FIELD TESTING AND LOAD RATING REPORT:
The Milo Adventist Academy’s Covered Bridge

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**EXECUTIVE SUMMARY**

In January 2013, Bridge Diagnostics, Inc. (BDI) was contracted by Rodney P. Kinney Associates, Inc. to perform diagnostic load testing on the Milo Adventist Academy Bridge over the Umpqua River in Days Creek Oregon. This structure was load tested and analyzed to help determine the participation of the kingpost A-frame system and to determine preliminary load ratings for the structure. In addition, the effects of turnbuckle tension on the structure’s live-load carrying capacity were examined.

During the field testing phase, the superstructure was instrumented with a combination of strain transducers and tilmeter rotation sensors. Once the structure was instrumented, controlled load tests were performed with a 3-axle local fire truck along one centered lateral position. Data obtained from the load tests was evaluated for quality and subsequently used to verify and calibrate a finite-element model of the structure.

In general, the response data recorded during the load tests was found to be of good quality and indicated no major signs of distress. The test data exhibited response magnitudes and shapes typical of a two girder structure and of the multi-stringer approach structures. The following notable observations were made from review of the load test data:

- The kingpost and turnbuckle system was significantly activated by the test truck. Both axial compressive stress and flexural stress were observed near the top of the kingpost. The relative magnitude of the flexural stress was somewhat concerning because it may be necessary to treat the kingpost struts as beam-columns rather than just compression members. The initial load rating capacities were calculated based on compression member specifications, however flexural and axial stress ratios were generated for additional consideration.

- Near the damaged girder section (section with a severely bent bottom flange), a substantial amount of out-of-plane bending was observed in the top flange. This indicated a non-uniform stress profile across the compression flange which would cause one edge to reach yield stress well before the theoretical moment capacity was reached. While this was not considered in the initial load rating, information about the compression flange stress profile has been provided to help determine if additional reduction factors should be applied to the moment capacity.

- Significant dynamic effects (20%) were observed during the normal speed test (5-7 mph). However negligible dynamics were observed during the crawl speed tests (1-3 mph). This observation indicates that if the heavier trucks cross at crawl speed, a much larger weight can safely cross.

A quasi three-dimensional finite-element model of the structure was created using the collected structural information, and subsequently calibrated until an acceptable match between the measured and analytical responses was achieved. Because the structure was not highly redundant, a good correlation between the measured and computed response was obtained fairly quickly.

Load ratings for the standard HS-20 rating vehicle and the local fire and sawdust trucks were performed according to the AASHTO Manual for Bridge Evaluation, 2011 Edition. The controlling ratings are shown in Table 1, and the following conclusions were made from the rating process:
Load rating results were controlled by the ultimate flexural capacity of the girders near midspan. Only the local fire truck crossing at crawl speed had a satisfactory load rating, which indicated that with the structure in its current state only the fire truck can cross the structure safely if it does so at crawl speed (1-3 mph).

A primary conclusion from the girder flexure ratings was that the dead-load responses were much greater than the live-load responses. It was found that the bridge covering accounted for approximately 30% of the dead-load, which greatly reduced the girders’ remaining live-load capacity.

The kingpost system was determined to be an important structural element and effectively reduced the peak live-load girder moment by approximately 40%. While not the critical rating component, the kingposts were found to have slightly deficient compression capacity at the upper section with no cover-plates. A recommended retrofit would be to extend the existing cover-plates to the ends and thereby increase the compression capacity by over 40 percent.

Other than a 0.85 condition factor at the damaged girder section, no additional capacity reductions or live-load amplifiers were applied to account for the observed bending stresses on primary compression members. Further investigation and consideration may be needed to account for bending effects in the kingpost and the top girder flange.

The approach span’s stringers have unsatisfactory ratings in their current state due to the lack of mechanical attachment to the decking members or other lateral bracing of the top flanges. Bracing the stringer top flanges at quarter points would increase the moment capacity so that all examined vehicles would have sufficient load ratings.

In addition to the load ratings performed on the structure in its current condition, an iterative analysis and rating process was performed in order to determine the effects of tightening the turnbuckle and increasing the participation of the kingpost system. The following was determined from this process:

It was found that by tightening the turnbuckle to 10 kips (which is approximately equivalent to ¼” of turnbuckle adjustment) the sawdust truck could cross at crawl speed (using reduced impact). With the turnbuckle tightened to this minimum level, the girders had a satisfactory rating in flexure of 1.00.

Additionally, it was determined that if the turnbuckle was tightened to 30 kips of force (~3/4” of turnbuckle tightening) that the sawdust truck had a higher overall live-load carrying capacity, with the controlling rating factor at crawl speed of 1.11 for girder flexure. It should be noted that at this level of kingpost system participation, the turnbuckle rating was 1.37 for reduced impact. Since breaking of the turnbuckle is not a ductile failure and therefore not desired, BDI considered this to be the maximum recommended level of turnbuckle tightening.

Tensioning the turnbuckle to 10 kips would likely be very difficult without some external load assistance (i.e. come-along between the girder and kingpost so that the turnbuckle can be adjusted).

This report contains details regarding the instrumentation and load testing procedures, a qualitative review of the load test data, a brief explanation of the modeling steps, and a summary of the load rating methods and results.
Table 1 - Controlling load rating factors – Current Structural Condition.

<table>
<thead>
<tr>
<th>RATING VEHICLE</th>
<th>LOCATION/LIMITING CAPACITY</th>
<th>CONTROLLING RATING FACTOR (FULL IMPACT)</th>
<th>CONTROLLING RATING FACTOR (REDUCED IMPACT)</th>
</tr>
</thead>
<tbody>
<tr>
<td>HS-20 (Inventory)</td>
<td>Near Midspan / Girder in Flexure</td>
<td>0.46</td>
<td>0.58</td>
</tr>
<tr>
<td>HS-20 (Operating)</td>
<td>Near Midspan / Girder in Flexure</td>
<td>0.60</td>
<td>0.76</td>
</tr>
<tr>
<td>Local Fire Truck</td>
<td>Near Midspan / Girder in Flexure</td>
<td>0.93</td>
<td>1.18</td>
</tr>
<tr>
<td>Local Sawdust Truck</td>
<td>Near Midspan / Girder in Flexure</td>
<td>0.74</td>
<td>0.94</td>
</tr>
</tbody>
</table>

Table 2 - Controlling load rating factors – With Kingpost Retrofit & Turnbuckle tightened 0.25” (10 kips).

<table>
<thead>
<tr>
<th>RATING VEHICLE</th>
<th>LOCATION/LIMITING CAPACITY</th>
<th>CONTROLLING RATING FACTOR (FULL IMPACT)</th>
<th>CONTROLLING RATING FACTOR (REDUCED IMPACT)</th>
</tr>
</thead>
<tbody>
<tr>
<td>HS-20 (Inventory)</td>
<td>Near Midspan / Girder in Flexure</td>
<td>0.50</td>
<td>0.63</td>
</tr>
<tr>
<td>HS-20 (Operating)</td>
<td>Near Midspan / Girder in Flexure</td>
<td>0.64</td>
<td>0.81</td>
</tr>
<tr>
<td>Local Fire Truck</td>
<td>Near Midspan / Girder in Flexure</td>
<td>1.01</td>
<td>1.28</td>
</tr>
<tr>
<td>Local Sawdust Truck</td>
<td>Near Midspan / Girder in Flexure</td>
<td>0.79</td>
<td>1.00</td>
</tr>
</tbody>
</table>

Table 3 - Controlling load rating factors – With Kingpost Retrofit & Turnbuckle tightened 0.75” (30 kips).

<table>
<thead>
<tr>
<th>RATING VEHICLE</th>
<th>LOCATION/LIMITING CAPACITY</th>
<th>CONTROLLING RATING FACTOR (FULL IMPACT)</th>
<th>CONTROLLING RATING FACTOR (REDUCED IMPACT)</th>
</tr>
</thead>
<tbody>
<tr>
<td>HS-20 (Inventory)</td>
<td>Near Midspan / Girder in Flexure</td>
<td>0.56</td>
<td>0.71</td>
</tr>
<tr>
<td>HS-20 (Operating)</td>
<td>Near Midspan / Girder in Flexure</td>
<td>0.72</td>
<td>0.91</td>
</tr>
<tr>
<td>Local Fire Truck</td>
<td>Near Midspan / Girder in Flexure</td>
<td>1.13</td>
<td>1.43</td>
</tr>
<tr>
<td>Local Sawdust Truck</td>
<td>Near Midspan / Girder in Flexure</td>
<td>0.88</td>
<td>1.11</td>
</tr>
</tbody>
</table>
Submittal Notes:

This submittal includes the following files on CD:

1. **Milo_Academy_Testing_Documents.pdf**
   This file provides pertinent details about the instrumentation plan and testing scenarios/procedures.

2. **BDI_Milo_Academy_Submittal_V3.pdf**
   This is the BDI report in “pdf” format. It contains details regarding the testing procedures, provides a qualitative data evaluation, displays response histories for each sensor, and discusses any notable observations and/or conclusions arising from the testing process.

3. **F&M_Milo_2.xlsx**
   This is the spreadsheet used to calculate the forces and moments induced by the test truck. It contains the data processed in units of stress which are used to calculate the observed moment and force histories at the instrumented sections. It also contains envelope tables of the maximum and minimum force and moment values.

4. **Milo_Processed_Data.xlsx**
   This spreadsheet contains data from all sensors from the selected crawl load test, normal speed test, and envelope tables for each sensor. It also contains a legend that describes each sensor location.

5. **Covered_Span_Rating_Files**
   The output files contain detailed information regarding the applied load and resistance factors, capacities, unfactored structural responses, and controlling load rating results for each of the rated vehicles. The summary files contain the controlling load rating results along with the controlling factored responses.

6. **Approach_Spans_Rating_Files**
   The output files contain detailed information regarding the applied load and resistance factors, capacities, unfactored structural responses, and controlling load rating results for each of the rated vehicles. The summary files contain the controlling load rating results along with the controlling factored responses.
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1. **STRUCTURAL TESTING PROCEDURES**

The Milo Academy Covered Bridge was a 132’ through girder structure that carries one lane of private traffic over the Umpqua River in Days Creek, OR. The covered bridge’s superstructure was made up of two 6’-1” deep plate girders, bailey transom floor-beams, built-up stringer tee members, and a king post system comprised of bailey transom members and a 1.5”Ø rod turnbuckle. The deck was comprised of 4”x12” timber planks and two steel runner plates. In addition to the covered span, three 20’ approach spans were present east of the covered span. These approach spans consisted of 5 bailey type transom stringers and the same decking as the covered span. All of the important readily-available geometric details were verified during the field visit; however, BDI did not perform an in-depth visual inspection of the structure or a full non-destructive evaluation (NDE).

The structure, including the approach spans, was instrumented with 65 reusable, surface-mount strain transducers (Figure 1.1 through Figure 1.4) and 4 tiltmeter rotation sensors (Figure 1.5). The final instrumentation plans, including sensor locations and IDs, have been provided in the attached drawings labeled “Milo_Academy_Testing_Documents.pdf”. Ropes access (SPRAT) was used to install some of the sensors on the covered span.

Once the instrumentation was installed, a series of diagnostic load tests was completed with the test truck traveling at crawl speed (1-3 mph). During testing, data was recorded on all channels at sample rate of 40 Hz as the test vehicle (3-axle fire truck) crossed the structure in the westbound direction along a centered lateral position, referred to as Path Y1 (further described in the attached testing documents). The truck’s longitudinal position was wirelessly tracked so that the response data could later be viewed as both a function of time and vehicle position. In addition to the crawl speed tests, one load test with the test truck traveling at 5-7 mph was performed to evaluate the dynamics observed at the normal crossing speed.

Information specific to the load tests can be found in Table 1.1. The test vehicle’s gross weight, axle weights, and wheel rollout distance (required for tracking its position along the structure) are provided in Table 1.2. A vehicle “footprint” is also shown in Figure 1.6. The vehicle weights were obtained from certified scales at a local truck stop, and all vehicle dimensions were measured in the field at the time of testing.

BDI would like to thank Rodney P. Kinney Associates for their help in scheduling, planning, and organizing the testing project. BDI would also like to thank the Milo Adventist Academy for their support during this project.
### Table 1.1 Structure description & testing info.

<table>
<thead>
<tr>
<th>Item</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>STRUCTURE NAME</strong></td>
<td>Milo Adventist Academy Covered Bridge</td>
</tr>
<tr>
<td>BDI Project Number</td>
<td>121201-OR</td>
</tr>
<tr>
<td><strong>TESTING DATE</strong></td>
<td>January 17, 2013</td>
</tr>
<tr>
<td><strong>CLIENT’S STRUCTURE ID #</strong></td>
<td>Milo</td>
</tr>
<tr>
<td>LOCATION/ROUTE</td>
<td>Over the Umpqua River/ Access Drive / Milo Adventist Academy</td>
</tr>
<tr>
<td><strong>STRUCTURE TYPE</strong></td>
<td>Two Built-Up Girder Bridge with Covering &amp; Wood deck with multi-stringer approach spans</td>
</tr>
<tr>
<td><strong>TOTAL NUMBER OF SPANS</strong></td>
<td>4</td>
</tr>
<tr>
<td><strong>SPAN LENGTHS</strong></td>
<td>Covered Bridge span - 132’</td>
</tr>
<tr>
<td></td>
<td>Approach spans - 20’</td>
</tr>
<tr>
<td><strong>SKEW</strong></td>
<td>0°</td>
</tr>
<tr>
<td><strong>STRUCTURE/ROADWAY WIDTHS</strong></td>
<td>Structure: 18’-3” / Roadway: 13’-0”</td>
</tr>
<tr>
<td><strong>WEARING SURFACE</strong></td>
<td>30”x 3/8” plate steel runners over 4”x12” timbers</td>
</tr>
<tr>
<td><strong>OTHER STRUCTURE INFO</strong></td>
<td>N/A</td>
</tr>
<tr>
<td><strong>SPANS TESTED</strong></td>
<td>3</td>
</tr>
<tr>
<td><strong>TEST REFERENCE LOCATION (BOW)</strong></td>
<td>South-east corner of the first approach span structure along the inside edge of the curb</td>
</tr>
<tr>
<td><strong>TEST VEHICLE DIRECTION</strong></td>
<td>Westbound</td>
</tr>
<tr>
<td><strong>TEST BEGINNING POINT</strong></td>
<td>Front axle 10 ft east of test reference location (BOW)</td>
</tr>
<tr>
<td><strong>LOAD POSITIONS</strong></td>
<td>Path Y1 – Driver’s side wheel 3’-4” from BOW (Centered along bridge)</td>
</tr>
<tr>
<td><strong>NUMBER/TYPE OF SENSORS</strong></td>
<td>- 65 Strain Transducers</td>
</tr>
<tr>
<td></td>
<td>- 4 Tiltmeters</td>
</tr>
<tr>
<td><strong>SAMPLE RATE</strong></td>
<td>40 Hz</td>
</tr>
<tr>
<td><strong>NUMBER OF TEST VEHICLES</strong></td>
<td>1</td>
</tr>
<tr>
<td><strong>STRUCTURE ACCESS TYPE</strong></td>
<td>Ropes (SPRAP)</td>
</tr>
<tr>
<td><strong>STRUCTURE ACCESS PROVIDED BY</strong></td>
<td>Bridge Diagnostics Inc.</td>
</tr>
<tr>
<td><strong>TRAFFIC CONTROL PROVIDED BY</strong></td>
<td>Milo Volunteer Fire Department</td>
</tr>
<tr>
<td><strong>TOTAL FIELD TESTING TIME</strong></td>
<td>2 day</td>
</tr>
<tr>
<td><strong>ADDITIONAL NDT INFO</strong></td>
<td>N/A</td>
</tr>
</tbody>
</table>
## Field Test and Load Rating Report - Milo Adventist Academy Bridge: Days Creek, OR

<table>
<thead>
<tr>
<th><strong>ITEM</strong></th>
<th><strong>Description</strong></th>
</tr>
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<tbody>
<tr>
<td>Test File Information:</td>
<td></td>
</tr>
<tr>
<td><strong>FILE NAME</strong></td>
<td><strong>LATERAL POSITION</strong></td>
</tr>
<tr>
<td>milo_y1_1</td>
<td>Y1</td>
</tr>
<tr>
<td>milo_y1_2</td>
<td>Y1</td>
</tr>
<tr>
<td>milo_y1_3</td>
<td>Y1</td>
</tr>
<tr>
<td>milo_y1_HS</td>
<td>Y1</td>
</tr>
<tr>
<td>Other Test Comments:</td>
<td></td>
</tr>
</tbody>
</table>

![Surface mounted strain transducers (typical).](image)

Figure 1.1 - Surface mounted strain transducers (typical).
Figure 1.2 - Strain transducers attached to Turnbuckle Stud (typical).

