\text{DCR} = 0 < 1
\]

Shear Rupture:

\[
A_{nv} = 5.75 \text{ in}^2
\]

\[
\phi R_{nv} = \phi 0.6 F_y A_g
\]

\[
\Phi = 0.75
\]

\[
\phi, R_{nv} = 150.08 \text{ kips}
\]

\[
\text{DCR} = 0 < 1
\]
Tensile Rupture: 
\[ A_t = 5.75 \text{ in}^2 \]
\[ U = 1 \]
\[ \phi R_n = \phi F_u A_t \]
\[ \phi = 0.75 \]
\[ R_n = 250.13 \text{ kips} \]
\[ DCR = 0.7 < 1 \]

Interaction of Axial, Flexure and Shear Rupture in Plate:
\[ N_u = 176.3 \text{ kips} \]
\[ V_u = 0.0 \text{ kips} \]
\[ \phi R_n = 250.13 \text{ kips} \]
\[ \phi R_{sw} = 107.75 \text{ kips} \]
\[ \phi M_n = 719.11 \text{ kip-in} \]
\[ \frac{N_u}{\phi R_n} = 0.7 > 0.2 \]

So use AISC 15th Ed. H1-1a

\[
\left[ \frac{N_u}{\phi R_n} + \frac{8}{9} \left( \frac{V_u a}{\phi M_n} \right)^2 + \left( \frac{V_u}{\phi R_n} \right)^2 \right] = 0.5 < 1
\]

\[
\left( \frac{N_u}{2\phi R_n} + \frac{V_u a}{\phi M_n} \right)^2 + \left( \frac{V_u}{\phi R_n} \right)^2 = N/A > 1
\]
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
- F_u = 58 ksi

Shear Plate:
- L_y = 5 in
- L_t = 13.5 in
- t = 1/2 in
- Weld to Embed Plate = 3/16 in
- F_y = 36 ksi
- F_u = 58 ksi

Rebar:
- Size = #7
- # of Bars per Row = 2
- # of Rows = 4
- Vertical Spacing = 4.5 in
- F_y (Rebar) = 60 ksi

Welds:
- D = 5/16 in
- LWy = 13.5 in
- LWx = 4 in

Beam Size:

<table>
<thead>
<tr>
<th>Beam Size</th>
<th>Width</th>
<th>Height</th>
<th>Thickness</th>
</tr>
</thead>
<tbody>
<tr>
<td>W18x50</td>
<td>18</td>
<td>50</td>
<td>0.57</td>
</tr>
</tbody>
</table>

APPLIED LOADS

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

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

\[ R_u = \sqrt{F_u^2 + N_u^2} \]

\[ \Theta = 49.35^\circ \]
REBAR DESIGN

TENSILE CAPACITY OF SINGLE BAR

\[ A_t = 0.6 \text{ in}^2 \]

\[ \phi = 0.75 \]

\[ \Phi N_t = 27.0 \text{ kips} \]

SHEAR CAPACITY OF SINGLE BAR

\[ A_s = 0.6 \text{ in}^2 \]

\[ \phi = 0.65 \]

\[ \Phi V_s = 23.4 \text{ kips} \]

TENSION & SHEAR INTERACTION

\[ N_{int} = 16.4 \text{ kips} \]

\[ \frac{N_{int}}{\Phi N_t} = 0.61 \]

\[ \frac{V_{int}}{\Phi V_s} = 0.39 \]

\[ \left( \frac{N_{int}}{\Phi N_t} \right) + \left( \frac{V_{int}}{\Phi V_s} \right) = 0.99 < 1.2 \]

EMBED PLATE DESIGN

EMBED PLATE THICKNESS

\[ L = 2 \text{ in} \]

\[ b = 5.25 \text{ in} \]

\[ T1 = 32.8 \text{ kips} \]

\[ M_n = \frac{TL^4}{8} \]

\[ M_n = 8.21 \text{ K-in} \]

Flexure Yield:

\[ Z = \frac{b^2 t}{4} \]

\[ Z = 0.33 \text{ in}^2 \]

\[ \phi M_n = \phi F_y Z \]

\[ \phi = 0.9 \]

\[ \phi M_n = 10.63 \text{ K-in} \]

\[ DCR = 0.77 < 1 \]

Shear Yield:

\[ A_{sv} = 7.5 \text{ in}^2 \]

\[ \phi R_n = \phi 0.6 F_y A_{sv} \]

\[ \phi = 0.75 \]

\[ \phi R_n = 121.5 \text{ kips} \]

\[ DCR = 0.6 < 1 \]

STRENGTH OF WELD

\[ \mu = 1.0 + 0.5 \sin^{1/5} \theta \]

\[ \theta = 49.95 \]

\[ \mu = 1.33 \]

\[ R_n = (1.392 \text{ kip/in}) \cdot D \cdot \mu \cdot 2 \text{ sides} \]

\[ R_n = 150.51 \text{ kips} \]

\[ DCR = 0.75 < 1 \]
STRENGTH OF WELDED CONN.

RESULTANT LOAD

\[ R_n = \sqrt{V_u^2 + N_u^2} \]
\[ \Theta = 49.95 \] kips

WELD STRENGTH

\[ \frac{kl}{l} = \frac{4}{13.5} = k = 0.3 \]
\[ x = 0.06 \]
\[ xl = 0.74 \text{ in} \]
\[ e_t = 4.26 \text{ in} \]
\[ a = 0.32 \]
\[ C = 3.27 \]
\[ \phi R_u = \phi C C_t D_l \]  
\[ \phi = 0.75 \]
\[ C_t = 1 \]

Gravity Load: \[ \phi R_u = 165.51 \text{ kips} \]
DCR = \[ 0.44 < 1 \]

Drag Load: \[ \phi R_u = 98.08 \]
DCR = \[ 0.88 < 1 \]

BEAM CHECKS

Shear Rupture of Beam Web:
\[ t_{min} = \frac{3.09 D}{F_U} \]  
\[ t_{min} = 0.01 \text{ in} \]
DCR = \[ 0.04 < 1 \]

Shear Yielding:
\[ A_s = 6.39 \text{ in}^2 \]
\[ \phi R_s = \phi 0.6 F_s A_s \]  
\[ \phi = 191.7 \text{ kips} \]
DCR = \[ 0.38 < 1 \]

Tensile Yielding:
\[ A_p = 14.7 \text{ in}^2 \]
\[ \phi R_p = \phi F_y A_p \]  
\[ \phi = 661.5 \text{ kips} \]
DCR = 0.13 < 1