Figure 1.3 - Sensors attached to A-frame and Turnbuckle Stud (typical).
Figure 1.4 – Strain Transducers attached to Approach Span Stringers (typical).

Figure 1.5 - Tiltmeters attached near centerline of support (typical).
Table 1.2 Test vehicle information.

<table>
<thead>
<tr>
<th>VEHICLE TYPE</th>
<th>TANDEM REAR AXLE DUMP TRUCK</th>
</tr>
</thead>
<tbody>
<tr>
<td>GROSS VEHICLE WEIGHT (GVW)</td>
<td>45,360 lbs</td>
</tr>
<tr>
<td>WEIGHT/WIDTH - AXLE 1: FRONT</td>
<td>10,220 lbs</td>
</tr>
<tr>
<td>WEIGHT/WIDTH - AXLE 3: REAR TANDEM PAIR</td>
<td>35,140 lbs (~17,570 lbs each)</td>
</tr>
<tr>
<td>SPACING: AXLE 1 - AXLE 2</td>
<td>11’-8”</td>
</tr>
<tr>
<td>SPACING: AXLE 2 – AXLE 3</td>
<td>4’-6”</td>
</tr>
<tr>
<td>WEIGHTS PROVIDED BY</td>
<td>Milo Volunteer Fire Dept.</td>
</tr>
<tr>
<td>WHEEL ROLLOUT DISTANCE</td>
<td>11.15’ per wheel revolution</td>
</tr>
</tbody>
</table>

Figure 1.6 Test truck footprint.
2. PRELIMINARY INVESTIGATION OF TEST RESULTS

All of the field data was examined graphically to provide a qualitative assessment of the structure's live-load response. Some indicators of data quality include reproducibility between tests along identical truck paths, elastic behavior (strains returning to zero after truck crossing), and any unusual-shaped responses that might indicate nonlinear behavior or possible gage malfunctions. This process can provide a significant amount of insight into how a structure responds to live-load, and is often extremely helpful in performing an efficient and accurate structural analysis.

- **RESPONSES AS A FUNCTION OF LOAD POSITION:** Manual markers or “Clicks”, representing a single rotation of the test truck’s front driver’s side wheel, were inserted into the data so that the corresponding strain and rotation data could be presented as a function of vehicle position. This step was crucial during the model calibration process since it allowed the analysis engineer to easily compare the measured and computed responses as the truck loads moved across the structure. Please note that the test reference location (denoted at “Beginning of World” or “BOW”) was located at the bridge’s east expansion joint of the approach spans and corresponds to the 0 ft load position in provided figures.

Once all data was processed as a function of load position, one file was selected as having the best apparent quality. Table 2.1 provides a list of the selected data. Please note that the selected data was used to determine the response envelopes for all gages.

- **REPRODUCIBILITY AND LINEARITY OF RESPONSES:** The structural responses from identical tests were very reproducible as shown in Figure 2.1 and Figure 2.2. In addition, all strains appeared to be linear with respect to magnitude and truck position, and all strains returned to nearly zero, indicating that the structure was acting in a linear-elastic manner. The majority of the response histories had a similar degree of reproducibility and linearity, indicating that the data was of good quality.

- **LATERAL LOAD DISTRIBUTION:** When evaluating a bridge for the purpose of developing a load rating, the bridge’s ability to laterally distribute load is an essential characteristic to quantify. Lateral distribution of the approach spans can easily be observed by plotting the responses from an entire lateral cross-section. Figure 2.3 displays the corresponding peak midspan strains of an approach span along its cross-section. The response values shown in these figures correspond to the longitudinal load positions producing the maximum midspan responses for each truck path. All five stringers were active in resisting the truck load with the majority of resistance coming from the three interior stringers.

- **END-RESTRAINT BEHAVIOR OBSERVED IN GIRDER NEAR SUPPORTS:** The structural responses measured in the girders near the support locations showed significant end-restraint, as shown in Figure 2.4. This end-restraint was observed in the curved response history and reversal of flexure (negative moment) after the truck had past the sensors. Response histories from a true simple support would appear as straight lines returning to zero when the truck reached the end of the bridge. The combination of negative moment and net compression in the girder section indicated the end-restraint was due to resistance of horizontal and rotational motion of the girder’s bottom flange. This condition was likely due to the long bearing length and embedment/connection of the girder ends to the abutment concrete and adjacent stringers, and was considered during the modeling of the structure.
- **Observed Dynamics During Normal Speed Test:** The structural responses measured in the girders during the normal speed test contained dynamic responses that were approximately 20% of the static responses, as shown in Figure 2.5. This observation was important for two reasons. First, it showed that when the test vehicle crossed the bridge at normal speed a typical impact factor was observed, which increased the load effect by around 20% during testing. Secondly, it illustrated that the dynamic response could be eliminated when the fire truck crossed at crawl speed. Therefore, if the heavier trucks cross at crawl speed (1-2 mph) it is possible to consider a reduced dynamic load allowance for rating.

- **Observed Out-Of-Plane Bending Effects of the Damaged Girder Section:** Near the damaged girder section (section with a significantly bent bottom flange), a substantial amount of lateral bending was observed in both flanges. This means axial stresses across the flanges were not uniform as shown in Figure 2.6. This was not a concern for the bottom flange, which was in tension, because yielding would result in a straightening effect. Local and asymmetrical yielding of the top flange could potentially lead to greater lateral displacement and greater flexural stresses. While the top flange is braced against buckling there is a potential for a reduced moment capacity. To quantify this effect, the following ratio based on the recorded load test data was calculated at the worst case location:

\[
\gamma_{\text{GIRDER}} = \frac{\sigma_{\text{Out.of.Plane.Bending.TopFlange}}}{\sigma_{\text{Compression.TopFlange}} + \sigma_{\text{Out.of.Plane.Bending.TopFlange}}}
\]

Where:
\[
\gamma_{\text{GIRDER}} = \text{Bending stress to Total Stress Ratio} = 0.45
\]
\[
\sigma_{\text{Out.of.Plane.Bending.TopFlange}} = \text{Peak out-of-plane bending stress in top flange} = 1.84 \text{ ksi}
\]
\[
\sigma_{\text{Compression.TopFlange}} = \text{Peak average compressive flexural stress in top flange} = 2.29 \text{ ksi}
\]

This ratio shows that the lateral bending stress increased the compressive stress by 45% on one edge while decreasing the stress on the opposite edge by the same amount. It should be noted that this bending stress is likely due to both the damaged flange and the floorbeam-girder interaction. With perfect lateral bracing this effect would not be an issue, however any ability for the flange to move laterally could cause the lateral bending effect to increase and thereby reduce the overall compressive capacity of the top flange and thereby reduce the girder moment capacity. Other than a 0.85 condition factor, no other capacity reduction or load amplification was considered in the preliminary load rating. The quality of the lateral bracing should be considered in determining if this capacity reduction is sufficient.

- **Kingpost Member & Turnbuckle Live-Load Stresses:** The responses measured on the kingpost members and turnbuckle rods indicated this system was in fact activated by the test truck. Figure 2.7 shows the average stress from the two gages installed on both turnbuckles as a function of truck position. Figure 2.8 shows the stress histories recorded on the top and bottom flange edges, indicating that live-load induced both compression and significant biaxial bending in the kingpost. To quantify the bending effects, a ratio of the biaxial flexural stresses and the total stress were computed as shown in the following equation:

\[
\gamma_{\text{KINGPOST}} = \frac{\sigma_{\text{Out.of.Plane.Bending}} + \sigma_{\text{In.Plane.Bending}}}{\sigma_{\text{Axial}} + \sigma_{\text{Out.of.Plane.Bending}} + \sigma_{\text{In.Plane.Bending}}}
\]
Where:
\[ \gamma_{\text{KINGPOST}} = \text{Bending stress based reduction factor} = 0.37 \]
\[ \sigma_{\text{Axial}} = \text{Peak average compressive stress in top section of kingpost} = 4.17 \text{ ksi} \]
\[ \sigma_{\text{Out.of.Plane.Bending}} = \text{Peak out-of-plane bending stress in top section of kingpost} = 1.93 \text{ ksi} \]
\[ \sigma_{\text{In.Plane.Bending}} = \text{Peak in-plane bending stress in top section of kingpost} = 0.52 \text{ ksi} \]

This ratio shows that the bending stresses made up 37% of the total stress induced at one outer edge of a kingpost cross-section (section without cover-plates). For the preliminary load ratings the kingpost was treated as a compression member primarily because the analysis did not compute significant flexure. The level of observed bending stress suggests that some type of capacity reduction or load amplification may be warranted due to the combined stress conditions.

- **Bending Observed in Kingpost Braces**: The responses recorded on the thin braces between the girder and kingposts were found to indicate that these members were acting as brace points but did not receive significant axial loads. Therefore the braces did not alter the axial load along the length of the kingposts. Figure 2.9 shows responses of a brace gage pair that primarily measured in-plane bending (depicted by the responses of the gages on either side of the strut being essentially equal and opposite of each other).

- **Moments and Forces Calculated from Load Test Data**: Since all of the measurements were made on steel members, all strain measurements were converted to stress assuming a Young’s modulus of 29,000 ksi. These stress values were then applied to the various section properties to compute axial force and flexural moment induced by the test truck as shown Figure 2.10. These member actions were calculated as reference to show the level of induced load in each of the selected instrumented sections. A Microsoft Excel spreadsheet, containing the moments and forces calculated using the selected data, has been attached to this report as reference. Because the measured member forces were limited to the instrumentation points, bridge load ratings were based on a field-verified model where responses could be obtained throughout the structure.

As previously stated, all test data was initially processed and assessed for quality. Then, one set of test data for each truck path was selected for having the best apparent quality. This selected data was then used to briefly calibrate the finite-element (FE) model of the structure, which was in turn used to produce the load ratings. Table 2.1 provides a list of the data file that was used in the FE analysis.

<table>
<thead>
<tr>
<th>TRUCK PATH</th>
<th>SELECTED DATA FILE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Y1</td>
<td>milo_y1_2.dat</td>
</tr>
</tbody>
</table>
Figure 2.1 - Example of Stress Response Reproducibility.

Figure 2.2 - Example of Rotation Response Reproducibility.
Figure 2.3 - Lateral Load Distribution observed in Maximum Strain Responses.

Figure 2.4 – Strain Response History – Girder near support - Highlighting observed end-restraint behavior.
Observed dynamics during normal speed crossing (blue response) were found to be approximately 20%.

Figure 2.5 – Strain Response Comparison – Girder bottom flange near midspan - Highlighting observed dynamic behavior.

The difference between stresses on top flange was equal to twice the out-of-plane bending stress.

Figure 2.6 – Stress Response History – Top flange stress highlighting out-of-plane bending induced by live-load.
Figure 2.7 – Average Turnbuckle Stress Responses– North & South Turnbuckle.

Figure 2.8 – Stress Response Histories – stresses measured on kingpost flange edges highlighting axial and bending effects.
Figure 2.9 – Stress Response History – Braces between girder and kingpost highlighting lack of compression in struts.

Equal and opposite strain responses on the bracing member indicated flexural responses with minimal axial load. These members did not significantly alter axial loads along the kingpost.
Typical Kingpost/Girder Instrumented Cross-sections

Calculation of Axial Force

\[ F_{AXIAL} = E \cdot A \cdot \left[ \frac{\varepsilon_1 + \varepsilon_3 + \varepsilon_2 + \varepsilon_4}{4} \right] \quad \text{OR} \quad F_{AXIAL} = E \cdot A \cdot \left[ \frac{\varepsilon_A + \varepsilon_B}{2} \right] \]

Calculation of Strong-axis (About X-Axis) bending

\[ M_X = \frac{E[(\varepsilon_3 + \varepsilon_4)/2 - (\varepsilon_1 + \varepsilon_2)/2]S_X}{2} \quad \text{OR} \quad M_X = \frac{E[(\varepsilon_A) - (\varepsilon_B)]S_X}{2} \]

Calculation of Weak-axis (About Y-Axis) bending

\[ M_Y = \frac{E[(\varepsilon_1 + \varepsilon_3)/2 - (\varepsilon_2 + \varepsilon_4)/2]S_Y}{2} \]

Figure 2.10 – Basic mechanic equations used to calculate cross-sectional force and moment.
### Table 2.2 – North Girder Section – Axial Force and Bending Moment Envelope.

<table>
<thead>
<tr>
<th>MEMBER DESIGNATION</th>
<th>MEMBER DESCRIPTION</th>
<th>VALUE DESCRIPTION</th>
<th>AXIAL FORCE (KIPS)</th>
<th>IN-PLANE BENDING (KIP-IN)</th>
<th>OUT-OF-PLANE BENDING (KIP-IN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>North Girder</td>
<td>Section J-J</td>
<td>Minimum</td>
<td>-17.6</td>
<td>-701.8</td>
<td>N/A</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Maximum</td>
<td>1.6</td>
<td>491.6</td>
<td>N/A</td>
</tr>
<tr>
<td>North Girder</td>
<td>Section L-L</td>
<td>Minimum</td>
<td>-16.8</td>
<td>-55.4</td>
<td>-26.2</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Maximum</td>
<td>7.1</td>
<td>2643.6</td>
<td>2.8</td>
</tr>
<tr>
<td>North Girder</td>
<td>Section M-M</td>
<td>Minimum</td>
<td>-20.2</td>
<td>-8.6</td>
<td>N/A</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Maximum</td>
<td>3.0</td>
<td>3647.1</td>
<td>N/A</td>
</tr>
<tr>
<td>North Girder</td>
<td>Section N-N</td>
<td>Minimum</td>
<td>-18.3</td>
<td>-192.8</td>
<td>N/A</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Maximum</td>
<td>0.5</td>
<td>795.5</td>
<td>N/A</td>
</tr>
<tr>
<td>North Kingpost</td>
<td>Section P-P</td>
<td>Minimum</td>
<td>-27.5</td>
<td>-11.7</td>
<td>N/A</td>
</tr>
<tr>
<td></td>
<td>(Cover Plate)</td>
<td>Maximum</td>
<td>0.0</td>
<td>0.8</td>
<td>N/A</td>
</tr>
<tr>
<td>North Kingpost</td>
<td>Section Q-Q</td>
<td>Minimum</td>
<td>-33.3</td>
<td>-13.0</td>
<td>-0.1</td>
</tr>
<tr>
<td></td>
<td>(No Cover Plate)</td>
<td>Maximum</td>
<td>0.0</td>
<td>0.4</td>
<td>4.2</td>
</tr>
<tr>
<td>North Kingpost</td>
<td>Turnbuckle</td>
<td>Minimum</td>
<td>-0.1</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Maximum</td>
<td>5.8</td>
<td>N/A</td>
<td>N/A</td>
</tr>
</tbody>
</table>
Table 2.3 – South Girder Section – Axial Force and Bending Moment Envelope.

<table>
<thead>
<tr>
<th>MEMBER DESIGNATION</th>
<th>MEMBER DESCRIPTION</th>
<th>VALUE DESCRIPTION</th>
<th>AXIAL FORCE (KIPS)</th>
<th>IN-PLANE BENDING (KIP-IN)</th>
<th>OUT-OF-PLANE BENDING (KIP-IN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>South Girder</td>
<td>Section S-S</td>
<td>Minimum</td>
<td>-12.9</td>
<td>-85.2</td>
<td>N/A</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Maximum</td>
<td>0.8</td>
<td>982.6</td>
<td>N/A</td>
</tr>
<tr>
<td>South Girder</td>
<td>Section T-T</td>
<td>Minimum</td>
<td>-2.6</td>
<td>-6.4</td>
<td>N/A</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Maximum</td>
<td>5.5</td>
<td>3099.0</td>
<td>N/A</td>
</tr>
<tr>
<td>South Girder</td>
<td>Section U-U</td>
<td>Minimum</td>
<td>-6.6</td>
<td>-608.1</td>
<td>N/A</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Maximum</td>
<td>3.6</td>
<td>324.8</td>
<td>N/A</td>
</tr>
<tr>
<td>South A-Frame</td>
<td>Section W-W</td>
<td>Minimum</td>
<td>-25.1</td>
<td>-1.2</td>
<td>-27.3</td>
</tr>
<tr>
<td></td>
<td>(Extra Bottom Cover Plate)</td>
<td>Maximum</td>
<td>0.1</td>
<td>11.5</td>
<td>1.6</td>
</tr>
<tr>
<td>South Kingpost</td>
<td>Section X-X</td>
<td>Minimum</td>
<td>-26.4</td>
<td>-11.2</td>
<td>-0.10</td>
</tr>
<tr>
<td></td>
<td>(No Cover Plate)</td>
<td>Maximum</td>
<td>0.61</td>
<td>1.37</td>
<td>5.89</td>
</tr>
<tr>
<td>South Kingpost</td>
<td>Section Y-Y</td>
<td>Minimum</td>
<td>-33.2</td>
<td>-34.8</td>
<td>N/A</td>
</tr>
<tr>
<td></td>
<td>(Cover Plate)</td>
<td>Maximum</td>
<td>0.3</td>
<td>0.3</td>
<td>N/A</td>
</tr>
<tr>
<td>South Kingpost</td>
<td>Turnbuckle</td>
<td>Minimum</td>
<td>-0.21</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Maximum</td>
<td>5.90</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>South Kingpost</td>
<td>Brace 1</td>
<td>Minimum</td>
<td>-0.14</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Maximum</td>
<td>0.16</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>South Kingpost</td>
<td>Brace 2</td>
<td>Minimum</td>
<td>-0.20</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Maximum</td>
<td>0.07</td>
<td>N/A</td>
<td>N/A</td>
</tr>
</tbody>
</table>
3. MODELING, ANALYSIS, AND DATA CORRELATION

This section briefly describes the methods and findings of the Milo Academy Bridge modeling procedures. A list of modeling and analysis parameters specific to this bridge is provided in Table 3.1.

**MODELING PROCEDURES**

First, geometric data collected in the field and insight gained from the qualitative data investigation were used to create an initial, finite-element model using BDI’s WinGEN modeling software which is illustrated in Figure 3.1 and Figure 3.2. Note that the covered span was modeled separately as a 2-D model with a 3-D kingpost system, as shown in Figure 3.1; while the approach spans were modeled strictly using a 2-D approach, as shown in Figure 3.2. The kingpost was modeled in a 3-D manner in order to achieve a more accurate interaction with the girders.

![Figure 3.1 – Rendering of finite-element model of covered girder superstructure highlighting three dimensional geometry of kingpost system.](image)

![Figure 3.2 - 2-D finite-element model of approach span superstructure with modeled test truck load.](image)
Once the initial models were created, the load test procedures were reproduced using BDI’s WinSAC structural analysis and data correlation software. This was done by moving a two-dimensional “footprint” of the test truck across the model in consecutive load cases that simulated the designated truck path used in the field, as shown in Figure 3.2. The analytical responses of this simulation were then compared to the field responses to validate the model’s basic structure and to identify any gross modeling deficiencies.

The models were then briefly calibrated until an acceptable match between the measured and analytical responses was achieved. This calibration involved an iterative process of optimizing material properties and boundary conditions until they were realistically quantified. In the case of these structures, the majority of the calibration effort was spent modeling the observed bearing conditions. Please note that the goal of this calibration process was to ensure that model was accurate and somewhat conservative in simulating the responses under the fire truck; given that some of the factors lowering the induced load, like the observed end-restraint, could not be fully reliable upon at the ultimate strength state.

### Table 3.1 Analysis and model details.

<table>
<thead>
<tr>
<th><strong>ANALYSIS TYPE</strong></th>
<th>- Linear-elastic finite element - stiffness method.</th>
</tr>
</thead>
</table>
| **MODEL GEOMETRY** | - Approach Spans: 2D composed of shell elements, frame elements, and springs.  
                        - Covered Bridge: 3D composed of shell frame elements |
| **NODAL LOCATIONS** | - Nodes placed at the ends of all frame elements.  
                          - Nodes placed at all four corners of each shell element. |
| **MODEL COMPONENTS** | - Shell elements representing the wearing surface and the connecting elements between the girder and the kingpost.  
                           - Frame elements representing the girders, stringers, floor-beams, kingposts, and cross-frames.  
                           - Springs representing the bearing conditions at supports. |
| **LIVE-LOAD** | - 2-D footprint of test truck consisting of 10 vertical point loads. Truck paths simulated by series of load cases with truck footprint moving at 1.5 ft increments along a straight path. |
| **DEAD-LOAD** | - Self-weight of structure, including the wooden components of the covered span. |
| **TOTAL NUMBER OF RESPONSE COMPARISONS** | - 61 strain gage locations x 186 load positions = 11,346 strain comparisons  
                                            - 4 rotation gage locations x 186 load positions = 744 displacement comparisons |
| **COVERED SPAN MODEL STATISTICS** | - 899 Nodes  
                                      - 1406 Elements  
                                      - 16 Cross-section/Material types  
                                      - 108 Load Cases  
                                      - 53 Gage locations |
| **APPROACH SPAN MODEL STATISTICS** | - 1482 Nodes  
                                           - 1955 Elements  
                                           - 9 Cross-section/Material types  
                                           - 78 Load Cases  
                                           - 12 Gage locations |
**MODEL CALIBRATION RESULTS**

Following the optimization procedures, the final model produced 0.968 and 0.982 correlations with the main span and approach spans, respectively. The main-span model did an excellent job of simulating the primary flexure and axial responses of all members but could not reproduce the local bending behavior observed in the kingposts or the lateral bending of the girder flanges at the damaged section. The accuracy of the approach span model can be considered excellent; the primary result being a realistic stringer load distribution. Numeric accuracy or error terms are provided in Table 3.2. Visual comparisons of the response histories are shown in Figure 3.3 through Figure 3.14.

**Table 3.2 Model accuracy & parameter values.**

<table>
<thead>
<tr>
<th>ERROR PARAMETERS</th>
<th>APPROACH SPANS MODEL VALUE</th>
<th>COVERED SPAN MODEL VALUE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Absolute Error</td>
<td>2,758.7</td>
<td>22,891.4</td>
</tr>
<tr>
<td>Percent Error</td>
<td>3.6%</td>
<td>11.8%</td>
</tr>
<tr>
<td>Scale Error</td>
<td>3.3%</td>
<td>17.8%</td>
</tr>
<tr>
<td>Correlation Coefficient</td>
<td>0.982</td>
<td>0.968</td>
</tr>
</tbody>
</table>

The following table contains the equations used to compute each of the statistical error values:

**Table 3.3 - Error Functions**

<table>
<thead>
<tr>
<th>ERROR FUNCTION</th>
<th>EQUATION</th>
<th>BRIEF DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>Absolute Error</td>
<td>( \sum</td>
<td>\varepsilon_m - \varepsilon_c</td>
</tr>
<tr>
<td>Percent Error</td>
<td>( \frac{\sum (\varepsilon_m - \varepsilon_c)^2}{\sum \varepsilon_m^2} )</td>
<td>Sum of the response differences squared divided by the sum of the measured responses squared. Helps provide a better qualitative measure of accuracy.</td>
</tr>
<tr>
<td>Scale Error</td>
<td>( \frac{\sum \text{max}</td>
<td>\varepsilon_m - \varepsilon_c</td>
</tr>
<tr>
<td>Correlation Coefficient</td>
<td>( \frac{\sum (\varepsilon_m - \bar{\varepsilon_m})(\varepsilon_c - \bar{\varepsilon_c})}{\sqrt{\sum (\varepsilon_m - \bar{\varepsilon_m})^2(\varepsilon_c - \bar{\varepsilon_c})^2}} )</td>
<td>Measure of the linearity between the measured and computed data. Helps determine the error with respect to response shape and phase.</td>
</tr>
</tbody>
</table>
Figure 3.3 - Approach Span Model – Example stress comparison plot – Section C-C.

Figure 3.4 - Approach Span Model – Example stress comparison plot – Section D-D.
Figure 3.5 - Main Span Model – Example girder stress comparison plot – Section S-S near the supports.

Figure 3.6 - Main Span Model – Example girder rotation comparison plot – Section O-O near the supports.
Analysis results do not contain lateral bending component in girder flanges.

Figure 3.7 - Main Span Model – Example girder stress comparison plot – Section L-L near damaged girder section.

Figure 3.8 - Main Span Model – Example girder stress comparison plot – Section M-M near midspan.
Figure 3.9 - Main Span Model – Example kingpost stress comparison plot – Section Q-Q.

Model responses show significantly less stress variation in cross-section.

Figure 3.10 - Main Span Model – Example kingpost stress comparison plot – Section Y-Y.

Model responses show significantly less stress variation in cross-section.
Figure 3.11 - Main Span Model – Example floor-beam stress comparison plot.

Figure 3.12 - Main Span Model – Example stringer-tee stress comparison plot.
Figure 3.13 - Main Span Model – Example strain comparison plot – Section J-J.

Figure 3.14 - Main Span Model – Example rotation comparison plot – Section V-V.
4. LOAD RATING PROCEDURES AND RESULTS

RATING PROCEDURES

Preliminary load ratings were performed on all primary bridge elements in accordance with the AASHTO LRFR guidelines. Structural responses were obtained from a slightly modified version of the final calibrated model, and member capacities were based on the notes from the on-site bridge investigation. The rating methods used in BDI’s approach closely match typical rating procedures, with the exception that a field-verified finite-element model analysis was used rather than a typical AASHTO girder-line analysis. This section briefly discusses the methods and findings of the load rating procedures.

Once the analytical model was calibrated to produce an acceptable match to the measured responses, the model was adjusted to ensure the reliability of all optimized model parameters. This adjustment involved the identification of any calibrated parameters that could change over time or could become unreliable under heavy loads. In the analysis of the Milo Academy Bridge, the only calibrated parameter that was determined to be unreliable was the end-restraint at the supports. The end-restraint behavior was therefore reduced by 50% to ensure that the ratings would remain accurate and slightly conservative over time.

Member capacities (provided in Appendix A) were calculated based on OBEC’s & RPKA’s structural investigations of the superstructure, the AASHTO LRFD specifications, and the following assumptions:

- **Resistance Factors:** Current AASHTO LRFR resistance factors were used as follows:
  - The total resistance factor was set to the following $\Phi_{\text{total}} = \Phi_r \times \Phi_s \times \Phi_c$
  - The resistance factors, $\Phi_r$, were the following: 1.0 (flexure), 1.0 (shear), 0.9 (compression), 0.9 (tension)
  - The system factors, $\Phi_s$, were the following: 0.85 for members in two girder bridge, 1.0 for the redundant stringer system of the approach spans
  - $\Phi_c = 0.95$ (fair for majority of members), 0.85 for the damaged girder section (poor condition)

- **Load Factors:** In order to better match the AASHTO LRFR resistance factors, the following current AASHTO LRFR load and impact factors were used (not the ODOT equivalent):
  - $\gamma_{DL} = 1.25$ for all dead load, which assumes all nonstructural load was well measured
  - $\gamma_{LL} = 1.30$ for the local routine vehicles (fire and sawdust trucks), and 1.75 & 1.35 for the inventory and operating HS20 ratings
  - IM = 33% for normal speed crossing and 5% for crawl speed crossing based on the crawl speed load test data

- **General Assumptions:**
  - All ratings were based on the current condition of each member type and the current AASHTO LRFR code.
  - All welds, splices and connections of any kind were assumed to be as strong as the primary members.
  - Both the covered span and the approach spans were evaluated separately.
  - Only strength limit states were checked. Due to low number of load cycles, fatigue was not considered. Additionally, serviceability was not checked based on
the assumption that the bridge would be routinely checked for any serviceability concerns like permanent deformation of the girders.

- The kingpost and turnbuckle were essentially assumed to only resist live-load. This was achieved by greatly reducing the modeled stiffness of these elements with the dead load application.
- Based on the age of the structure, the steel yield strength, $F_y$, was assumed to be 33 ksi for the built-up sections (girder, stringer tees), and 50 ksi for the Bailey Transom Trestle members (floor-beams, approach span stringers, kingposts).

**Girder Ratings:**

- **Moment Capacity**: the girders were found to be non-compact slender members that can reach approximately 93% of yield moment due to an unbraced length of the top flange of 6’. Note this un-braced length assumes that the diagonal brace from the floorbeam ends provide sufficient bracing of the top flange.
- **Shear Capacity**: Due to the large web depth, the webs had to be considered unstiffened during the capacity calculation according to the LRFD code. However, since the deck brace (large steel angle that connects the timber decking to the girders also braces the web from buckling; a reduced web depth was used when computing the web shear buckling coefficient. This resulted in a shear capacity equal to approximately 33% of the plastic shear force.
- The effects of the cuts made to allow the floor beams to cross through bottom of girder web were not considered.
- Beyond decreasing the condition factor to 0.85 for the damaged girder section, the effects of the out-of-plane bending of the top flange **were not** directly considered. This may need to be looked at further due to the substantial level of out-of-plane bending observed, as discussed in Section 2. However for consistency in these preliminary ratings, BDI rated this damaged section according to the current AASHTO LRFR code which does not directly discuss this type of damage effect.
- $\Phi_{\text{total}} = 0.808$ for majority of the girder section, 0.722 for bent girder section.

**Turnbuckle Capacity:**

- Based on the turnbuckle strength table provided by RPKA, The ultimate turnbuckle breaking strength of 85 kips was used for rating.
- $\Phi_{\text{total}} = 0.808$.

**Kingpost Capacity:**

- The kingpost braces (connecting the kingpost to the girder between the turnbuckle and the end of the kingpost) experienced minor flexural stress and negligible axial load. Therefore the struts were considered to be braces only, and not directly considered for rating. Based on this finding axial forces were assumed to be constant along each kingpost member. This assumption led to the conclusion that the kingpost with minimum cross-sectional area was the critical section, which was located at the top Bailey transom section without cover plates.
- Capacities for the following kingpost sections were computed for rating and provided in Appendix A: top Bailey transom section without cover plates; top Bailey transom section with cover plates; and the bottom Bailey transom section with cover plates and the large 12” bottom cover plate.
- Since the model could not simulate the observed bending of the top sections of the kingpost, it may be prudent to reduce the axial capacities or increasing load
effects by some kind of stress ratio as discussed in Section 2. However for consistency in these preliminary ratings, BDI rated this section for compression only using the AASHTO LRFD capacity that accounts for flexural buckling of a concentrically loaded member.

- $\Phi_{total} = 0.808$

- **Floorbeam Capacity:**
  - **Moment strength:** This capacity was based on an un-braced strength between the stringer tees (~4.5’), which resulted in a flexural resistance that is approximately 105% of the yielding moment.
  - **Shear strength:** This resistance was based on the section with a 4” hole in the web, which reduced the capacity by ~40%.

- **Stringer Capacities – Main Span:**
  - **Moment strength:** Due to a relatively coarse mesh refinement of the model in each stringer bay, it was decided that these ratings were best based on simple span assumptions and an un-braced length between the floorbeams ($w^*L^2/8$ for dead load and $P^*L/4$ for live load). In addition to this assumption, consideration for a 1 foot variation in vehicle lateral position was implemented, which was determined to possibly increase the load on a given stringer by about 15.5%. It should be noted that before the addition of the lateral variation of load, that the stringer moment ratings from the model were comparable to the slightly more conservative ratings provided in this report.
  - **Shear strength:** Based on the double tee webs.

- **Stringer Capacities - Approach Span:**
  - **Moment strength:** Due to lack of evidence of mechanical connection between timber deck and the stringers, this capacity was based on an un-braced length of ~20’ (entire span length). This un-braced length reduces the capacity to 33% of the yield moment, which could be addressed by the addition of proper bracing of the stringers’ top flange.
  - **Shear strength:** Was based on the section with the 4” hole. Note that shear capacity considering the hole is significantly reduced (38.5 kip vs 67.5 kip). Given the holes between the transverse stiffeners, the webs were assumed to be “unstiffened” since the stiffened capacity relies on post buckling behavior (tension field action) which these holes would prevent.

- $\Phi_{total} = 0.95$
A summary of the calculated member capacities has been provided in Table 4.1.

Table 4.1 – Nominal member capacities.