Tensile Rupture:

\[ A_v = 14.7 \text{ in}^2 \]

\[ U = 1 - \frac{(x_{\text{bar}}/l)}{l} \]

\[ \Phi = \frac{2\sqrt{2}t_f + t_s^2(d - 2t_f)}{8\gamma_f t_f + 4t_s(d - 2t_f)} \]

\[ x_{\text{bar}} = 1.14 \text{ in} \]

\[ U = 0.72 \]

\[ \Phi R_v = \Phi F_u A_y \quad (AISC 15^{th} \text{ Ed. J4-2}) \]

\[ \Phi = 0.75 \]

\[ \Phi R_v = 512.48 \text{ kips} \]

DCR = 0.17 < 1

Block Shear Rupture:

\[ A_v = 2.84 \text{ in}^2 \]

\[ A_{vn} = 2.84 \text{ in}^2 \]

\[ A_{nt} = 4.79 \text{ in}^2 \]

\[ U_{nt} = 1 \]

\[ \Phi = 0.75 \]

\[ \Phi R_v = 375.41 \text{ kips} \]

\[ \Phi 0.6 F_v A_{vn} + U_{bn} F_u A_{nt} = 394.58 \text{ kips} \]

\[ \frac{0.6 F_v A_{vn} + U_{bn} F_u A_{nt}}{\Phi R_v} = 375.41 \text{ kips} \]

DCR = 0.16 < 1

PLATE CHECKS

Shear Rupture of Plate:

\[ t_{\text{min}} = \frac{3.09 D}{F_u} \quad (AISC 15^{th} \text{ Ed. 8-21}) \]

\[ t_{\text{min}} = 0.02 \text{ in} \]

DCR = 0.03 < 1

Flexure Yield:

\[ \Phi M_n = \Phi F_u Z \quad (AISC 15^{th} \text{ Ed. F11-1}) \]

\[ Z = \frac{r \epsilon o \sigma}{k} \]

\[ Z = 22.78 \text{ in}^4 \]

\[ \Phi = 0.9 \]

\[ \Phi M_n = 738.11 \text{ k-in} \]

\[ \Phi R_v = 173.37 \text{ kips} \]

DCR = 0.42 < 1

Lateral-Torsional Buckling:

\[ \frac{0.08E}{F_v} = 64.44 \]

\[ \frac{1.9E}{F_v} = 1530.56 \]

\[ \frac{L_d}{P} = 216 \]

\[ \frac{0.08E}{L_d} < \frac{1.9E}{P} \quad \text{So use AISC 15^{th} Ed. F11-2b} \]
$F_r \cdot \phi = F_r,$

$$M_n = C_n \left[ 1.52 \cdot 0.274 \left( \frac{L_d}{t} \right) \frac{F_r}{E} \right] M_t$$  \hspace{1cm} (AISC 15th Ed. F11-2)

$$C_n = \frac{12.5 M_{yes}}{2.5 M_{yes} + 3M_n + 4M_e + 3M_c}$$  \hspace{1cm} (AISC 15th Ed. F1-1)

$M_t = 820.13$ kip-in

$C = 1.67$

$\phi = 0.9$

$\phi M_n = 1779.5$ kip-in

$\phi R_n = 417.97$ kips

$DCR = \frac{0.17}{<1}$

Shear Yielding:

$A_s = 6.75$ in$^2$

$\phi R_s = 0.6 F_r A_s$

$\phi = \frac{1}{1}$

$\phi R_s = 145.8$ kips

$DCR = \frac{0.5}{<1}$

Tensile Yielding:

$A_g = 6.75$ in$^2$

$\phi R_g = 0.9 F_r A_g$

$\phi = 0.9$

$\phi R_g = 218.7$ kips

$DCR = \frac{0.39}{<1}$

Interaction of Axial, Flexure and Shear Yielding in Plate:

$N_b = 86.0$ kips

$V_c = 72.3$ kips

$\phi R_{np} = 218.7$ kips

$\phi R_{np} = 145.8$ kips

$\phi M_{np} = 738.11$ kip-in

$$\frac{N_b}{\Phi R_{mp}} = 0.39 > 0.2 \hspace{0.5cm} \text{So use AISC 15th Ed. H1-1a}$$

$$\left( \frac{N_b}{\Phi R_{mp}} \right) + \frac{\delta}{9} \left( \frac{V_{c}}{\Phi M_{np}} \right)^2 + \left( \frac{V_{c}}{\Phi R_{np}} \right)^2 = 0.83 < 1$$  \hspace{1cm} (AISC 15th Ed. H1-1a)

$$\left( \frac{N_b}{2\phi R_{np}} \right) + \left( \frac{V_{c}}{\phi M_{np}} \right)^2 + \left( \frac{V_{c}}{\phi R_{np}} \right)^2 = N/A > 1$$  \hspace{1cm} (AISC 15th Ed. H1-1b)

Flexure Rupture:

$A_g = 6.75$ in$^2$

$\phi R_g = 0.9 F_r A_g$

$\phi = 0.75$

$\phi R_{np} = 990.98$ k-in

$\phi R_{np} = 232.76$

$DCR = \frac{0.31}{<1}$

Shear Rupture:

$A_s = 6.75$ in$^2$

$\phi R_s = 0.6 F_r A_s$

$\phi = 0.75$

$\phi R_s = 176.18$ kips

$DCR = \frac{0.41}{<1}$
Tensile Rupture: 

\[ A_t = 6.75 \text{ in}^2 \]
\[ U = 1 \]
\[ \phi R_n = \frac{\phi F_{u} A_t}{\theta} = 0.75 \]
\[ \phi R_n = 293.63 \text{ kips} \]
\[ \text{DCR} = 0.29 < 1 \]

Interaction of Axial, Flexure and Shear Rupture in Plate:
\[ N_u = 86.0 \text{ kips} \]
\[ V_u = 72.3 \text{ kips} \]
\[ \phi R_{np} = 293.63 \text{ kips} \]
\[ \phi R_{nv} = 176.18 \text{ kips} \]
\[ \phi M_n = 990.98 \text{ kip-in} \]
\[ \frac{N_u}{\phi R_{np}} = 0.29 > 0.2 \text{ So use AISC 15th Ed. H1-1a} \]

\[ \frac{N_u}{\phi R_{np}} + \frac{8}{9} \left( \frac{V_u a}{\phi M_n} \right)^2 + \frac{V_u}{\phi R_{np}} = 0.49 < 1 \]  
\[ (\text{AISC 15th Ed. H1-1a}) \]

\[ \left( \frac{N_u}{2\phi R_{np}} + \frac{V_u a}{\phi M_n} \right)^2 + \left( \frac{V_u}{\phi R_{np}} \right)^2 = N/A > 1 \]  
\[ (\text{AISC 15th Ed. H1-1b}) \]
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
$\ell = \frac{1}{2}$ in
$F_r = 36$ ksi
$F_u = 58$ ksi

Shear Plate:  
$\ell = 5$ in
$t = 3/8$ in
Weld to Embed Plate = 3/16 in
$F_r = 36$ ksi
$F_u = 58$ ksi

Rebar:  
Size = #7
# of Bars per Row = 2
# of Rows = 4
Vertical Spacing = 5.5 in
F (Rebar) = 60 ksi

Bolts:  
$\varnothing = 1$ in
# of Rows = 5
Vert. Spacing = 3.15 in
Horiz. Spacing = 3 in
Fnt (Bolts) = 45 ksi
Fnv (Bolts) = 27 ksi

Beam Size:  
W21X93
$t_f = 0.93$
$tw = 0.58$
$b_f = 8.42$
$d = 21.6$
$A_g = 27.3$

APPLIED LOADS

Drag Load = 45.0 kips
Gravity Load = 70.0 kips

Drag Tensile Load = 8.0 kips
Gravity Tensile Load = 6.8 kips
Total Tensile Load = 14.8 kips
Gravity Shear Load = 8.8 kips

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RESULTANT LOAD

\[ R_u = \sqrt{V_u^2 + N_u^2} \]
\[ R_u = 95.04 \text{ kips} \]
\[ \theta = 42.56 \]

REBAR DESIGN

TENSILE CAPACITY OF SINGLE BAR

\[ A_s = 0.6 \text{ in}^2 \]
\[ \Phi = 0.75 \]
\[ \Phi N_u = 27.0 \text{ kips} \]

SHEAR CAPACITY OF SINGLE BAR

\[ A_s = 0.6 \text{ in}^2 \]
\[ \Phi = 0.65 \]
\[ \Phi V_u = 23.4 \text{ kips} \]

TENSION & SHEAR INTERACTION

\[ N_{ux}/\Phi N_u = 0.55 \]
\[ V_{ux}/\Phi V_u = 0.37 \]
\[ (N_{ux}/\Phi N_u) + (V_{ux}/\Phi V_u) = 0.92 < 1.2 \]

EMBED PLATE DESIGN

EMBED PLATE THICKNESS

\[ L = 2 \text{ in} \]
\[ b = 5.75 \text{ in} \]
\[ T1 = 29.6 \text{ kips} \]
\[ M_u = \frac{T1^2}{8} \]
\[ M_u = 7.41 \text{ K-in} \]

Flexure Yield:

\[ Z = \frac{\Phi V_u}{4} \]
\[ Z = 0.36 \text{ in}^3 \]
\[ \phi M_u = \phi V_u \frac{Z}{\Phi} \]
\[ \phi M_u = 11.64 \text{ K-in} \]
\[ DCR = 0.64 < 1 \]

Shear Yield:

\[ A_s = 9 \text{ in}^2 \]
\[ \phi R_u = 0.6 \frac{F_y A_s}{\Phi} \]
\[ \phi R_u = 145.8 \text{ kips} \]
\[ DCR = 0.48 < 1 \]

STRENGTH OF WELD

\[ \mu = 1.0 + 0.5 \sin^{1.5} \theta \]
\[ \theta = 42.56 \]
\[ \mu = 1.28 \]
\[ R_u = (1.392 \text{ kip/in}) D \mu(2 \text{ sides}) \]
\[ R_u = 176.14 \text{ kips} \]
\[ DCR = 0.54 < 1 \]
STRENGTH OF BOLTED CONN.

RESULTANT LOAD

\[ R_u = \sqrt{V_u^2 + N_u^2} \]

\[ R_u = 95.04 \text{ kips} \]

\[ \Theta = 42.56 \]

\[ e = 4.5 \]

\[ C = 6.92 \]

(AISC 15th Ed. T.10-9)

BEAM WEB STRENGTH

Bolt Shear:

\[ \phi r_u = \phi F_u A_b \]

\[ \Theta = 0.75 \]

\[ \Phi r_u = 15.9 \text{ kips/bolt} \]

Bolt Bearing Strength:

\[ \phi r_u = \phi 3.0 \ell_i t F_u \]

\[ \Theta = 0.75 \]

\[ \Phi r_u = 84.83 \text{ kips/bolt} \]

Bolt Tearout Strength:

\[ \phi r_u = \phi 1.5 l_{ct} t F_u \]

\[ \Theta = 0.75 \]

\[ \Phi r_u = 60.97 \text{ kips/bolt} \]

Governing\( \Phi r_u = 15.9 \text{ kips} \)

\[ \phi R_u = C \phi r_u \]

\[ \Phi R_u = 110.07 \text{ kips} \]

\[ \text{DCR} = 0.86 < 1 \]

SHEAR PLATE STRENGTH

Bolt Bearing Strength:

\[ \phi r_u = \phi 3.0 \ell_i t F_u \]

\[ \Theta = 0.75 \]

\[ \Phi r_u = 48.94 \text{ kips/bolt} \]

Bolt Tearout Strength:

\[ \phi r_u = \phi 1.5 l_{ct} t F_u \]

\[ \Theta = 0.75 \]

\[ \Phi r_u = 35.17 \text{ kips/bolt} \]

Governing\( \phi r_u = 15.9 \text{ kips} \)

\[ \phi R_u = C \phi r_u \]

\[ \Phi R_u = 110.07 \text{ kips} \]

\[ \text{DCR} = 0.86 < 1 \]

BEAM CHECKS

Shear Yielding:

\[ A_{pv} = 12.53 \text{ in}^2 \]

\[ \phi R_{pv} = \phi 0.6 F_y A_{pv} \]

(AISC 15th Ed. J4-3)
**SAN LUIS OBISPO | PASADENA | SANTA ROSA**

2020 CBC / 2019 IBC Drag Analysis - Version 0.10

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

\[ \phi R_e = \phi 0.6 F_y A_g \]
\[ \phi = \frac{1}{2} \]
\[ \phi R_e = 375.84 \text{ kips} \]

DCR = **0.19** <1

**Tensile Yielding:**
\[ A_s = 27.3 \text{ in}^2 \]
\[ \phi R_e = \phi F_y A_s \]
\[ \phi = 0.9 \]
\[ \phi R_e = 1228.5 \text{ kips} \]

DCR = **0.05** <1

**Tensile Rupture:**
\[ U = 1 - (x_{bar}/l) \]
\[ x_{bar} = 1.28 \text{ in} \]
\[ u = 0.57 \]
\[ \phi R_e = \phi F_y A_s \]
\[ \phi = 0.75 \]
\[ \phi R_e = 677.99 \text{ kips} \]

DCR = **0.09** <1

**Block Shear Rupture:**
\[ A_g = 24.22 \text{ in}^2 \]
\[ U = 1 - (x_{bar}/l) \]
\[ x_{bar} = 1.28 \text{ in} \]
\[ u = 0.57 \]
\[ \phi R_e = \phi F_y A_s \]
\[ \phi = 0.75 \]
\[ \phi R_e = 488.65 \text{ kips} \]