<table>
<thead>
<tr>
<th>MEMBER</th>
<th>TENSILE CAPACITY, KIP-INCH</th>
<th>COMPRESSION CAPACITY, KIPS</th>
<th>SHEAR CAPACITY, KIPS</th>
<th>MOMENT CAPACITY, KIP-INCH</th>
</tr>
</thead>
<tbody>
<tr>
<td>Girder</td>
<td>---</td>
<td>---</td>
<td>167.6</td>
<td>39630</td>
</tr>
<tr>
<td>Stringer Tee</td>
<td>---</td>
<td>---</td>
<td>86.1</td>
<td>698</td>
</tr>
<tr>
<td>Floorbeam</td>
<td>---</td>
<td>---</td>
<td>37.0</td>
<td>1853</td>
</tr>
<tr>
<td>Turnbuckle</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td>Kingpost – No Cover Plate</td>
<td>---</td>
<td>192.4</td>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td>Kingpost – Cover Plate</td>
<td>---</td>
<td>278.8</td>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td>Kingpost – Extra Cover Plate</td>
<td>---</td>
<td>293.6</td>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td>Stringer – Approach Slab</td>
<td>---</td>
<td>38.5</td>
<td></td>
<td>724</td>
</tr>
</tbody>
</table>

Load ratings were performed using the final adjusted rating models according to the AASHTO Manual for Bridge Evaluation, 2011 Edition. Given the 13’ wide roadway, only a one lane loaded condition was considered for the rating. Note that a ±6”-8” variation in lateral truck position from a perfectly centered position was considered. Figure 4.1 through Figure 4.3 show the rated load configurations, which were the standard HS-20 load rating vehicle, and the local fire and sawdust trucks. The majority of the structural dead loads were automatically applied by the modeling program’s self-weight function, with the exception of the extraneous loads of the sidings, roof, utilities, knee brace, etc. Based on the preliminary calculations performed by OBEC Consulting engineers, a 210 lb/ft load was applied to the length of each girder to account for these non-structural loads.

In addition to the load ratings performed on the structural in its current condition, an iterative analysis and rating process was performed in order to determine the effects of tightening the turnbuckle and increasing the participation of the kingpost system. During this analysis, it was assumed that the kingpost was retrofit so that the cover plates continued to the top of the A frame. The analysis was performed by applying nodal loads at the ends of the turnbuckle elements (that represented the tightening of the turnbuckle) while neglecting the stiffness of these members. This process was performed until a satisfactory rating in girder flexure was achieved, and then continued until the kingpost system (kingpost members and turnbuckle) reached a lower but acceptable satisfactory rating.
Figure 4.1 - AASHTO HS-20 load rating vehicle configuration.

Figure 4.2 – Local Fire Truck - Load rating vehicle configuration (kips & feet).

Figure 4.3 – Local Sawdust Truck - Load rating vehicle configuration (kips & feet).
**Rating Results & Conclusions**

For clarification, a rating factor is the ratio of the available live-load capacity to the applied factored live-load. Therefore any rating factor equal to or greater than one (1.0) indicates that the structure can support the specific vehicle loads with a sufficient factor of safety. Consequently, any rating factor less than one (1.0) indicates that the structure can only support a portion of the given vehicle’s weight with a sufficient factor of safety. The load rating factor for a particular truck can essentially be considered a multiplier on a specific truck load to obtain the maximum load limit for that vehicle.

The following is a breakdown of the load rating factors for the structure in its current condition for the three rated vehicles:

- Kingpost and turnbuckle rating factors were slightly deficient when full impact was considered but satisfactory with a reduced impact factor. It should be noted that further consideration of the biaxial bending observed may be needed.

- Shear capacities of the girders and floorbeams were deficient with full impact applied but satisfactory with the impact reduced to 5%. **Vehicles would need to cross the bridge at 1-2 mph.**

- The controlling rating factors for this structure are for the girder in flexure. The girders only have a satisfactory flexural rating for the fire truck when the reduced impact factor is applied. This indicates that the fire truck can cross the structure in its current state with the required factor of safety if it does so at crawl speed.

- An important conclusion regarding the controlling load ratings were that the covering caused a 30% increase dead load moment, which directly reduced the capacity available for live-load.

- The approach span’s stringers have insufficient ratings in their current state due to the lack of top flange bracing. It was found that the stringer ratings would be sufficient for all vehicles examined by simply bracing the top flange at quarter points.

- Although the 4” holes in the stringer near the ends of the approach spans significantly reduce their shear capacity, all of the shear ratings of the approach span were satisfactory. This result was unexpected and was a result of the level of lateral load distribution observed in these approach spans.

- Additional secondary Sawdust truck ratings were performed as reference considering that the covering and the kingpost system were both removed, see Table 4.26. From comparison of the Sawdust truck rating at current condition, the following conclusions were made:
  - The maximum live-load girder moment was reduced by 40% due to the kingpost interaction.
  - The kingpost system accounted for 10% of the girder’s total dead load moment.
  - The covering (sidings & roof) represent 30% of the total girder dead load moment.
The following is a breakdown of the load rating factors for the structure when the turnbuckle is tightened:

- By tightening the turnbuckle to 10 kips (which equates to shortening the turnbuckle by approximately \(\frac{1}{4}\)”) the sawdust truck was found to be able to safely cross at crawl speed (using reduced impact). With the turnbuckle tightened to 10 kips, the girders reached a satisfactory rating in flexure of 1.00.

- If the turnbuckle was tightened to 30 kips of tension (shortening the turnbuckle by \(~3/4\)”), the sawdust truck had a higher overall live-load carrying capacity, with the controlling rating factor at crawl speed of 1.11 for girder flexure. It should be noted that at this level of kingpost system participation, that the turnbuckle rating was 1.37 for reduced impact. Since breaking of the turnbuckle is not a ductile failure and therefore not desired, BDI considered this to be the maximum recommended level of turnbuckle tightening.

- Note that the 0.25” and 0.75” of turnbuckle adjustment mentioned above (equivalent to 10 & 30 kips of tensioning) was determined by computing the relative difference in nodal displacements between the top of the kingpost and the girder.
Table 4.2 - Rating factors & responses for rated vehicles - Kingpost Compression – Current Condition.

<table>
<thead>
<tr>
<th>Truck</th>
<th>Dead Load Effects, kips</th>
<th>Live Load Effects, kips</th>
<th>Controlling Operating RF (Full Impact)</th>
<th>Controlling Operating RF (Reduced Impact)</th>
</tr>
</thead>
<tbody>
<tr>
<td>HS-20 (Inventory)</td>
<td>-4.19</td>
<td>-65.03</td>
<td>0.89</td>
<td>1.13</td>
</tr>
<tr>
<td>HS-20 (Operating)</td>
<td>-4.19</td>
<td>-65.03</td>
<td>1.15</td>
<td>1.46</td>
</tr>
<tr>
<td>Local Fire Truck</td>
<td>-4.19</td>
<td>-41.01</td>
<td>1.90</td>
<td>2.41</td>
</tr>
<tr>
<td>Local Sawdust Truck</td>
<td>-4.19</td>
<td>-68.20</td>
<td>1.14</td>
<td>1.44</td>
</tr>
</tbody>
</table>

* Both the Live Load and Dead Load Responses are unfactored responses.

Table 4.3 - Rating factors & responses for rated vehicles - Kingpost Compression with cover plate RETROFIT – Tightened Turnbuckle to Minimum Level – 0.25” (10 kips).

<table>
<thead>
<tr>
<th>Truck</th>
<th>Dead Load Effects, kips</th>
<th>Live Load Effects, kips</th>
<th>Controlling Operating RF (Full Impact)</th>
<th>Controlling Operating RF (Reduced Impact)</th>
</tr>
</thead>
<tbody>
<tr>
<td>HS-20 (Inventory)</td>
<td>-11.7</td>
<td>-65.05</td>
<td>1.24</td>
<td>1.57</td>
</tr>
<tr>
<td>HS-20 (Operating)</td>
<td>-12.41</td>
<td>-65.00</td>
<td>1.60</td>
<td>2.03</td>
</tr>
<tr>
<td>Local Fire Truck</td>
<td>-12.41</td>
<td>-40.99</td>
<td>2.64</td>
<td>3.34</td>
</tr>
<tr>
<td>Local Sawdust Truck</td>
<td>-11.7</td>
<td>-68.23</td>
<td>1.59</td>
<td>2.01</td>
</tr>
</tbody>
</table>

* Both the Live Load and Dead Load Responses are unfactored responses.

Table 4.4 - Rating factors & responses for rated vehicles - Kingpost Compression with cover plate RETROFIT – Tightened Turnbuckle to Maximum Level – 0.75” (30 kips).

<table>
<thead>
<tr>
<th>Truck</th>
<th>Dead Load Effects, kips</th>
<th>Live Load Effects, kips</th>
<th>Controlling Operating RF (Full Impact)</th>
<th>Controlling Operating RF (Reduced Impact)</th>
</tr>
</thead>
<tbody>
<tr>
<td>HS-20 (Inventory)</td>
<td>-28.93</td>
<td>-65.00</td>
<td>1.10</td>
<td>1.39</td>
</tr>
<tr>
<td>HS-20 (Operating)</td>
<td>-28.93</td>
<td>-65.00</td>
<td>1.43</td>
<td>1.81</td>
</tr>
<tr>
<td>Local Fire Truck</td>
<td>-28.93</td>
<td>-40.99</td>
<td>2.35</td>
<td>2.98</td>
</tr>
<tr>
<td>Local Sawdust Truck</td>
<td>-28.93</td>
<td>-68.18</td>
<td>1.41</td>
<td>1.79</td>
</tr>
</tbody>
</table>

* Both the Live Load and Dead Load Responses are unfactored responses.
Table 4.5 - Rating factors & responses for rated vehicles - Turnbuckle Tension – Current Condition.

<table>
<thead>
<tr>
<th>Truck</th>
<th>Dead Load Effects, kips</th>
<th>Live Load Effects, kips</th>
<th>Controlling Operating RF (Full Impact)</th>
<th>Controlling Operating RF (Reduced Impact)</th>
</tr>
</thead>
<tbody>
<tr>
<td>HS-20 (Inventory)</td>
<td>0.00</td>
<td>14.04</td>
<td>1.99</td>
<td>2.52</td>
</tr>
<tr>
<td>HS-20 (Operating)</td>
<td>0.00</td>
<td>14.04</td>
<td>2.58</td>
<td>3.27</td>
</tr>
<tr>
<td>Local Fire Truck</td>
<td>0.00</td>
<td>8.86</td>
<td>4.24</td>
<td>5.38</td>
</tr>
<tr>
<td>Local Sawdust Truck</td>
<td>0.00</td>
<td>14.70</td>
<td>2.56</td>
<td>3.24</td>
</tr>
</tbody>
</table>

* Both the Live Load and Dead Load Responses are unfactored responses.

Table 4.6 - Rating factors & responses for rated vehicles - Turnbuckle Tension – Tightened Turnbuckle to Minimum Level – 0.25” (10 kips).

<table>
<thead>
<tr>
<th>Truck</th>
<th>Dead Load Effects, kips</th>
<th>Live Load Effects, kips</th>
<th>Controlling Operating RF (Full Impact)</th>
<th>Controlling Operating RF (Reduced Impact)</th>
</tr>
</thead>
<tbody>
<tr>
<td>HS-20 (Inventory)</td>
<td>10.00</td>
<td>14.04</td>
<td>1.61</td>
<td>2.04</td>
</tr>
<tr>
<td>HS-20 (Operating)</td>
<td>10.00</td>
<td>14.04</td>
<td>2.08</td>
<td>2.64</td>
</tr>
<tr>
<td>Local Fire Truck</td>
<td>10.00</td>
<td>8.86</td>
<td>3.43</td>
<td>4.34</td>
</tr>
<tr>
<td>Local Sawdust Truck</td>
<td>10.00</td>
<td>14.70</td>
<td>2.07</td>
<td>2.62</td>
</tr>
</tbody>
</table>

* Both the Live Load and Dead Load Responses are unfactored responses.

Table 4.7 - Rating factors & responses for rated vehicles - Turnbuckle Tension – Tightened Turnbuckle to Maximum Level – 0.75” (30 kips).

<table>
<thead>
<tr>
<th>Truck</th>
<th>Dead Load Effects, kips</th>
<th>Live Load Effects, kips</th>
<th>Controlling Operating RF (Full Impact)</th>
<th>Controlling Operating RF (Reduced Impact)</th>
</tr>
</thead>
<tbody>
<tr>
<td>HS-20 (Inventory)</td>
<td>30.00</td>
<td>14.04</td>
<td>0.84</td>
<td>1.07</td>
</tr>
<tr>
<td>HS-20 (Operating)</td>
<td>30.00</td>
<td>14.04</td>
<td>1.09</td>
<td>1.38</td>
</tr>
<tr>
<td>Local Fire Truck</td>
<td>30.00</td>
<td>8.86</td>
<td>1.80</td>
<td>2.28</td>
</tr>
<tr>
<td>Local Sawdust Truck</td>
<td>30.00</td>
<td>14.70</td>
<td>1.08</td>
<td>1.37</td>
</tr>
</tbody>
</table>

* Both the Live Load and Dead Load Responses are unfactored responses.
Table 4.8 – Rating factors & responses for rated vehicles - Stringer Tees Shear – Current Condition.

<table>
<thead>
<tr>
<th>Truck</th>
<th>Dead Load Effects, kips</th>
<th>Live Load Effects, kips</th>
<th>Controlling Operating RF (Full Impact)</th>
<th>Controlling Operating RF (Reduced Impact)</th>
</tr>
</thead>
<tbody>
<tr>
<td>HS-20 (Inventory)</td>
<td>0.34</td>
<td>9.12</td>
<td>3.26</td>
<td>4.13</td>
</tr>
<tr>
<td>HS-20 (Operating)</td>
<td>0.34</td>
<td>9.12</td>
<td>4.22</td>
<td>5.35</td>
</tr>
<tr>
<td>Local Fire Truck</td>
<td>0.34</td>
<td>6.77</td>
<td>5.90</td>
<td>7.47</td>
</tr>
<tr>
<td>Local Sawdust Truck</td>
<td>0.36</td>
<td>6.10</td>
<td>6.55</td>
<td>8.30</td>
</tr>
</tbody>
</table>

* Both the Live Load and Dead Load Responses are unfactored responses.

Table 4.9 – Rating factors & responses for rated vehicles - Stringer Tees Shear – Tightened Turnbuckle to Minimum Level – 0.25” (10 kips).

<table>
<thead>
<tr>
<th>Truck</th>
<th>Dead Load Effects, kips</th>
<th>Live Load Effects, kips</th>
<th>Controlling Operating RF (Full Impact)</th>
<th>Controlling Operating RF (Reduced Impact)</th>
</tr>
</thead>
<tbody>
<tr>
<td>HS-20 (Inventory)</td>
<td>0.34</td>
<td>9.12</td>
<td>3.26</td>
<td>4.13</td>
</tr>
<tr>
<td>HS-20 (Operating)</td>
<td>0.34</td>
<td>9.12</td>
<td>4.22</td>
<td>5.35</td>
</tr>
<tr>
<td>Local Fire Truck</td>
<td>0.34</td>
<td>6.77</td>
<td>5.90</td>
<td>7.47</td>
</tr>
<tr>
<td>Local Sawdust Truck</td>
<td>0.36</td>
<td>6.10</td>
<td>6.55</td>
<td>8.30</td>
</tr>
</tbody>
</table>

* Both the Live Load and Dead Load Responses are unfactored responses.

Table 4.10 – Rating factors & responses for rated vehicles - Stringer Tees Shear – Tightened Turnbuckle to Maximum Level – 0.75” (30 kips).

<table>
<thead>
<tr>
<th>Truck</th>
<th>Dead Load Effects, kips</th>
<th>Live Load Effects, kips</th>
<th>Controlling Operating RF (Full Impact)</th>
<th>Controlling Operating RF (Reduced Impact)</th>
</tr>
</thead>
<tbody>
<tr>
<td>HS-20 (Inventory)</td>
<td>0.34</td>
<td>9.12</td>
<td>3.26</td>
<td>4.13</td>
</tr>
<tr>
<td>HS-20 (Operating)</td>
<td>0.34</td>
<td>9.12</td>
<td>4.22</td>
<td>5.35</td>
</tr>
<tr>
<td>Local Fire Truck</td>
<td>0.34</td>
<td>6.77</td>
<td>5.90</td>
<td>7.47</td>
</tr>
<tr>
<td>Local Sawdust Truck</td>
<td>0.36</td>
<td>6.10</td>
<td>6.55</td>
<td>8.30</td>
</tr>
</tbody>
</table>

* Both the Live Load and Dead Load Responses are unfactored responses.
### Table 4.11 - Rating factors & responses for rated vehicles - Floorbeam Shear – Current Condition.

<table>
<thead>
<tr>
<th>Truck</th>
<th>Dead Load Effects, kips</th>
<th>Live Load Effects, kips</th>
<th>Controlling RF (Full Impact)</th>
<th>Controlling RF (Reduced Impact)</th>
</tr>
</thead>
<tbody>
<tr>
<td>HS-20 (Inventory)</td>
<td>0.81</td>
<td>15.82</td>
<td>0.78</td>
<td>0.99</td>
</tr>
<tr>
<td>HS-20 (Operating)</td>
<td>0.75</td>
<td>15.78</td>
<td>1.02</td>
<td>1.29</td>
</tr>
<tr>
<td>Local Fire Truck</td>
<td>1.62</td>
<td>11.19</td>
<td>1.44</td>
<td>1.82</td>
</tr>
<tr>
<td>Local Sawdust Truck</td>
<td>1.62</td>
<td>10.73</td>
<td>1.50</td>
<td>1.90</td>
</tr>
</tbody>
</table>

* Both the Live Load and Dead Load Responses are unfactored responses.

### Table 4.12 - Rating factors & responses for rated vehicles - Floorbeam Shear – Tightened Turnbuckle to Minimum Level – 0.25” (10 kips).

<table>
<thead>
<tr>
<th>Truck</th>
<th>Dead Load Effects, kips</th>
<th>Live Load Effects, kips</th>
<th>Controlling RF (Full Impact)</th>
<th>Controlling RF (Reduced Impact)</th>
</tr>
</thead>
<tbody>
<tr>
<td>HS-20 (Inventory)</td>
<td>0.80</td>
<td>15.82</td>
<td>0.78</td>
<td>0.99</td>
</tr>
<tr>
<td>HS-20 (Operating)</td>
<td>0.74</td>
<td>15.78</td>
<td>1.02</td>
<td>1.29</td>
</tr>
<tr>
<td>Local Fire Truck</td>
<td>1.62</td>
<td>11.19</td>
<td>1.44</td>
<td>1.82</td>
</tr>
<tr>
<td>Local Sawdust Truck</td>
<td>1.62</td>
<td>10.73</td>
<td>1.50</td>
<td>1.90</td>
</tr>
</tbody>
</table>

* Both the Live Load and Dead Load Responses are unfactored responses.

### Table 4.13 - Rating factors & responses for rated vehicles - Floorbeam Shear – Tightened Turnbuckle to Maximum Level – 0.75” (30 kips).

<table>
<thead>
<tr>
<th>Truck</th>
<th>Dead Load Effects, kips</th>
<th>Live Load Effects, kips</th>
<th>Controlling RF (Full Impact)</th>
<th>Controlling RF (Reduced Impact)</th>
</tr>
</thead>
<tbody>
<tr>
<td>HS-20 (Inventory)</td>
<td>0.78</td>
<td>15.82</td>
<td>0.78</td>
<td>0.99</td>
</tr>
<tr>
<td>HS-20 (Operating)</td>
<td>0.72</td>
<td>15.78</td>
<td>1.02</td>
<td>1.29</td>
</tr>
<tr>
<td>Local Fire Truck</td>
<td>1.62</td>
<td>11.19</td>
<td>1.44</td>
<td>1.82</td>
</tr>
<tr>
<td>Local Sawdust Truck</td>
<td>1.62</td>
<td>10.73</td>
<td>1.50</td>
<td>1.90</td>
</tr>
</tbody>
</table>

* Both the Live Load and Dead Load Responses are unfactored responses.
Table 4.14 - Rating factors & responses for rated vehicles - Girder Shear – Current Condition.

<table>
<thead>
<tr>
<th>Truck</th>
<th>Dead Load Effects, kips</th>
<th>Live Load Effects, kips</th>
<th>Controlling RF (Full Impact)</th>
<th>Controlling RF (Reduced Impact)</th>
</tr>
</thead>
<tbody>
<tr>
<td>HS-20 (Inventory)</td>
<td>57.73</td>
<td>32.36</td>
<td>0.84</td>
<td>1.06</td>
</tr>
<tr>
<td>HS-20 (Operating)</td>
<td>57.73</td>
<td>32.36</td>
<td>1.09</td>
<td>1.38</td>
</tr>
<tr>
<td>Local Fire Truck</td>
<td>57.73</td>
<td>21.15</td>
<td>1.73</td>
<td>2.19</td>
</tr>
<tr>
<td>Local Sawdust Truck</td>
<td>57.73</td>
<td>34.23</td>
<td>1.07</td>
<td>1.36</td>
</tr>
</tbody>
</table>

* Both the Live Load and Dead Load Responses are-unfactored responses.

Table 4.15 - Rating factors & responses for rated vehicles - Girder Shear – Tightened Turnbuckle to Minimum Level – 0.25” (10 kips).

<table>
<thead>
<tr>
<th>Truck</th>
<th>Dead Load Effects, kips</th>
<th>Live Load Effects, kips</th>
<th>Controlling RF (Full Impact)</th>
<th>Controlling RF (Reduced Impact)</th>
</tr>
</thead>
<tbody>
<tr>
<td>HS-20 (Inventory)</td>
<td>57.70</td>
<td>32.36</td>
<td>0.84</td>
<td>1.06</td>
</tr>
<tr>
<td>HS-20 (Operating)</td>
<td>57.70</td>
<td>32.36</td>
<td>1.09</td>
<td>1.38</td>
</tr>
<tr>
<td>Local Fire Truck</td>
<td>57.70</td>
<td>21.15</td>
<td>1.73</td>
<td>2.19</td>
</tr>
<tr>
<td>Local Sawdust Truck</td>
<td>57.70</td>
<td>34.23</td>
<td>1.07</td>
<td>1.36</td>
</tr>
</tbody>
</table>

* Both the Live Load and Dead Load Responses are-unfactored responses.

Table 4.16 - Rating factors & responses for rated vehicles - Girder Shear – Tightened Turnbuckle to Maximum Level – 0.75” (30 kips).

<table>
<thead>
<tr>
<th>Truck</th>
<th>Dead Load Effects, kips</th>
<th>Live Load Effects, kips</th>
<th>Controlling RF (Full Impact)</th>
<th>Controlling RF (Reduced Impact)</th>
</tr>
</thead>
<tbody>
<tr>
<td>HS-20 (Inventory)</td>
<td>57.63</td>
<td>32.36</td>
<td>0.84</td>
<td>1.06</td>
</tr>
<tr>
<td>HS-20 (Operating)</td>
<td>57.63</td>
<td>32.36</td>
<td>1.09</td>
<td>1.38</td>
</tr>
<tr>
<td>Local Fire Truck</td>
<td>57.63</td>
<td>21.15</td>
<td>1.73</td>
<td>2.19</td>
</tr>
<tr>
<td>Local Sawdust Truck</td>
<td>57.63</td>
<td>34.23</td>
<td>1.07</td>
<td>1.36</td>
</tr>
</tbody>
</table>

* Both the Live Load and Dead Load Responses are-unfactored responses.
Table 4.17 – Rating factors & responses for rated vehicles - Stringer Tees Moment – All Conditions (Turnbuckle does not significantly affect Stringers).

<table>
<thead>
<tr>
<th>Truck</th>
<th>Dead Load Effects, kip-inch</th>
<th>Live Load Effects, kip-in</th>
<th>Controlling RF (Full Impact)</th>
<th>Controlling RF (Reduced Impact)</th>
</tr>
</thead>
<tbody>
<tr>
<td>HS-20 (Inventory)</td>
<td>11.14</td>
<td>288</td>
<td>0.71</td>
<td>0.90</td>
</tr>
<tr>
<td>HS-20 (Operating)</td>
<td>11.14</td>
<td>288</td>
<td>0.92</td>
<td>1.17</td>
</tr>
<tr>
<td>Local Fire Truck</td>
<td>11.14</td>
<td>158.11</td>
<td>1.74</td>
<td>2.21</td>
</tr>
<tr>
<td>Local Sawdust Truck</td>
<td>11.14</td>
<td>128.61</td>
<td>2.14</td>
<td>2.71</td>
</tr>
</tbody>
</table>

Note: - Both the Live Load and Dead Load Responses are-unfactored responses.
- Stringer moment ratings were based on simple span assumption due to model mesh refinement issues.
Table 4.18 - Rating factors & responses for rated vehicles - Floorbeam Moment – Current Condition.

<table>
<thead>
<tr>
<th>TRUCK</th>
<th>DEAD LOAD EFFECTS, KIP-INCH</th>
<th>LIVE LOAD EFFECTS, KIP-IN</th>
<th>CONTROLLING RF (FULL IMPACT)</th>
<th>CONTROLLING RF (REDUCED IMPACT)</th>
</tr>
</thead>
<tbody>
<tr>
<td>HS-20 (Inventory)</td>
<td>30.44</td>
<td>475.74</td>
<td>1.32</td>
<td>1.67</td>
</tr>
<tr>
<td>HS-20 (Operating)</td>
<td>30.44</td>
<td>475.74</td>
<td>1.71</td>
<td>2.17</td>
</tr>
<tr>
<td>Local Fire Truck</td>
<td>52.17</td>
<td>361.54</td>
<td>2.29</td>
<td>2.90</td>
</tr>
<tr>
<td>Local Sawdust Truck</td>
<td>52.17</td>
<td>343.15</td>
<td>2.41</td>
<td>3.05</td>
</tr>
</tbody>
</table>

* Both the Live Load and Dead Load Responses are unfactored responses.

Table 4.19 - Rating factors & responses for rated vehicles - Floorbeam Moment – Tightened Turnbuckle to Minimum Level – 0.25” (10 kips).

<table>
<thead>
<tr>
<th>TRUCK</th>
<th>DEAD LOAD EFFECTS, KIP-INCH</th>
<th>LIVE LOAD EFFECTS, KIP-IN</th>
<th>CONTROLLING RF (FULL IMPACT)</th>
<th>CONTROLLING RF (REDUCED IMPACT)</th>
</tr>
</thead>
<tbody>
<tr>
<td>HS-20 (Inventory)</td>
<td>30.30</td>
<td>475.74</td>
<td>1.32</td>
<td>1.67</td>
</tr>
<tr>
<td>HS-20 (Operating)</td>
<td>30.30</td>
<td>475.74</td>
<td>1.71</td>
<td>2.17</td>
</tr>
<tr>
<td>Local Fire Truck</td>
<td>52.05</td>
<td>361.54</td>
<td>2.29</td>
<td>2.90</td>
</tr>
<tr>
<td>Local Sawdust Truck</td>
<td>52.05</td>
<td>343.15</td>
<td>2.41</td>
<td>3.05</td>
</tr>
</tbody>
</table>

* Both the Live Load and Dead Load Responses are unfactored responses.

Table 4.20 - Rating factors & responses for rated vehicles - Floorbeam Moment – Tightened Turnbuckle to Maximum Level – 0.75” (30 kips).

<table>
<thead>
<tr>
<th>TRUCK</th>
<th>DEAD LOAD EFFECTS, KIP-INCH</th>
<th>LIVE LOAD EFFECTS, KIP-IN</th>
<th>CONTROLLING RF (FULL IMPACT)</th>
<th>CONTROLLING RF (REDUCED IMPACT)</th>
</tr>
</thead>
<tbody>
<tr>
<td>HS-20 (Inventory)</td>
<td>30.04</td>
<td>475.74</td>
<td>1.32</td>
<td>1.67</td>
</tr>
<tr>
<td>HS-20 (Operating)</td>
<td>30.04</td>
<td>475.74</td>
<td>1.71</td>
<td>2.17</td>
</tr>
<tr>
<td>Local Fire Truck</td>
<td>51.81</td>
<td>361.54</td>
<td>2.29</td>
<td>2.90</td>
</tr>
<tr>
<td>Local Sawdust Truck</td>
<td>51.81</td>
<td>343.15</td>
<td>2.41</td>
<td>3.05</td>
</tr>
</tbody>
</table>

* Both the Live Load and Dead Load Responses are unfactored responses.
Table 4.21 - Rating factors & responses for rated vehicles - Girder Moment – Current Condition.

<table>
<thead>
<tr>
<th>Truck</th>
<th>Dead Load Effects, kip-inch</th>
<th>Live Load Effects, kip-inch</th>
<th>Controlling RF (Full Impact)</th>
<th>Controlling RF (Reduced Impact)</th>
</tr>
</thead>
<tbody>
<tr>
<td>HS-20 (Inventory)</td>
<td>19633.03</td>
<td>6914.90</td>
<td>0.46</td>
<td>0.58</td>
</tr>
<tr>
<td>HS-20 (Operating)</td>
<td>19517.81</td>
<td>7005.80</td>
<td>0.60</td>
<td>0.76</td>
</tr>
<tr>
<td>Local Fire Truck</td>
<td>19633.03</td>
<td>4620.40</td>
<td>0.93</td>
<td>1.18</td>
</tr>
<tr>
<td>Local Sawdust Truck</td>
<td>19517.81</td>
<td>5943.60</td>
<td>0.74</td>
<td>0.94</td>
</tr>
</tbody>
</table>

* Both the Live Load and Dead Load Responses are unfactored responses.

Table 4.22 - Rating factors & responses for rated vehicles - Girder Moment – Tightened Turnbuckle to Minimum Level – 0.25” (10 kips).

<table>
<thead>
<tr>
<th>Truck</th>
<th>Dead Load Effects, kip-inch</th>
<th>Live Load Effects, kip-inch</th>
<th>Controlling RF (Full Impact)</th>
<th>Controlling RF (Reduced Impact)</th>
</tr>
</thead>
<tbody>
<tr>
<td>HS-20 (Inventory)</td>
<td>19131.40</td>
<td>7005.80</td>
<td>0.50</td>
<td>0.63</td>
</tr>
<tr>
<td>HS-20 (Operating)</td>
<td>19131.40</td>
<td>7005.80</td>
<td>0.64</td>
<td>0.81</td>
</tr>
<tr>
<td>Local Fire Truck</td>
<td>19131.40</td>
<td>4621.10</td>
<td>1.01</td>
<td>1.28</td>
</tr>
<tr>
<td>Local Sawdust Truck</td>
<td>19131.40</td>
<td>5943.60</td>
<td>0.79</td>
<td>1.00</td>
</tr>
</tbody>
</table>

* Both the Live Load and Dead Load Responses are unfactored responses.

Table 4.23 - Rating factors & responses for rated vehicles - Girder Moment – Tightened Turnbuckle to Minimum Level – 0.25” (10 kips).

<table>
<thead>
<tr>
<th>Truck</th>
<th>Dead Load Effects, kip-inch</th>
<th>Live Load Effects, kip-inch</th>
<th>Controlling RF (Full Impact)</th>
<th>Controlling RF (Reduced Impact)</th>
</tr>
</thead>
<tbody>
<tr>
<td>HS-20 (Inventory)</td>
<td>18358.61</td>
<td>7005.80</td>
<td>0.56</td>
<td>0.71</td>
</tr>
<tr>
<td>HS-20 (Operating)</td>
<td>18358.61</td>
<td>7005.80</td>
<td>0.72</td>
<td>0.91</td>
</tr>
<tr>
<td>Local Fire Truck</td>
<td>18223.67</td>
<td>4700.80</td>
<td>1.13</td>
<td>1.43</td>
</tr>
<tr>
<td>Local Sawdust Truck</td>
<td>18358.61</td>
<td>5943.60</td>
<td>0.88</td>
<td>1.11</td>
</tr>
</tbody>
</table>

* Both the Live Load and Dead Load Responses are unfactored responses.
### Table 4.24 – Rating factors & responses - Approach Stringer Shear – Current Condition.

<table>
<thead>
<tr>
<th>Truck</th>
<th>Dead Load Effects, Kips</th>
<th>Live Load Effects, Kips</th>
<th>Controlling RF</th>
</tr>
</thead>
<tbody>
<tr>
<td>HS-20 (Inventory)</td>
<td>0.97</td>
<td>13.04</td>
<td>1.17</td>
</tr>
<tr>
<td>HS-20 (Operating)</td>
<td>0.97</td>
<td>13.04</td>
<td>1.51</td>
</tr>
<tr>
<td>Local Fire Truck</td>
<td>0.88</td>
<td>10.08</td>
<td>2.03</td>
</tr>
<tr>
<td>Local Sawdust Truck</td>
<td>0.96</td>
<td>10.02</td>
<td>1.90</td>
</tr>
</tbody>
</table>

* Both the Live Load and Dead Load Responses are unfactored responses.

### Table 4.25 – Rating factors & responses - Approach Stringer Moment – Current Condition.

<table>
<thead>
<tr>
<th>Truck</th>
<th>Dead Load Effects, KIP-INCH</th>
<th>Live Load Effects, KIP-INCH</th>
<th>Controlling Rating Factor (Unbraced)</th>
<th>Controlling Rating Factor (Braced at Quarter Points)</th>
</tr>
</thead>
<tbody>
<tr>
<td>HS-20 (Inventory)</td>
<td>64.88</td>
<td>514.9</td>
<td>0.51</td>
<td>1.42</td>
</tr>
<tr>
<td>HS-20 (Operating)</td>
<td>64.88</td>
<td>514.9</td>
<td>0.66</td>
<td>1.85</td>
</tr>
<tr>
<td>Local Fire Truck</td>
<td>64.88</td>
<td>409.03</td>
<td>0.86</td>
<td>2.41</td>
</tr>
<tr>
<td>Local Sawdust Truck</td>
<td>64.78</td>
<td>402.14</td>
<td>0.87</td>
<td>2.28</td>
</tr>
</tbody>
</table>

* Both the Live Load and Dead Load Responses are unfactored responses.