DCR = **0.09** <1

**PLATE CHECKS**

**Maximum Plate Thick:**
\[ t_{max} = \frac{E' M_{max}}{F_y' A} \]

\[ M_{max} = \frac{F_{cy}}{0.8} (\phi \delta' C') \]
\[ C' = 38.7 \]
\[ M_{max} = 911.85 \text{ k-in} \]
\[ t = 0.4 \text{ in} \]

**Flexure Yield:**
\[ \phi M_y = \phi F_y Z \]
\[ Z = \frac{t^2 D_y}{2} \]
\[ Z = 25.52 \text{ in}^2 \]
\[ \phi = 0.9 \]
\[ \phi M_y = 826.96 \text{ k-in} \]
\[ \phi R_e = 183.77 \text{ kips} \]
Lateral-Torsional Buckling:

\[
\frac{0.08E}{F_y} = 64.44 \\
\frac{1.9E}{F_y} = 1530.56 \\
\frac{L \cdot d}{t} = 352
\]

So use AISC 15th Ed. F11-2b

\[
M_y = C_y[1.52 - 0.274 \left( \frac{L \cdot d}{t} \right) \frac{F_y}{E}] M_y
\]

(AISC 15th Ed. F11-2)

\[
C_y = \frac{12.5M_{plas}}{2.5M_y + 3M_x + 4M_w + 3M_c}
\]

(AISC 15th Ed. F1-1)

\[
\phi M_x = 612.56 \text{ kip-in} \\
\phi M_y = 1286.63 \text{ kip-in} \\
\phi R_u = 285.92 \text{ kips}
\]

DCR = 0.38 < 1

Shear Yielding:

\[
A_y = 6.19 \text{ in}^2 \\
\phi R_y = \phi F_y A_y \\
\phi R_u = 133.65 \text{ kips}
\]

DCR = 0.24 < 1

Tensile Yielding:

\[
A_t = 6.19 \text{ in}^2 \\
\phi R_t = \phi F_y A_t \\
\phi R_u = 200.48 \text{ kips}
\]

DCR = 0.32 < 1

Interaction of Axial, Flexure and Shear Yielding in Plate:

\[
N_u = 64.3 \text{ kips} \\
V_u = 70.0 \text{ kips} \\
\phi R_u = 200.48 \text{ kips} \\
\phi M_u = 826.96 \text{ kip-in}
\]

\[
\frac{N_u}{\phi R_u} = 0.32 > 0.2 \\
\left( \frac{N_u}{\phi R_u} \right) = 0.71 < 1
\]

(AISC 15th Ed. H1-1a)

Flexure Rupture:

\[
Z_{net} = \frac{d^2}{4} \left[ \frac{1}{4} \left( d_t + \frac{1}{4s} \text{ in} \right) \left( \eta^2 - 1 \right) + \left( d_r + \frac{1}{4s} \text{ in} \right)^2 \right] \\
\phi M_n = \phi F_y Z_{net}
\]

(AISC 15th Ed. 9-4)

\[
Z_{net} = 17.89 \text{ in}^3 \\
\phi = 0.75
\]
$$\phi M_n = 778.09 \text{ k-in}$$
$$\phi R_u = 172.91$$

DCR = 0.4 < 1

Shear Rupture:
$$A_n = 4.2 \text{ in}^2$$
$$\phi R_u = \phi F_u A_n / 0.75$$
$$\phi R_u = 109.5 \text{ kips}$$

DCR = 0.64 < 1

Tensile Rupture:
$$A_n = 4.2 \text{ in}^2$$
$$U = 1$$
$$\phi R_u = \phi F_u A_n / 0.75$$
$$\phi R_u = 182.5$$

DCR = 0.38 < 1

Interaction of Axial, Flexure and Shear Rupture in Plate:
$$N_u = 64.3 \text{ kips}$$
$$V_u = 70.0 \text{ kips}$$
$$\phi R_u = 182.5 \text{ kips}$$
$$\phi M_n = 778.09 \text{ kip-in}$$

$$\frac{N_u}{\phi R_u} = 0.35 > 0.2$$

So use AISC 15th Ed. H1-1a

$$\left[ \frac{N_u}{\phi R_u} + \frac{V_u}{\phi M_n} \right]^{1/2} + \left[ \frac{V_u}{\phi R_u} \right]^{1/2} = 0.92 < 1$$

(AISC 15th Ed. H1-1a)

$$\left[ \frac{N_u}{2\phi R_{uy}} + \frac{V_u}{\phi M_n} \right]^{3/2} + \left[ \frac{V_u}{\phi R_{uy}} \right]^{3/2} = \text{N/A} > 1$$

(AISC 15th Ed. H1-1b)

Block Shear Rupture (Beam):
$$A_w = 5.63 \text{ in}^2$$
$$A_n = 3.84 \text{ in}^2$$
$$A_n = 1.13 \text{ in}^2$$
$$U_n = 0.5$$
$$\phi = 0.75$$

$$\phi R_u = \phi F_u A_n + U_u F_u A_n \leq \phi F_u A_n + U_u F_u A_n$$
$$\phi F_u A_n + U_u F_u A_n = 123.75 \text{ kips}$$

$$\phi 0.6 F_u A_n + U_u F_u A_n = 135.39 \text{ kips}$$

$$\phi 0.6 F_u A_n + U_u F_u A_n = 123.75 \text{ kips}$$

DCR = 0.57 < 1

Block Shear Rupture (Beam):
$$A_w = 1.69 \text{ in}^2$$
$$A_n = 1.13 \text{ in}^2$$
$$A_n = 3.84 \text{ in}^2$$
$$U_n = 1$$
$$\phi = 0.75$$

$$\phi R_u = \phi 0.6 F_u A_n + U_u F_u A_n \leq \phi 0.6 F_u A_n + U_u F_u A_n$$
$$\phi 0.6 F_u A_n + U_u F_u A_n = 255.71 \text{ kips}$$

$$\phi 0.6 F_u A_n + U_u F_u A_n = 257.74 \text{ kips}$$

$$\phi 0.6 F_u A_n + U_u F_u A_n = 255.71 \text{ kips}$$

DCR = 0.18 < 1

Block Shear Rupture (Beam):
$$A_w = 3.38 \text{ in}^2$$
$$A_n = 2.25 \text{ in}^2$$
$$A_n = 3.56 \text{ in}^2$$
$$U_n = 1$$
\[ \Phi = 0.75 \]

\[ \phi R_{bsv} = \phi 0.6 F_e A_v + U_{bs} F_s A_w \leq \phi 0.6 F_e A_v + U_{bs} F_s A_w \]

\[ \phi 0.6 F_e A_v + U_{bs} F_s A_w = 261.3 \text{ kips} \]

\[ \phi 0.6 F_e A_v + U_{bs} F_s A_w = 265.35 \text{ kips} \]

\[ \text{DCR} = 0.17 < 1 \]

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

\[ V_s = 70.0 \text{ kips} \]

\[ N_s = 45.0 \text{ kips} \]

\[ \phi R_{bsv} = 123.75 \text{ kips} \]

\[ \phi R_{bsn} = 255.71 \text{ kips} \]

\[ \left( \frac{V_s}{\phi R_{bsv}} \right)^2 + \left( \frac{N_s}{\phi R_{bsn}} \right)^2 = 0.35 < 1 \]
STEEL DRAG BEAM CONNECTION CALCULATION

WHAT TYPE OF CONNECTION IS BEING USED?

BOLT: DOUBLE ROW  EXTENDED CONFIGURATION

MATERIAL AND CONFIGURATION

<table>
<thead>
<tr>
<th>Embed Plate:</th>
<th>Height = 24 in</th>
</tr>
</thead>
<tbody>
<tr>
<td>Width = 8 in</td>
<td></td>
</tr>
<tr>
<td>t = 3/8 in</td>
<td></td>
</tr>
<tr>
<td>Fy = 36 ksi</td>
<td></td>
</tr>
<tr>
<td>Fu = 58 ksi</td>
<td></td>
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<table>
<thead>
<tr>
<th>Shear Plate:</th>
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<tbody>
<tr>
<td>Lh = 5 in</td>
</tr>
<tr>
<td>t = 1/4 in</td>
</tr>
<tr>
<td>Weld to Embed Plate = 3/16 in</td>
</tr>
<tr>
<td>Fy = 36 ksi</td>
</tr>
<tr>
<td>Fu = 58 ksi</td>
</tr>
</tbody>
</table>

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<thead>
<tr>
<th>Rebar:</th>
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<tbody>
<tr>
<td>Size = #6</td>
</tr>
<tr>
<td># of Bars per Row = 2</td>
</tr>
<tr>
<td># of Rows = 4</td>
</tr>
<tr>
<td>Vertical Spacing = 6.5 in</td>
</tr>
<tr>
<td>Fy (Rebar) = 60 ksi</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Bolts:</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ø = 1 in</td>
</tr>
<tr>
<td># of Rows = 5</td>
</tr>
<tr>
<td>Vert. Spacing = 3.78 in</td>
</tr>
<tr>
<td>Horiz. Spacing = 3 in</td>
</tr>
<tr>
<td>Fnt (Bolts) = 45 ksi</td>
</tr>
<tr>
<td>Fnv (Bolts) = 27 ksi</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Beam Size:</th>
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</thead>
<tbody>
<tr>
<td>W24X84</td>
</tr>
<tr>
<td>tf = 0.77</td>
</tr>
<tr>
<td>tw = 0.47</td>
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<tr>
<td>bf = 9.02</td>
</tr>
<tr>
<td>d = 24.1</td>
</tr>
<tr>
<td>Ag = 24.7</td>
</tr>
</tbody>
</table>

APPLIED LOADS

| Drag Load = 43.6 kips |
| Gravity Load = 52.8 kips |

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

\[ R_x = \sqrt{V_x^2 + N_x^2} \]

\[ \Theta = 49.71 \]
REBAR DESIGN

TENSILE CAPACITY OF SINGLE BAR

\[ A_t = 0.44 \text{ in}^2 \]
\[ \phi = 0.75 \]
\[ \Phi N_t = 19.8 \text{ kips} \]  

ACI 318-19 17.5.3a

SHEAR CAPACITY OF SINGLE BAR

\[ A_s = 0.44 \text{ in}^2 \]
\[ \phi = 0.65 \]
\[ \Phi V_s = 17.2 \text{ kips} \]  

ACI 318-19 17.5.3a

TENSION & SHEAR INTERACTION

\[ N_u = 11.8 \text{ kips} \]
\[ V_u = 6.6 \text{ kips} \]
\[ N_u/\phi N_t = 0.59 \]
\[ V_u/\phi V_s = 0.38 \]

\[ (N_u/\phi N_t) + (V_u/\phi V_s) = 0.98 < 1.2 \]

EMBED PLATE DESIGN

EMBED PLATE THICKNESS

\[ L = 2 \text{ in} \]
\[ b = 6.25 \text{ in} \]
\[ T_1 = 23.5 \text{ kips} \]  

LRFD

\[ M_n = \frac{T_1 t}{8} \]  

AISC 15th Ed. 3-23.16

Flexure Yield:

\[ Z = \frac{b t^2}{4} \]  

\[ \phi M_n = \phi F_y Z \]  

AISC 15th Ed. F11-1

\[ \phi = 0.9 \]
\[ \Phi M_n = 7.12 \text{ k-in} \]

\[ DCR = 0.83 < 1 \]

Shear Yield:

\[ A_n = 7.88 \text{ in}^2 \]
\[ \phi R_n = 0.6 F_y A_n \]  

AISC 15th Ed. J4-3

\[ \phi = 0.75 \]
\[ \Phi R_n = 127.58 \text{ kips} \]

\[ DCR = 0.41 < 1 \]

STRENGTH OF WELD

\[ \mu = 1.0 + 0.5 \sin^{1/2} \theta \]  

AISC 15th Ed. J2-5

\[ \theta = 49.71 \]
\[ \mu = 1.33 \]

\[ R_n = (1.392 \text{ kip/in}) D I \mu (2 \text{ sides}) \]  

AISC 15th Ed. 8-2a

\[ R_n = 217.12 \text{ kips} \]

\[ DCR = 0.38 < 1 \]
STRENGTH OF BOLTED CONN.