### Table 4.26 - Secondary rating factor & responses considering removal of both the Kingpost & Covering - Girder Moment.

<table>
<thead>
<tr>
<th>Truck</th>
<th>Dead Load Effects, KIP-INCH</th>
<th>Live Load Effects, KIP-INCH</th>
<th>Controlling RF</th>
</tr>
</thead>
<tbody>
<tr>
<td>Local Sawdust Truck (Full Impact)</td>
<td>12333</td>
<td>13614</td>
<td>0.70</td>
</tr>
<tr>
<td>Local Sawdust Truck (Reduced Impact)</td>
<td>12419</td>
<td>13688</td>
<td>0.88</td>
</tr>
</tbody>
</table>

* Both the Live Load and Dead Load Responses are unfactored responses.
5. CONCLUSIONS

In general, the response data recorded during the load tests was found to be of good quality and indicated no major signs of distress. The test data exhibited response magnitudes and shapes typical of at two girder structure and of a multi-stringer structure. The following notable observations were made from review of the load test data:

- Significant end-restraint was observed at the girder supports of the covered span and was likely due to the embedment/connection of the girder-ends to the abutment concrete and adjacent stringers. This behavior reduced the midspan moment and was partially considered for rating.

- The kingpost and turnbuckle system was significantly activated by the test truck. Both axial compressive stress and flexural stress were observed near the top of the kingpost. The relative magnitude of the flexural stress was somewhat concerning because it may be necessary to treat the kingpost struts as beam-columns rather than just compression members. The initial load rating capacities were calculated based on compression member specifications, however flexural and axial stress ratios were generated for additional consideration.

- Near the damaged girder section (section with a severely bent bottom flange), a substantial amount of out-of-plane bending was observed in the top flange. This indicated a non-uniform stress profile across the compression flange which would cause one edge to reach yield stress well before the theoretical moment capacity was reached. While this was not considered in the initial load rating, information about the compression flange stress profile has been provided to help determine if additional reduction factors should be applied to the moment capacity.

- Significant dynamic effects (20%) were observed during the normal speed test (5-7 mph). However negligible dynamics were observed during the crawl speed tests (1-3 mph). This observation indicates that if the heavier trucks cross at crawl speed, a much larger weight can safely cross.

A quasi three-dimensional finite-element model of the structure was created using the collected structural information, and subsequently calibrated until an acceptable match between the measured and analytical responses was achieved. A good correlation between the measured and computed response was obtained during the modeling process.

Load ratings for the standard HS-20 rating and the local fire and sawdust trucks were performed according to the AASHTO Manual for Bridge Evaluation, 2011 Edition. The following conclusions were made from these ratings:

- Load rating results were controlled by the ultimate flexural capacity of the girders near midspan. Only the local fire truck crossing at crawl speed had a satisfactory load rating, which indicated that with the structure in its current state only the fire truck can cross the structure safely if it does so at crawl speed (1-3 mph).

- A primary conclusion from these ratings was that the dead-load responses were much greater than the live-load responses. It was found that the bridge covering accounted for approximately 30% of the dead-load, which greatly reduced the remaining live-load capacity.
The kingpost system was determined to be an important structural element and effectively reduced the peak live-load girder moment by approximately 40 percent. While not the critical rating component, the kingposts were found to have slightly deficient compression capacity at the upper section with no cover-plates. A recommended retrofit would be to extend the existing cover-plates to the ends and thereby increase the compression capacity by over 40 percent.

Other than a 0.85 condition factor at the damaged girder section, no additional capacity reductions or live-load amplifiers were applied to account for the observed bending stresses on primary compression members. Further investigation and consideration may be needed to account for bending effects in the kingpost and the top girder flange.

The approach span’s stringers have unsatisfactory ratings in their current state due to the lack of mechanical attachment to the decking members or other lateral bracing of the top flanges. Bracing the stringer top flanges at quarter points would increase the moment capacity so that all examined vehicles would have sufficient load ratings.

In addition to the load ratings performed on the structure in its current condition, an iterative analysis and rating process was performed in order to determine the effects of tightening the turnbuckle and increasing the participation of the kingpost system. The following was determined from this process:

- It was found that by tightening the turnbuckle to 10 kips (shortening the turnbuckle by approximately ¼”) the sawdust truck could cross at crawl speed (using reduced impact). With the turnbuckle tightened to this minimum level, the girders had a satisfactory rating in flexure of 1.00.

- Additionally, it was determined that if the turnbuckle was tightened to 30 kips of force (~3/4” turnbuckle shortening) that the sawdust truck had a higher overall live-load carrying capacity, with the controlling rating factor at crawl speed of 1.11 for girder flexure. It should be noted that at this level of kingpost system participation, that the turnbuckle rating was 1.37 for reduced impact. Since breaking of the turnbuckle is not a ductile failure and therefore not desired, BDI considered this to be the maximum recommended level of turnbuckle tightening.

Additional information about BDI’s integrated approach (testing, modeling and rating procedures) and other supporting documents are available at www.bridgetest.com.

The load test, structural investigation, and load rating results presented in this report correspond to the structure at the time of testing. Any structural degradation, damage, and/or retrofits must be taken into account in future ratings.
LRFD NONCOMPOSITE GIRDER CAPACITY

Structure: Milo Academy Bridge (Built-up Girder)
Location: Moment - At Midspan, Shear - Near Support

Beam Dimensions:
Beam Depth: \( d = 73 \text{ in} \)

Flange Thickness:
Top Flange: \( t_{f,top} = 0.75 \text{ in} \)
Bottom Flange: \( t_{f,bot} = 0.75 \text{ in} \)

Flange Width:
Top Flange: \( b_{f,top} = 11.375 \text{ in} \)
Bottom Flange: \( b_{f,bot} = 14 \text{ in} \)

Web Depth:
\( D_c = 36.4 \text{ in} \)

Depth of Web in Compression:
\( D_c = d - t_{f,top} - t_{f,bot} - 7.15 \text{ in} \)

Distance Between Web Centerlines:
\( h = d - \frac{(t_{f,top} + t_{f,bot})}{2} = 72.25 \text{ in} \)

Web Width:
\( t_w = 0.375 \text{ in} \)

Unbraced Length:
\( I_u = 6 \text{ ft} \)

Beam Properties:

Steel Yield & Ultimate Strength:
\( F_y = 33 \text{ ksi} \)
\( F_u = 66 \text{ ksi} \)

Compression Flange Stress at Yield Onset:
\( F_{yr} = 0.7F_y = 23.1 \text{ ksi} \)

Modulus of Elasticity:
\( E = 29000 \text{ ksi} \)

Moment of Inertia of Compression Flange:
\( I_{yc} = \frac{(t_{f,top}b_{f,top})^3}{12} = 91.99 \text{ in}^4 \)

Moment of Inertia of Tension Flange:
\( I_{yt} = \frac{(t_{f,bot}(b_{f,bot})^3}{12} = 171.5 \text{ in}^4 \)

Major Axis Elastic Section Modulus:
\( S_{x,top} = 1247.4 \text{ in}^3 \)
\( S_{x,bot} = 1292.6 \text{ in}^3 \)

Torsional Constant:
\( J_{beam} = 2.59 \text{ in}^4 \)

Radius of Gyration:
\( r_g = \sqrt{\frac{D_c}{12\left(\frac{h}{d} + \frac{1}{3}\left(\frac{D_c}{b_{f,top}}\frac{t_w}{t_{f,top}}h_d\right)\right)^2}} = 2.68 \text{ in} \)
**Strength Limit State Moment Capacity Calculations:**

\[
CHECK_{WS} := \text{if } \left( \frac{2D_c}{t_w} < 5.7 \sqrt{\frac{E}{F_y}} \right) = 0
\]

\[
CHECK_{FS} := \text{if } \left( \frac{t_c}{t_y} \geq 0.3, 1, 0 \right) = 1
\]

Capacity := if \( CHECK_{WS} = 1 \land CHECK_{FS} = 1 \), "Appendix A6 Capacity", "6.10.8 Capacity" = "6.10.8 Capacity"

**Beam Factor Calculations:**

**Hybrid Factor:**

\[
\beta := \frac{(2D_c - t_w)}{(b_{t,\text{top}}}b_{t,\text{top}})} = 3.2
\]

\[
\rho = 1
\]

\[
R_h = \left[ \frac{12 + \beta \left( 3 \rho - \rho^3 \right)}{(12 + 2 \beta)^2} \right] = 1
\]

**Web Shedding Factor:**

**Compression Flange Slenderness Ratio:**

\[
\lambda_w := \frac{(2D_c)}{t_w} = 194.133
\]

**Limiting Slenderness Ratio for Noncompact Web:**

\[
\lambda_{TW} := 5.7 \sqrt{\frac{E}{F_y}} = 168.973
\]

**Area Ratio between Compression Flange & Web:**

\[
a_{WC} := \frac{2D_c - t_w}{b_{t,\text{top}} b_{t,\text{top}}} = 3.2
\]

\[
R_B := \text{if } \lambda_w < \lambda_{TW}, 1.0, \left[ 1 - \left( \frac{2a_{WC}}{1200 + 300a_{WC}} \right) (\lambda_w - \lambda_{TW}) \right] = 0.96
\]
**Local Buckling Resistance:**

*Compression Flange Slenderness Ratio:*

\[ \lambda_f = \frac{b_{flattened}}{2h_{top}} = 7.583 \]  
AASHTO LRFD 6.10.8.2.2.3

*Compact Flange Limiting Slenderness Ratio:*

\[ \lambda_{pf} = 0.38 \cdot \frac{F}{F_y} = 11.265 \]  
AASHTO LRFD 6.10.8.2.4

*Noncompact Flange Limiting Slenderness Ratio:*

\[ \lambda_{nf} = 0.56 \cdot \frac{F}{F_y} = 19.842 \]  
AASHTO LRFD 6.10.8.2.2

**Local Buckling Resistance of the Compression Flange:**

\[ F_{nc1} = \left[ \left( \lambda_f < \lambda_{pf} \right) \cdot R_h \cdot R_b \cdot F_y \cdot \left[ 1 - \left( \lambda_f - \lambda_{pf} \right) \cdot R_h \cdot R_b \cdot F_y \right] \right] \]  
AASHTO LRFD 6.10.8.2.2

**Lateral Torsional Buckling Resistance:**

*Moment Gradient Modifier:*

assumed to be 1.0

\[ C_b = 1.0 \]

**Limiting Lengths:**

\[ I_1 = \pi \cdot t_1 \cdot \frac{E}{F_y} = 24.815 \text{ ft} \]  
AASHTO LRFD 6.10.8.2.3-5

\[ I_2 = 1.0 \cdot t_1 \cdot \frac{E}{F_y} = 6.609 \text{ ft} \]  
AASHTO LRFD 6.10.8.2.4

*L_b < L_p:*

\[ F_{nc2a} = R_h \cdot R_b \cdot F_y = 31.77 \text{ ksi} \]

*L_p < L_b < L_r:*

\[ F_{nc2b} = \left[ 1 - \left( \frac{F_y}{R_h \cdot F_y} \right) \left( \frac{L_b - L_p}{L_1 - L_p} \right) \right] \cdot R_h \cdot R_b \cdot F_y = 32.09 \text{ ksi} \]

*L_b > L_r:*

\[ r_{cr} = \left( \frac{C_b}{F_y} \right) \frac{L_b^2}{t_1} = 395.12 \text{ ksi} \]  
AASHTO LRFD 6.10.8.2.3-8

\[ F_{nc2c} = \min \left( F_{cr}, R_h \cdot R_b \cdot F_y \right) = 31.77 \text{ ksi} \]

**Lateral Torsional Buckling Controlling Resistance:**

\[ F_{nc2} = \text{if} \left( I_b < I_p, F_{nc2a}, \text{if} \left( I_b > I_r, F_{nc2c}, F_{nc2b} \right) \right) = 31.77 \text{ ksi} \]
**Tension Flange Yielding Resistance:**

**Tension Flange Yield Stress:**

\[ F_{nt} := F_{ht} F_y = 33 \text{ ksi} \quad \text{AASHTO LRFD 6.10.8.3} \]

**Nominal Moment Strength:**

\[ M_n = \text{min} \left( \min \left( \frac{F_{nc1}}{F_{nc2}} \right), \frac{S_{x,\text{top}}}{S_{x,\text{bot}}} \left( \frac{F_{nt}}{F_y} \right) \right) \left| S_{x,\text{bot}} \right| = 39630 \text{ kip-ft} \]

**Shear Strength Calculations:**

**Unbraced Web Depth:**

\[ D_w = D - 17 \text{ in} = 54.5 \text{ in} \quad \text{AASHTO LRFD 6.10.9.2} \]

**Shear Buckling Coefficient:**

\[ k = 5 \quad \text{AASHTO LRFD 6.10.9.2} \]

**Depth to Thickness Ratio Limits:**

\[ \text{lim}_1 := 1.12 \frac{\sqrt{(E-K)}}{F_y} = 74.24 \quad \text{AASHTO LRFD 6.10.9.3.2-4} \]

\[ \text{lim}_2 := 1.40 \frac{\sqrt{(E-K)}}{F_y} = 92.8 \quad \text{AASHTO LRFD 6.10.9.3.2-5} \]

**Depth to Thickness Ratio:**

\[ \text{web ratio} = \frac{D_w}{t_w} = 145.33 \quad \text{AASHTO LRFD 6.10.9.3.2} \]

**Web Shear Coefficient:**

\[ C_v = \begin{cases} 1.12 \frac{(E-K)}{F_y} & \text{if web ratio } < \text{lim}_1, 1.0, \text{if web ratio } > \text{lim}_2, \\ \frac{1.57}{(E-K)} \frac{(E-K)}{F_y} & \left( \frac{D_w}{t_w} \right)^2 \left( \frac{D_w}{t_w} \right) \end{cases} = 0.33 \quad \text{AASHTO LRFD 6.10.9.3.2} \]

**Plastic Shear Force:**

\[ V_p := 0.58 F_y D t_w = 513.19 \text{ kip} \quad \text{AASHTO LRFD 6.10.9.2-2} \]

**Nominal Shear Strength:**

\[ V_n = C_v V_p = 167.61 \text{ kip} \quad \text{AASHTO LRFD 6.10.9.2-1} \]
LRFD NONCOMPOSITE MEMBER CAPACITY

Structure: Milo Academy Bridge (Transom Trestle As Floorbeams)
Location: Moment - At Midspan, Shear - Near Support

Beam Dimensions:
Beam Depth: \( d = 10.75 \text{ in} \)
Flange Thickness (w/ Cover Plate): \( t_f = 0.825 \text{ in} \)
Flange Width (Average w/ Cover Plate): \( b_f = 4.27 \text{ in} \)
Web Depth: \( D := d - t_f - t_f = 9.1 \text{ in} \)
Depth of Web in Compression: \( D_c = \frac{D}{2} = 4.55 \text{ in} \)
Distance Between Web Centerlines: \( h = d - t_f = 9.925 \text{ in} \)
Web Width: \( t_w = 0.25 \text{ in} \)
Unbraced Length: \( l_b = 4.5 \text{ ft} \)

Beam Properties:
Steel Yield & Ultimate Strength: \( f_y = 50 \text{ ksi} \)
Compression Flange Stress at Yield Onset: \( f_{fy} = 0.7 f_y = 35 \text{ ksi} \)
Modulus of Elasticity: \( E = 290000 \text{ ksi} \)
Moment of Inertia of Compression/Tension Flange:
\[ I_{yc} = \frac{b_f t_f^3}{12} = 5.35 \text{ in}^4 \]
\[ I_{yt} = \frac{[b_f (t_f)]^3}{12} = 5.35 \text{ in}^4 \]
Major Axis Elastic Section Modulus: \( S_x = 35.11 \text{ in}^3 \)
Major Axis Elastic Section Modulus: \( Z_x = 41.3 \text{ in}^3 \)
Torsional Constant: \( I_{beam} = 0.461 \text{ in}^4 \)
Radius of Gyration:
\[ r_l = \sqrt{\frac{b_f}{12} \left[ 1 + \frac{D_c}{3} \frac{t_w}{b_f t_f} \right]} = 1.17 \text{ in} \]
Strength Limit State Moment Capacity Calculations:

\[
\text{CHECK}_{\text{WS}} = \begin{cases} 
1 & \text{if } \left( \frac{2D_c}{t_w} \right) < 5.7 \sqrt{\frac{E}{F_y}} \geq 1.0 \\
0 & \text{otherwise}
\end{cases}
\]

\[
\text{CHECK}_{\text{FS}} = \begin{cases} 
1 & \text{if } \left( \frac{y_c}{y_t} \right) \geq 0.3, 1.0 \\
0 & \text{otherwise}
\end{cases}
\]