RESULTANT LOAD

\[ R_U = \sqrt{V_U^2 + N_U^2} \]
\[ R_U = 81.65 \text{ kips} \]
\[ \Theta = 49.71 \]
\[ \epsilon = 4.5 \]
\[ C = 7.24 \]  
(AISC 15th Ed. J.10-9)

BEAM WEB STRENGTH

Bolt Shear:
\[ \Phi \sigma_n = \Phi F_n A_n \]  
\[ \Theta = 0.75 \]
\[ \Phi \sigma_n = 15.9 \text{ kips/bolt} \]

Bolt Bearing Strength:
\[ \Phi \sigma_n = \Phi 3.0 \delta t F_U \]  
\[ \Theta = 0.75 \]
\[ \Phi \sigma_n = 68.74 \text{ kips/bolt} \]

Bolt Tearout Strength:
\[ \Phi \sigma_n = \Phi 1.5 l \delta t F_U \]  
\[ \Theta = 0.75 \]
\[ \Phi \sigma_n = 49.41 \text{ kips/bolt} \]

Governing \( \Phi \sigma_n = 15.9 \text{ kips} \)
\[ \Phi R_n = C \Phi \sigma_n \]
\[ \Phi R_n = 115.07 \text{ kips} \]

DCR = 0.71 < 1

SHEAR PLATE STRENGTH

Bolt Bearing Strength:
\[ \Phi \sigma_n = \Phi 3.0 \delta t F_U \]  
\[ \Theta = 0.75 \]
\[ \Phi \sigma_n = 32.63 \text{ kips/bolt} \]

Bolt Tearout Strength:
\[ \Phi \sigma_n = \Phi 1.5 l \delta t F_U \]  
\[ \Theta = 0.75 \]
\[ \Phi \sigma_n = 23.45 \text{ kips/bolt} \]

Governing \( \Phi \sigma_n = 15.9 \text{ kips} \)
\[ \Phi R_n = C \Phi \sigma_n \]
\[ \Phi R_n = 115.07 \text{ kips} \]

DCR = 0.71 < 1

BEAM CHECKS

Shear Yielding:
\[ A_v = 11.33 \text{ in}^2 \]
\[ \Phi R_{sv} = \Phi 0.6 F_y A_v \]  
(AISC 15th Ed. J.4-3)
Project: **18412 – Angelo View Revisions – Uberion**

**Tensile Yielding:**  
\[ \phi R_y = \phi F_y A_g \]  
\[ \Phi = 0.9 \]  
\[ \Phi R_y = 1111.5 \text{ kips} \]  
\[ \text{DCR} = 0.06 < 1 \]

**Tensile Rupture:**  
\[ \Phi = 0.9 \]  
\[ \Phi R_y = 602.65 \text{ kips} \]  
\[ \text{DCR} = 0.1 < 1 \]

**Block Shear Rupture:**  
\[ \Phi = 0.75 \]  
\[ \Phi R_y = 487.63 \text{ kips} \]  
\[ \Phi R_y = 497.44 \text{ kips} \]  
\[ \Phi R_y = 487.63 \text{ kips} \]  
\[ \text{DCR} = 0.09 < 1 \]

**PLATE CHECKS**

**Maximum Plate Thick:**  
\[ t_{\text{max}} = \frac{E' M_{\text{max}}}{F_y} \]  
\[ M_{\text{max}} = \frac{F_{\text{max}}}{0.5} (\phi_y C') \]  
\[ C' = 38.7 \]  
\[ t = 0.3 \text{ in} \]

**Flexure Yield:**  
\[ \phi M_y = \phi F_y Z \]  
\[ Z = \frac{t_0 Z_{\phi y}}{2} \]  
\[ Z = 23.77 \text{ in}^4 \]  
\[ \Phi = 0.9 \]  
\[ \phi M_y = 770.01 \text{ k-in} \]  
\[ \phi R_y = 171.11 \text{ kips} \]
Lateral-Torsional Buckling:

\[
\frac{0.08E}{F_y} = 64.44 \\
\frac{1.9E}{F_y} = 1530.56 \\
\frac{L_d d}{t} = 936
\]

\[
\frac{0.08E}{F_y} < \frac{L_d d}{t} < \frac{1.9E}{F_y} \quad \text{So use AISC 15th Ed. F11-2b}
\]

\[
M_u = C_b[1.52 - 0.274 \left( \frac{L_d d}{t} \right) \left( \frac{F_y}{E} \right)] M_y
\]

Shear Yielding:

\[
A_y = 4.88 \text{ in}^2 \\
\phi R_y = \phi F_y A_y \\
\phi R_y = 105.3 \text{ kips}
\]

Tensile Yielding:

\[
A_y = 4.88 \text{ in}^2 \\
\phi R_y = \phi F_y A_y \\
\phi R_y = 157.95 \text{ kips}
\]

Interaction of Axial, Flexure and Shear Yielding in Plate:

\[
N_u = 62.3 \text{ kips} \\
V_u = 52.8 \text{ kips} \\
\phi R_y = 157.95 \text{ kips} \\
\phi R_y = 105.3 \text{ kips} \\
\phi M_y = 770.01 \text{ kip-in}
\]

\[
\frac{N}{\phi R_y} = 0.39 > 0.2 \quad \text{So use AISC 15th Ed. H1-1a}
\]

Flexure Rupture:

\[
Z_{net} = \frac{d^2}{4} \left[ \frac{1}{4} \left( d_h + \frac{1}{2} s \right) \left( \frac{d^2}{E} - 1 \right) + \left( d_h + \frac{1}{2} s \right) \right]
\]

\[
\phi M_y = \phi F_y Z_{net}
\]

\[
\phi = 0.75
\]
\[ \phi M_n = 769.02 \text{ k-in} \]
\[ \phi R_n = 170.89 \]

\[ \text{DCR} = 0.31 < 1 \]

Shear Rupture: \[ A_n = 3.55 \text{ in}^2 \]
\[ \phi R_n = \phi \frac{0.6 F_c A_n}{0.75} = 92.57 \text{ kips} \]
\[ \text{DCR} = 0.57 < 1 \]

Tensile Rupture: \[ A_n = 3.55 \text{ in}^2 \]
\[ U = 1 \]
\[ \phi R_n = \frac{\phi F_c A_n}{0.75} = 154.29 \text{ kips} \]
\[ \text{DCR} = 0.34 < 1 \]

Interaction of Axial, Flexure and Shear Rupture in Plate:
\[ \frac{N_u}{\phi R_n} = 0.4 > 0.2 \] So use AISC 15th Ed. H1-1a

\[ \left( \frac{N_u}{\phi R_n} \right)^2 + \left( \frac{V_u}{\phi M_n} \right)^2 + \left( \frac{U_b}{\phi R_n} \right)^2 = 0.79 < 1 \] (AISC 15th Ed. H1-1a)

\[ \left( \frac{N_u}{2 \phi R_n} + \frac{V_u}{\phi M_n} \right)^2 + \left( \frac{U_b}{\phi R_n} \right)^2 = 0.09 < 1 \] (AISC 15th Ed. H1-1b)

Block Shear Rupture (Beam Shear Direction):
\[ A_n = 4.5 \text{ in}^2 \]
\[ A_n = 3.38 \text{ in}^2 \]
\[ A_n = 0.75 \text{ in}^2 \]
\[ U_b = 0.5 \]
\[ \phi = 0.75 \]
\[ \phi R_n = \phi \frac{0.6 F_c A_n}{0.75} + U_b F_c A_n \leq \phi \frac{0.6 F_c A_n}{0.75} + U_b F_c A_n = 94.65 \text{ kips} \]
\[ \phi R_n = 94.65 \text{ kips} \]
\[ \text{DCR} = 0.56 < 1 \]

Block Shear Rupture (Beam Axial Direction L Shape):
\[ A_n = 1.13 \text{ in}^2 \]
\[ A_n = 0.75 \text{ in}^2 \]
\[ A_n = 3.38 \text{ in}^2 \]
\[ U_b = 1 \]
\[ \phi = 0.75 \]
\[ \phi R_n = \phi \frac{0.6 F_c A_n}{0.75} + U_b F_c A_n \leq \phi \frac{0.6 F_c A_n}{0.75} + U_b F_c A_n = 213.98 \text{ kips} \]
\[ \phi R_n = 213.98 \text{ kips} \]
\[ \text{DCR} = 0.2 < 1 \]

Block Shear Rupture (Beam Axial Direction U Shape):
\[ A_n = 2.25 \text{ in}^2 \]
\[ A_n = 1.5 \text{ in}^2 \]
\[ A_n = 3.13 \text{ in}^2 \]
\[ U_b = 1 \]
Block Shear Rupture (Comb. Axial & Shear U Shape):

\[
\frac{V_u}{\phi R_{bsv}} \leq 0.6 F_y A_g + U_{bs} F_{u A} \leq \frac{N_u}{\phi R_{bsn}}
\]

\[
\phi \leq 0.75
\]

\[
DRC = 0.2 < 1
\]
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
- F_u = 58 ksi

Shear Plate:
- L_y = 5 in
- t = 3/8 in

Weld to Embed Plate:
- 1/16 in
- F_y = 36 ksi
- F_u = 58 ksi

Rebar:
- Size = #7
- # of Bars per Row = 2
- # of Rows = 4
- Vertical Spacing = 6.5 in
- F_y (Rebar) = 60 ksi

Welds:
- D = 5/16 in
- Lwy = 19.5 in
- Lwx = 8 in

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
Gravity Load = 27.9 kips

Drag Tensile Load = 22.4 kips
Gravity Tensile Load = 2.5 kips
Total Tensile Load = 24.9 kips
Gravity Shear Load = 3.5 kips

RESULTANT LOAD

\[
R_u = \frac{\sqrt{V_u^2 + N_u^2}}{F_y}
\]

\[
\theta = 181.73 \text{ kips}
\]

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18412 - Drag (ED-4W).ods

98/103
REBAR DESIGN

TENSILE CAPACITY OF SINGLE BAR

\[ A_t = 0.6 \text{ in}^2 \]

\[ \phi = 0.75 \]

\[ \Phi N_t = 27.0 \text{ kips} \]  

(ACI 318-19 17.5.3a)

SHEAR CAPACITY OF SINGLE BAR

\[ A_s = 0.6 \text{ in}^2 \]

\[ \phi = 0.65 \]

\[ \Phi V_s = 23.4 \text{ kips} \]  

(ACI 318-19 17.5.3a)

TENSION & SHEAR INTERACTION

\[ N_t = 24.9 \text{ kips} \]

\[ V_s = 3.5 \text{ kips} \]

\[ N_t/\Phi N_t = 0.92 \]

\[ V_s/\Phi V_s = 0.15 \]

\[ (N_t/\Phi N_t) + (V_s/\Phi V_s) = 1.07 < 1.2 \]

EMBED PLATE DESIGN

EMBED PLATE THICKNESS

\[ L = 2 \text{ in} \]

\[ b = 6.25 \text{ in} \]

\[ T1 = 49.8 \text{ kips} \]  

(LRFD)

\[ M_n = \frac{T1 \cdot t}{8} \]

\[ M_n = 12.45 \text{ k-in} \]  

(AISC 15th Ed. F11-1)

Flexure Yield:

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

\[ Z = 0.39 \text{ in}^2 \]  

(AISC 15th Ed. F11-1)

\[ \phi M_n = \phi F_y Z \]

\[ \phi = 0.9 \]

\[ \Phi M_n = 12.66 \text{ k-in} \]

DCR = \[ 0.98 < 1 \]

Shear Yield:

\[ A_w = 10.5 \text{ in}^2 \]

\[ \phi R_n = 0.6 F_y A_w \]

\[ \phi = 0.75 \]

\[ \Phi R_n = 170.1 \text{ kips} \]

DCR = \[ 0.16 < 1 \]

STRENGTH OF WELD

\[ \mu = 1.0 + 0.5 \sin^{1.5}\theta \]

\[ \Theta = 81.17 \]

\[ \phi = 1.49 \]

\[ R_n = (1.392 \text{ kip/in}) Dl \mu (2 \text{ sides}) \]

\[ R_n = 242.85 \text{ kips} \]  

(AISC 15th Ed. 8-2a)

DCR = \[ 0.75 < 1 \]
STRENGTH OF WELDED CONN.

RESULTANT LOAD

\[ R_u = \sqrt{V_u^2 + N_u^2} \]
\[ R_u = 181.73 \text{ kips} \]
\[ \Theta = 81.17 \]

WELD STRENGTH

\[ \frac{kl}{l} = \frac{8}{19.5} = k = 0.41 \]
\[ x = 0.09 \]
\[ xl = 1.81 \text{ in} \]
\[ e_x = 7.19 \text{ in} \]
\[ \frac{e_x}{l} = a = 0.37 \]
\[ C = 3.62 \]
\[ \phi R_u = \phi CC_1 Dl \]  
\[ \Phi = 0.75 \]
\[ C_1 = 1 \]  
Gravity Load: \[ \phi R_u = 264.53 \text{ kips} \]
\[ \text{DCR} = 0.11 < 1 \]
Drag Load: \[ \phi R_u = 217.05 \]
\[ \text{DCR} = 0.83 < 1 \]

BEAM CHECKS

Shear Rupture of Beam Web:
\[ t_{min} = \frac{3.09 D}{F_u} \]  
\[ t_{min} = 0.01 \text{ in} \]
\[ \text{DCR} = 0.03 < 1 \]

Shear Yielding:
\[ A_s = 11.33 \text{ in}^2 \]
\[ \phi R_y = \phi 0.6 F_y A_s \]  
\[ \Phi = \]  
\[ R_y = 339.81 \text{ kips} \]
\[ \text{DCR} = 0.08 < 1 \]

Tensile Yielding:
\[ A_s = 24.7 \text{ in}^2 \]
\[ \phi R_y = \phi F_y A_s \]  
\[ \Phi = 0.9 \]
\[ R_y = 1111.5 \text{ kips} \]
DCR = 0.16 < 1