Capacity := if (CHECK_{WS} = 1 ∧ CHECK_{FS} = 1, "Appendix A6 Capacity", "6.10.2 Capacity") = "Appendix A6 Capacity"

Web Plastification Factor Calculations:

Beam Yield Moment:
\[
M_y := F_y \cdot S_x = 1755.5 \text{ kip in}
\]

Beam Plastic Moment:
\[
M_p := F_y \cdot Z_x = 2065 \text{ kip in}
\]

Hybrid Factor:
\[
\beta := \left( \frac{2D_c}{t_w} \right) = 0.646
\]

\[
\rho := 1
\]

\[
R_h := \frac{12 + \beta \left( 3 - \rho - \beta \right)}{12 + 2 \beta} = 1
\]

Depth of Web in Compression at Plastic Moment:
\[
D_{cp} = \left( \frac{D}{2} \right) \left[ \frac{n_Tb^-Y^-b_TY^T}{F_y D_tw} \right] + 1 = 4.55 \text{ in}
\]

Limiting Slenderness Ratio for Compact Web:
\[
\lambda_{pw1} = \frac{E}{F_y} \left( \frac{M_p}{R_h M_y} \right) - 0.69 = 44.173
\]

Limiting Slenderness Ratio for Noncompact Web:
\[
\lambda_{pw2} = \lambda_{pw1} \left( \frac{D_{cp}}{D_c} \right) = 137.274
\]

\[
\lambda_{pw} = \min(\lambda_{pw1}, \lambda_{pw2}) = 44.173
\]

\[
\lambda_w = \left( \frac{2D_c}{t_w} \right) = 36.4
\]

Compact Check := if (\lambda_{pw} ≥ \lambda_w, "COMPACT", "NONCOMPACT") = "COMPACT"
Web Plastification Factor: AASHTO LRFD A6.2.2.4

\[ R_p = \frac{1}{\text{Compact Check} = \text{"COMPACT"}, \left( \frac{M_p}{M_y} \right)_{\min} \left[ 1 - \frac{\left( R_p \cdot M_y \right)_{\min} \left[ \frac{\lambda_{f} - \lambda_{pf}}{(1 - \lambda_{pf})} \right] \cdot \frac{M_p}{M_y} \right] \cdot \frac{M_p}{M_y} }{1.176} \]

**Local Buckling Resistance:**

**Compression Flange Slenderness Ratio:**
\[ \lambda_f = \frac{b_f}{2\cdot t_f} = 2.588 \quad \text{AASHTO LRFD A6.3.2.3} \]

**Compact Flange Limiting Slenderness Ratio:**
\[ \lambda_{pf} = 0.38 \frac{E}{F_y} = 9.152 \quad \text{AASHTO LRFD A6.3.2.4} \]

**Flange Local Buckling Coefficient:**
\[ k_c = \max \left( \frac{4}{D}, \frac{1}{\sqrt{\lambda_{w}}} \right) = 0.663 \quad \text{AASHTO LRFD A6.3.2.6} \]

**Noncompact Flange Limiting Slenderness Ratio:**
\[ \lambda_{nf} = 0.95 \frac{E}{F_{yr}} k_c = 22.266 \quad \text{AASHTO LRFD A6.3.2.5} \]

**Local Buckling Resistance:**
\[ M_{nc1} = \left[ \lambda_f < \lambda_{pf}, R_p \cdot M_y \left[ 1 - \frac{\left( \frac{F_{yr} \cdot S_x}{S_y} \right) \left( \lambda_f - \lambda_{pf} \right)}{R_p \cdot M_y \cdot \lambda_{nf} - \lambda_{pf}} \right] \cdot R_p \cdot M_y \right] = 2065\text{-kip-in} \quad \text{AASHTO LRFD A6.3.2.1.2} \]

**Lateral Torsional Buckling Resistance:**

**Limiting Lengths:**
\[ L_y = 1.95 t_f \left( \frac{F_{yr}}{E} \right) \sqrt{t_{\text{beam}} \frac{I_{\text{beam}}}{S_y \cdot b}} \sqrt{1 + \frac{6.76 \left( \frac{F_{yr} \cdot S_x}{E} \right)^2}{S_y \cdot \text{beam}}} = 10.844\text{ft} \quad \text{AASHTO LRFD A6.3.3.5} \]

\[ L_p = 1.0 t_f \sqrt{\frac{E}{F_y}} = 2.351\text{ft} \quad \text{AASHTO LRFD A6.3.3.4} \]

\[ L_y < L_p: \quad M_{nc2a} = R_p \cdot M_y = 2065\text{-kip-in} \quad \text{AASHTO LRFD A6.3.3.1} \]

\[ L_p < L_y < L_r: \quad M_{nc2b} = \left[ 1 - \frac{\left( \frac{F_{yr} \cdot S_x}{S_y} \right) \left( \frac{b_f - L_p}{L_r - L_p} \right)}{R_p \cdot M_y} \right] \cdot R_p \cdot M_y = 1853.39\text{-kip-in} \quad \text{AASHTO LRFD A6.3.3.2} \]
\[ F_{cr} = \frac{P_{cr}}{A_{cr}} = \left( \frac{I}{L_b} \right)^{2/3} \sqrt{1 + \frac{1}{1 - \frac{L_b}{L_r}} \left( \frac{I_{beam}}{S_x h} \right)} = 148.68 \text{ ksi} \]

\[ M_{nc2c} = F_{cr} S_x = 5220.16 \text{ kip in} \]

**Lateral Torsional Buckling Controlling Resistance:**

\[ M_{nc2} = \begin{cases} M_{pc} & \text{if} \left( L_b < L_p, M_{nc2a} \right) \\ M_{nc2c} & \text{if} \left( L_b > L_r, M_{nc2c} > M_{nc2a} \right) \end{cases} = 1853.39 \text{ kip in} \]

**Tension Flange Yielding Resistance:**

**Tension Flange Yield Moment:**

\[ M_{nt} = R_p M_y = 206.5 \text{ kip in} \]

**Nominal Moment Strength:**

\[ M_n = \min(M_{nc1}, M_{nc2}, M_{nt}) = 1853 \text{ kip in} \]

**Shear Strength Calculations:**

**Web Depth assuming a 4'' hole:**

\[ D_w = D - 4 \text{ in} \]

**Shear Buckling Coefficient:**

\[ k = 5 \]

**Depth to Thickness Ratio Limits:**

\[ \lim_1 = 1.12 \frac{(E - k)}{F_y} = 60.31 \]

\[ \lim_2 = 1.40 \frac{(E - k)}{F_y} = 75.39 \]

**Depth to Thickness Ratio:**

\[ \text{web ratio} = \frac{D}{t_w} = 36.4 \]
Web Shear Coefficient:

\[ C_v = \begin{cases} \text{web ratio} < \text{lim}_1 \cdot 1.0, & \text{if web ratio} > \text{lim}_2, \end{cases} \]

\[ \left[ \frac{1.57}{(D/t_w)^2} \frac{(E:k)}{F_y} \right] \left[ \frac{1.12}{(D/t_w)^2} \frac{(E:k)}{F_y} \right] = 1 \]

AASHTO LRFD 6.10.9.3.2

Plastic Shear Force:
(with 4" hole)

\[ V_{p.h} = 0.58 \cdot F_y \cdot D_w \cdot t_w = 36.98 \text{ kip} \]

AASHTO LRFD 6.10.9.2-2

Plastic Shear Force:
(without 4" hole)

\[ V_{p.nh} = 0.58 \cdot F_y \cdot D_w \cdot t_w = 65.98 \text{ kip} \]

Nominal Shear Strength:
(with 4" hole)

\[ V_{n.h} = C_v \cdot V_{p.h} = 36.98 \text{ kip} \]

AASHTO LRFD 6.10.9.2-1

Nominal Shear Strength:
(with 4" hole)

\[ V_{n.nh} = C_v \cdot V_{p.nh} = 65.98 \text{ kip} \]
LRFD NONCOMPOSITE MEMBER CAPACITY

Structure: Milo Academy Bridge (Built-up Stringer Tee)
Location: Moment - Between Floorbeams, Shear - Near Floorbeam

Beam Dimensions:
Beam Depth: \(d := 6.75\) in

Flange Thickness:
Top Flange: \(t_{f,\text{top}} := 0.375\) in
Bottom Flange: \(t_{f,\text{bot}} := 0.375\) in

Flange Width: \(b_{f,\text{top}} := 30\) in, \(b_{f,\text{top,UB}} := 6\) in, \(b_{f,\text{bot}} := 6\) in

Web Depth:
\(D := d - t_{f,\text{top}} - t_{f,\text{bot}} := 6\) in

Depth of Web in Compression:
\(D_c := 1.405\) in

Distance Between Web Centerlines:
\(h := d - \frac{(t_{f,\text{top}} + t_{f,\text{bot}})}{2} = 6.375\) in

Web Width:
\(t_w := 0.75\) in

Unbraced Length:
\(L_b := 6\) ft

Beam Properties:
Steel Yield & Ultimate Strength:
\(F_y := 33\) ksi, \(F_u := 66\) ksi

Compression Flange Stress at Yield Onset:
\(F_{yr} := 0.7 \cdot F_y = 23.1\) ksi

Modulus of Elasticity:
\(E := 25000\) ksi

Moment of Inertia of Compression Flange:
\(I_{yc} := \frac{t_{f,\text{top}} (b_{f,\text{top}})^3}{12} = 843.75\) in\(^4\)

Moment of Inertia of Tension Flange:
\(I_{yt} := \frac{t_{f,\text{bot}} (b_{f,\text{bot}})^3}{12} = 6.75\) in\(^4\)

Major Axis Elastic Section Modulus:
\(S_{x,\text{top}} := 59.04\) in\(^3\), \(S_{x,\text{bot}} := 21.15\) in\(^3\)

Major Axis Elastic Section Modulus:
\(Z_x := 37.33\) in\(^3\)

Torsional Constant:
\(J_{\text{beam}} := 0.844\) in\(^4\)

Radius of Gyration:
\(r_t := \sqrt{\frac{\left[b_{f,\text{top}}^2 + \frac{D_c}{3} \cdot t_w \cdot b_{f,\text{top}} \cdot t_{f,\text{top}}\right]}{12 \left[1 + \frac{D_c}{3 b_{f,\text{top}} t_{f,\text{top}}} \right]}} = 8.53\) in
Strength Limit State Moment Capacity Calculations:

\[
CHECK_{WS} := \text{if} \left( \frac{(2\cdot D_c)}{t_w} < 5.7 \sqrt{\frac{E}{F_y}}, 1, 0 \right) = 1
\]

\[
CHECK_{FS} := \text{if} \left( \frac{f_{yc}}{f_{yt}} \geq 0.3, 1, 0 \right) = 1
\]

\[
\text{Capacity} := \text{if} \left( CHECK_{WS} = 1 \land CHECK_{FS} = 1, "Appendix A6 Capacity", "6.10.2 Capacity" \right) = "Appendix A6 Capacity"
\]

Web Plasticity Factor Calculations:

\[
\text{Beam Yield Moment:} \quad M_y := f_{y2} S_{x,bot} = 697.95 \text{ kip \cdot in}
\]

\[
\text{Beam Plastic Moment:} \quad M_p := f_{y2} Z_x = 1231.89 \text{ kip \cdot in}
\]

\[
\text{Hybrid Factor:} \quad \beta := \frac{(2 \cdot D_c - t_w)}{\left( f_{top} \cdot b_{top} \right)} = 0.187
\]

\[
\rho := 1
\]

\[
R_h := \left[ 12 + \beta \left( \frac{3 \cdot \rho - \rho^3}{(2 + 2 \cdot \beta)} \right) \right] = 1
\]

\[
\text{Depth of Web in Compression at Plastic Moment:} \quad D_{cp} := D = 6 \text{ in}
\]

\[
\text{Limiting Slenderness Ratio for Compact Web:} \quad \lambda_{pw1} := \frac{E}{\sqrt{\frac{F_y}{0.54 \left( \frac{M_p}{R_h \cdot M_y} \right)}}} = 34.346
\]

\[
\text{Limiting Slenderness Ratio for Noncompact Web:} \quad \lambda_{pw2} = \lambda_{TW} \left( \frac{D_p}{D_c} \right) = 721.592
\]

\[
\lambda_{pw} := \min(\lambda_{pw1}, \lambda_{pw2}) = 24.346
\]

\[
\lambda_w := \frac{(2 \cdot D_c)}{t_w} = 3.747
\]

Compact Check := if \((\lambda_{pw} \geq \lambda_w, "COMPACT"), "NONCOMPACT"\) = "COMPACT"
Web Plastification Factor:  \[ R_p = \text{if} \left( \frac{M_p}{M_y} \right) \min \left[ 1 - \left[ \frac{(R_b M_y)}{M_p} \frac{1}{M_y} \right], \frac{1}{\lambda_p}, \frac{1}{\lambda_{pw}} \right] \geq 1.765 \]  

Local Buckling Resistance:

Compression Flange Slenderness Ratio:  \[ \lambda_f = \frac{b_L \phi_{dp, \text{UB}}}{(f_{f, \text{top}})} = 16 \]  

Compact Flange Limiting Slenderness Ratio:  \[ \lambda_{lf} = 0.38 \frac{E}{F_y} = 11.265 \]  

Flange Local Buckling Coefficient:  \[ k_c = 0.76 = 0.76 \]  

Noncompact Flange Limiting Slenderness Ratio:  \[ \lambda_{lf} = 0.95 \frac{E k_c}{F_y} = 29.344 \]  

Lateral Torsional Buckling Resistance:

Limiting Lengths:

\[ L_f = 1.95 t_f \left( \frac{E}{F_y} \right) \left( \frac{I_{\text{beam}}}{S_{x, \text{bot}}} \right) \left( 1 + \sqrt{1 + 0.76 \left( \frac{F_{y, x, \text{bot}}}{E J_{\text{beam}}} \right)^2} \right) = 197.241 \text{ ft} \]  

\[ L_p = 1.0 t_f \left( \frac{E}{F_y} \right) = 21.068 \text{ ft} \]  

\[ L_b < L_p \quad M_{nc2a} := R_p M_y = 1231.89 \text{ kip in} \]  

\[ L_p < L_b < L_f \quad M_{nc2b} := \left[ 1 - \left( 1 - \frac{F_{y, x, \text{bot}}}{R_p M_y} \right) \left( \frac{L_p}{L_b} \right) \right] \left( R_p M_y - 1295.46 \text{ kip in} \right) \]  

\[ L_b > L_f \quad F_{cr} = \left( \frac{1 + 0.078 \left( J_{\text{beam}} / S_{x, \text{bot}} h \right) \left( \frac{L_b}{L_p} \right)^2}{1 + 0.078 \left( J_{\text{beam}} / S_{x, \text{bot}} h \right) \left( \frac{L_b}{L_p} \right)^2} \right) \]  

\[ M_{nc2c} := F_{cr} S_{x, \text{bot}} = 86393.43 \text{ kip in} \]  

AASHTO LRFD A6.3.2-4

AASHTO LRFD A6.3.2-3

AASHTO LRFD A6.3.2-4

AASHTO LRFD A6.3.2-6

AASHTO LRFD A6.3.2-5

AASHTO LRFD A6.3.3-5

AASHTO LRFD A6.3.3-4

AASHTO LRFD A6.3.1-1

AASHTO LRFD A6.3.3-2

AASHTO LRFD A6.3.3-8

AASHTO LRFD A6.3.3-3
Lateral Torsional Buckling Controlling Resistance:
\[ M_{nc2} := \text{if} \left( \frac{L_b}{L} \right. \left. < \frac{L_p}{L} \right. \left. ; M_{nc2a} \right. \left. ; \text{if} \left( \frac{L_b}{L} \right. \left. > \frac{L_p}{L} \right. \left. ; M_{nc2b} \right. \left. \right) = 1231.89 \text{kip-in} \]

Due to the double-T member configuration, the AASHTO LFFD local buckling checks do not apply. Furthermore, due to the compression flange’s slenderness, this section’s moment strength was restricted to its yield moment.