Tensile Rupture:
\[ A_y = 24.7 \text{ in}^2 \]

\[ U = 1 - (x_{\text{bar}}/l) \]

\[ x_{\text{bar}} = \frac{2h_y^2 + h_y^2 (d - 2t_y)}{8h_y l_y + 4t_y (d - 2t_y)} \]

\[ \Phi = 0.75 \]

\[ \Phi R_n = 1003.98 \text{ kips} \]

DCR = 0.18 < 1

Block Shear Rupture:
\[ A_y = 7.52 \text{ in}^2 \]
\[ A_{\text{en}} = 7.52 \text{ in}^2 \]
\[ A_{\text{nt}} = 9.17 \text{ in}^2 \]

\[ U = 1 \]

\[ \Phi = 0.75 \]

\[ \phi R_y = \phi F_u A_y \]

\[ \phi R_n = \phi F_u A_y \]

\[ \phi 0.6 F_u A_y + U_{\text{bi}} F_u A_{\text{nt}} \leq \phi 0.6 F_u A_y + U_{\text{bi}} F_u A_{\text{nt}} \]

\[ \phi 0.6 F_u A_y + U_{\text{bi}} F_u A_{\text{nt}} = 764.93 \text{ kips} \]

\[ \phi 0.6 F_u A_y + U_{\text{bi}} F_u A_{\text{nt}} = 815.69 \text{ kips} \]

DCR = 0.16 < 1

PLATE CHECKS

Shear Rupture of Plate:
\[ t_{\text{min}} = \frac{3.09 D}{F_u} \]

\[ t_{\text{min}} = 0.02 \text{ in} \]

DCR = 0.04 < 1

Flexure Yield:
\[ \phi M_y = \phi F_y Z \]

\[ Z = \frac{r e^2}{t} \]

\[ Z = 35.65 \text{ in}^4 \]

\[ \Phi = 0.9 \]

\[ \phi M_y = 1155.01 \text{ k-in} \]

\[ \phi R_y = 160.59 \text{ kips} \]

DCR = 0.17 < 1

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

\[ \frac{1.9E}{F_y} = 1530.56 \]

\[ \frac{L d}{F} = 1109.33 \]

\[ \frac{0.08E}{F} < \frac{L d}{F} < \frac{1.9E}{F} \]

So use AISC 15th Ed. F11-2b
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\[ F_r \_{\phi} = \frac{F_r}{\phi} \]

\[ M_n = C_n[1.52-0.274\left(\frac{Ld^2}{E}\right)\frac{F_r}{E}]M_s \]

\[ C_n = \frac{12.5M_{raw}}{2.5M_{raw} + 3M_a + 4M_e + 3M_r} \]

\[ M_a = 1283.34 \text{ kip-in} \]

\[ C_a = 1.67 \]

\[ \phi_x M_s = 2199.66 \text{ kip-in} \]

\[ \phi_y R_{x} = 305.83 \text{ kips} \]

\[ \text{DCR} = 0.09 < 1 \]

**Shear Yielding:**

\[ A_y = \text{7.31 in}^2 \]

\[ \phi_y R_{y} = \phi_x 0.6 F_r A_y \]

\[ \phi_y R_{y} = 157.95 \text{ kips} \]

\[ \text{DCR} = 0.18 < 1 \]

**Tensile Yielding:**

\[ A_y = \text{7.31 in}^2 \]

\[ \phi_y R_{y} = \phi_x F_r A_y \]

\[ \phi_y R_{y} = 236.93 \text{ kips} \]

\[ \text{DCR} = 0.76 < 1 \]

**Interaction of Axial, Flexure and Shear Yielding in Plate:**

\[ N_x = 179.6 \text{ kips} \]

\[ V_x = 27.9 \text{ kips} \]

\[ \phi_y R_{y} = 236.93 \text{ kips} \]

\[ \phi_y R_{y} = 157.95 \text{ kips} \]

\[ \phi_y M_{r} = 1555.01 \text{ kip-in} \]

\[ \frac{N_x}{\phi_y R_{y}} = 0.76 > 0.2 \text{ So use AISC 15th Ed. H1-1a} \]

\[ \left[ \frac{N_x}{\phi_y R_{y}} + \frac{8}{9} \left( \frac{V_x}{\phi_x M_s} \right)^2 + \frac{V_x}{\phi_y R_{y}} \right]^2 = 0.86 < 1 \]

\[ \left( \frac{N_x}{2\phi_y R_{y}} + \frac{V_x}{\phi_x M_s} \right)^2 + \left( \frac{V_x}{\phi_y R_{y}} \right)^2 = \text{N/A} > 1 \]

**Flexure Rupture:**

\[ Z_{\text{cr}} = \frac{r^2 d^2 \phi}{4} \]

\[ Z_{\text{cr}} = 35.65 \text{ in}^4 \]

\[ \phi M_{r} = 1550.71 \text{ k-in} \]

\[ \phi_y R_{y} = 215.6 \]

\[ \text{DCR} = 0.13 < 1 \]

**Shear Rupture:**

\[ A_y = \text{7.31 in}^2 \]

\[ \phi_y R_{y} = \phi_x 0.6 F_r A_y \]

\[ \phi_y R_{y} = 190.86 \text{ kips} \]

\[ \text{DCR} = 0.15 < 1 \]
Tensile Rupture: 
\[ A_t = \frac{7.31}{\Phi} \text{ in}^2 \]
\[ \Phi R_n = \frac{\Phi F_u A_t}{\Phi} = \frac{318.09}{0.75} \text{ kips} \]
\[ \Phi R_n = 318.09 \text{ kips} \]
\[ DCR = 0.56 < 1 \]

Interaction of Axial, Flexure and Shear Rupture in Plate:
\[ N_u = 179.6 \text{ kips} \]
\[ V_u = 27.9 \text{ kips} \]
\[ \Phi R_w = 318.09 \text{ kips} \]
\[ \Phi R_{wp} = 190.86 \text{ kips} \]
\[ \Phi M_w = 1550.71 \text{ kip-in} \]
\[ \frac{N_u}{\Phi R_{wp}} = 0.56 > 0.2 \text{ So use AISC 15th Ed. H1-1a} \]

\[ \left[ \frac{N_u}{\Phi R_{wp}} + \frac{8}{9} \left( \frac{V_u}{\Phi M_w} \right)^2 + \left( \frac{V_u}{\Phi R_{wp}} \right)^2 \right] = 0.48 < 1 \]  \hspace{1cm} \text{(AISC 15th Ed. H1-1a)}

\[ \frac{N_u}{2\Phi R_{wp}} + \frac{V_u a}{\Phi M_w} \left( \frac{V_u}{\Phi R_{wp}} \right)^2 = \text{N/A} > 1 \]  \hspace{1cm} \text{(AISC 15th Ed. H1-1b)}