Nominal Moment Strength:
\[ M_n := M_y = 698 \text{ kip-in} \]

Shear Strength Calculations:

Shear Buckling Coefficient:
\[ k := 5 \quad \text{AASHTO LRFD 6.10.9.2} \]

Depth to Thickness Ratio Limits:
\[ \lim_{1} := 1.12 \sqrt{\frac{(E-k)}{F_y}} = 74.24 \quad \text{AASHTO LRFD 6.10.9.3.2.4} \]
\[ \lim_{2} := 1.40 \sqrt{\frac{(E-k)}{F_y}} = 92.8 \quad \text{AASHTO LRFD 6.10.9.3.2.5} \]

Depth to Thickness Ratio:
\[ \text{web}_{\text{ratio}} = \frac{D}{t_w} = 8 \quad \text{AASHTO LRFD 6.10.9.3.2} \]

Web Shear Coefficient:
\[ C_y := \begin{cases} \frac{1.57}{\left( \frac{D}{t_w} \right)^2} \sqrt{\frac{(E-k)}{F_y}} & \text{if } \text{web}_{\text{ratio}} < \lim_{1}, 1.0, \text{if } \text{web}_{\text{ratio}} > \lim_{2}, \left[ \frac{1.12}{\left( \frac{D}{t_w} \right)^2} \sqrt{\frac{(E-k)}{F_y}} \right] = 1 \end{cases} \quad \text{AASHTO LRFD 6.10.9.3.2} \]

Plastic Shear Force:
\[ V_p := 0.58 F_y D t_w = 86.13 \text{ kip} \quad \text{AASHTO LRFD 6.10.9.2-2} \]

Nominal Shear Strength:
\[ V_n := C_y V_p = 86.13 \text{ kip} \quad \text{AASHTO LRFD 6.10.9.2-1} \]
LRFD COLUMN CAPACITY
Structure: Milo Academy Bridge (Transom Trestle in Kingpost without Cover Plates)
Location: Near top of Kingpost

Beam Dimensions:
Beam Depth: \( d = 10 \text{ in} \)
Flange Thickness: \( t_f = 0.45 \text{ in} \)
Flange Width: \( b_f = 4.5 \text{ in} \)
Web Depth: \( D := d - 2t_f = 9.1 \text{ in} \)
Web Depth in Compression: \( D_c := \frac{D}{2} = 4.55 \text{ in} \)
Web Width: \( t_w = 0.25 \text{ in} \)
Unbraced Length (flexure): \( L_{b,\text{flex}} := 10.833 \text{ ft} \)
Unbraced Length (torsion): \( L_{b,\text{tor}} := 10.833 \text{ ft} \)
Effective Length Factor: \( K := 0.75 \)

Beam Properzies:
Steel Yield & Ultimate Strength: \( f_y := 50 \text{ ksi} \)
Modulus of Elasticity: \( E := 29000 \text{ ksi} \)
Shear Modulus: \( G := 12609 \text{ ksi} \)
Gross Area: \( A_g := 6.325 \text{ in}^2 \)
Moment of Inertia of Compression/Tension Flange: \( I_x := 108.1 \text{ in}^4 \quad I_y := 6.846 \text{ in}^4 \)
Warping Constant: \( C_w := 155.83 \text{ in}^6 \)
Torsional Constant: \( J_{\text{beam}} := 0.321 \text{ in}^4 \)
Radius of Gyration: 
\[
r_s := \sqrt{12 \left( \frac{1}{12} + \frac{1}{3} \frac{D-c}{b_f} \frac{t_w}{t_f} \right)} = 1.15 \text{ in}
\]
Elastic Flexural Buckling Resistance:

\[ p_{e,\text{flex}} = \left[ \frac{\pi^2 E}{\left( \frac{K L_{b,\text{flex}}}{R_s} \right)^2} \right] A_g = 270.7 \text{ kip} \]

AASHTO LRFD 6.9.4.1.2-1

Elastic Torsional Buckling and Flexural-Torsional Buckling Resistance:

\[ p_{e,\text{tor}} = \left[ \frac{\left( \pi^2 E C_w \right)}{\left( K L_{b,\text{flex}} \right)^2} + G I_{\text{beam}} \right] A_g \left( \frac{1}{I_x + I_y} \right) = 480.9 \text{ kip} \]

AASHTO LRFD 6.9.4.1.2-2

Minimum Elastic Buckling Resistance:

\[ p_e = \min(p_{e,\text{flex}}, p_{e,\text{tor}}) = 270.7 \text{ kip} \]

Slenderness Check:

Slender Check For Flanges:

Buckling Coefficient:

\[ k_e = \frac{4}{D} = 0.66 \]

\[ \text{Slender Limit}_1 = 0.64 \frac{k_e E}{F_y} = 12.35 \]

Slender Limit:

\[ \text{Slender Value}_1 = \frac{b_f}{t_f} = 10 \]

Slender Check:

\[ \text{Slender check}_1 = \text{if } (\text{Slender Value}_1 < \text{Slender Limit}_1, \text{"Nonslender"}, \text{"Slender"}) \]

\[ \text{Slender check}_1 = \text{"Nonslender"} \]
Slender Check For Webs:

Buckling Coefficient:

\[ K_{web} = 1.49 \]

Slender Limit:

\[ \text{Slender}_{\text{Limit2}} = K_{\text{web}} \frac{E}{F_y} = 35.88 \]

Slender Limit:

\[ \text{Slender}_{\text{Value2}} = \frac{D}{t_w} = 36.4 \]

Slender Check:

\[ \text{Slender}_{\text{Check2}} = \text{if} \left( \text{Slender}_{\text{Value2}} < \text{Slender}_{\text{Limit2}}, \text{"Nonslender"}, \text{"Slender"} \right) \]

\[ \text{Slender}_{\text{Check2}} = \text{"Slender"} \]

Slender Element Reduction Factor Check:

\[ \text{Slender}_{Q} = \text{if} \left( \text{Slender}_{\text{Check1}} = \text{"Nonslender"}, \text{if} \left( \text{Slender}_{\text{Check2}} = \text{"Nonslender"}, \text{"Q = 1.0"}, \text{"Slender"} \right), \text{"Slender"} \right) \]

\[ \text{Slender}_{Q} = \text{"Slender"} \]

Slender Element Reduction Factor Calculation (Based on slender section):

For Slender Web, the effective width is the following:

\[ b_k = 1.92 \, t_w \sqrt{\frac{E}{F_y}} \left( 1 - 0.34 \frac{D}{t_w} \sqrt{\frac{E}{F_y}} \right) = 8.96 \text{ in} \]

For Slender Web, the slender element reduction factor is the following:

\[ Q = \frac{b_k \, t_w}{D \, t_w} = 0.98 \]
**Equivalent Nominal Yield Strength:**

\[ p_0 := Q A_g f_y = 311.4 \text{ kip} \]

**Nominal Compressive Resistance:**

\[ p_n := \begin{cases} p_c & \text{if} \quad \frac{p_c}{p_0} < 0.44, \\ \left[ 0.658 \left( \frac{p_c}{p_0} \right) \right] p_0, 0.877 p_c & \text{if} \quad \frac{p_c}{p_0} \geq 0.44 \end{cases} = 192.4 \text{ kip} \]
LRFD COLUMN CAPACITY
Structure: Milo Academy Bridge
(Transom Trestle in Kingpost with Cover Plates)
Location: Near Top of Kingpost

**Beam Dimensions:**

- **Beam Depth:** \(d := 10.75\text{ in}\)
- **Flange Thickness (w/ cover plate):** \(t_f := 0.825\text{ in}\)
- **Flange Width (average w/ cover plate):** \(b_f := 4.27\text{ in}\)
- **Web Depth:** \(D := d - 2t_f = 9.1\text{ in}\)

**Web Depth in Compression:**
\(D_c := \frac{D}{2} = 4.55\text{ in}\)

**Web Width:**
\(t_w := 0.25\text{ in}\)

**Unbraced Length (flexure):**
\(L_b,\text{flex} := 10.833\text{ ft}\)

**Unbraced Length (torsion):**
\(L_b,\text{tor} := 10.833\text{ ft}\)

**Effective Length Factor:**
\(K := 0.75\)

**Beam Properties:**

- **Steel Yield & Ultimate Strength:** \(f_y := 50\text{ ksi}\)
- **Modulus of Elasticity:** \(E := 29000\text{ ksi}\)
- **Shear Modulus:** \(G := 12699\text{ ksi}\)
- **Gross Area:** \(A_g := 9.325\text{ in}^2\)
- **Moment of Inertia of Compression/Tension Flange:**
  \(I_z := 188.88\text{ in}^4\)
  \(I_y := 10.846\text{ in}^4\)
- **Warping Constant:** \(C_w := 263.47\text{ in}^6\)
- **Torsional Constant:** \(J_{beam} := 0.461\text{ in}^4\)
- **Radius of Gyration:**
  \(r_z := \sqrt{\frac{t_f}{12} + \frac{1}{3} \frac{D_c t_w}{b_f t_f}} = 1.17\text{ in}\)
Elastic Flexural Buckling Resistance:

\[ P_{e,\text{flex}} = \left[ \frac{\pi^2 E}{(K L_{0,\text{flex}})^2} \right] A_g = 385.2 \text{ kip} \]

AASHTO LRFD 6.9.4.1.2.1

Elastic Torsional Buckling and Flexural-Torsional Buckling Resistance:

\[ P_{e,\text{tor}} = \left[ \frac{\left( \frac{\pi^2 E C_w}{(K L_{\text{h,flex}})^2} + G J_{\text{beam}} \right)}{I_x + I_y} \right] A_g = 641.8 \text{ kip} \]

AASHTO LRFD 6.9.4.1.2.2

Minimum Elastic Buckling Resistance:

\[ P_e = \min(P_{e,\text{flex}}, P_{e,\text{tor}}) = 385.2 \text{ kip} \]

Slenderness Check:

Slender Check For Flanges:

Buckling Coefficient:

\[ k_e = \frac{4}{1} = 0.66 \]

Slender Limit:

\[ \text{Slender}_{\text{limit}} = 0.64 \sqrt{\frac{k_e E}{F_y}} = 12.55 \]

Slender Limit:

\[ \text{Slender}_{\text{value}} = \frac{b_t}{t_f} = 5.18 \]

Slender Check:

\[ \text{Slender}_{\text{check1}} = \text{if} \left( \text{Slender}_{\text{value}} < \text{Slender}_{\text{limit}}, \text{"Nonlender"}, \text{"Slender"} \right) \]

\[ \text{Slender}_{\text{check1}} = \text{"Nonlender"} \]
Slender Check For Webs:

**Buckling Coefficient:**

\[ K_{\text{web}} = 1.49 \]

**Slender Limit:**

\[ \text{Slender Limit} = K_{\text{web}} \sqrt{\frac{E}{F_y}} = 35.88 \]

**Slender Limit:**

\[ \text{Slender Value} = \frac{D}{t_w} = 36.4 \]

**Slender Check:**

\[ \text{Slender Check} = \text{if} (\text{Slender Value} < \text{Slender Limit}, "\text{Non-slimmer}", "\text{Slender}") \]

\[ \text{Slender Check} = "\text{Slender}" \]

**Slender Element Reduction Factor Check:**

\[ \text{Slender Q} = \text{if} (\text{Slender Check} = "\text{Non-slimmer}", \text{if} (\text{Slender Check} = "\text{Non-slimmer}", Q = 1.0", "\text{Slender}")", "\text{Slender}") \]

\[ \text{Slender Q} = "\text{Slender}" \]

**Slender Element Reduction Factor Calculation (Based on slender section):**

*For Slender Web, the effective width is the following:*

\[ b_c = 1.92 t_w \sqrt{\frac{E}{F_y}} \left(1 - 0.34 \frac{D}{t_w} \sqrt{\frac{E}{F_y}}\right) = 8.96 \text{ in} \]

*For Slender Web, the slender element reduction factor is the following:*

\[ Q = \frac{(b_c t_w)}{(D t_w)} = 0.98 \]
Equivalent Nominal Yield Strength:

\[ P_d = Q A_g f_y = 459.1 \text{kip} \]

Nominal Compressive Resistance:

\[ P_n = \begin{cases} \frac{P_o}{0.44} & \text{if } \frac{P_o}{P_d} > 0.658, \\ P_o & \text{if } 0.44 \leq \frac{P_o}{P_d} \leq 0.658, \\ 0.877 P_s & \text{if } \frac{P_o}{P_d} < 0.44 \end{cases} \]

\[ = 278.8 \text{ kip} \]
LRFD COLUMN CAPACITY
Structure: Milo Academy Bridge
(Transom Trestle in Kingpost with Cover Plates & Extra Bottom Plate)
Location: Near the bottom of the kingpost frame

Beam Dimensions:
Beam Depth: \( d = 11.12 \text{ in} \)
Flange Thickness (average large plate): \( t_f = 1.2 \text{ in} \)
Flange Width (average large plate): \( b_f = 7.1 \text{ in} \)
Web Depth: \( D = 9.1 \text{ in} \)

Web Depth in Compression: \( D_c = \frac{D}{2} = 4.55 \text{ in} \)
Web Width: \( t_w = 0.25 \text{ in} \)
Unbraced Length (flexure): \( L_{b,\text{flex}} = 23 \text{ ft} \)
Unbraced Length (torsion): \( L_{b,\text{tort}} = 4 \text{ ft} \)
Effective Length Factor: \( K = 0.75 \)

Beam Properties:
Steel Yield & Ultimate Strength: \( f_y = 33 \text{ ksi} \)
Modulus of Elasticity: \( E = 29000 \text{ ksi} \)
Shear Modulus: \( G = 12609 \text{ ksi} \)
Gross Area: \( A_g = 13.825 \text{ in}^2 \)
Moment of Inertia of Compression/Tension Flange:
\( I_x = 282.84 \text{ in}^4 \)
\( I_y = 64.85 \text{ in}^4 \)
Warping Constant:
\( C_w = 1059.1 \text{ in}^6 \)
Torsional Constant:
\( I_{\text{beam}} = 0.672 \text{ in}^4 \)
Radius of Gyration:
\( r_s = \sqrt{\frac{I_y}{A_g}} = 2.17 \text{ in} \)
Elastic Flexural Buckling Resistance:

\[ p_{e,\text{flex}} = \frac{\pi^2 E}{\left(\frac{K_{b,\text{flex}}}{L_s}\right)^2} A_g = 433.2 \text{ kip} \]

AASHTO LRFD 6.9.4.1.2.1

Elastic Torsional Buckling and Flexural-Torsional Buckling Resistance:

\[ p_{e,\text{tor}} = \frac{\left(\frac{\pi^2 E \cdot C_w}{K_{b,\text{flex}}^2}\right)}{G J_{\text{beam}} \left(\frac{A_g}{(I_k + I_y)}\right)} = 618.2 \text{ kip} \]

AASHTO LRFD 6.9.4.1.2.2

Minimum Elastic Buckling Resistance:

\[ p_e = \min(p_{e,\text{flex}}, p_{e,\text{tor}}) = 433.2 \text{ kip} \]

Slenderness Check:

Slender Check For Flanges:

Buckling Coefficient:

\[ k_c = \frac{4}{\sqrt{t_w}} = 0.66 \]

Slender Limit:

\[ \text{Slender Limit}_{1} = \frac{k_c E}{\sqrt{F_y}} = 15.45 \]

Slender Limit:

\[ \text{Slender Value}_{1} = \frac{b_f}{t_f} = 5.83 \]

Slender Check:

\[ \text{Slender check}_{1} = \text{if}(\text{Slender Value}_{1} < \text{Slender Limit}_{1}, "\text{Nonslender}", "\text{Slender}") \]

\[ \text{Slender check}_{1} = "\text{Nonslender}" \]
Slender Check For Webs:

Buckling Coefficient:

\[ K_{web} = 1.49 \]

Slender Limit:

\[ \text{Slender}_{\text{Limit}2} = K_{web} \sqrt{\frac{E}{F_y}} = 44.17 \]

Slender Limit:

\[ \text{Slender}_{\text{Value}2} = \frac{D}{t_w} = 36.4 \]

Slender Check:

\[ \text{Slender}_{\text{check}2} = \text{if} \left( \text{Slender}_{\text{Value}2} < \text{Slender}_{\text{Limit}2}, \text{"Nonslender"}, \text{"Slender"} \right) \]

\[ \text{Slender}_{\text{check}2} = \text{"Nonslender"} \]

Slender Element Reduction Factor Check:

\[ \text{Slender}_Q = \text{if} \left( \text{Slender}_{\text{check}1} = \text{"Nonslender"}, \text{if} \left( \text{Slender}_{\text{check}2} = \text{"Nonslender"}, \text{"Q = 1.0"}, \text{"Slender"} \right), \text{"Slender"} \right) \]

\[ \text{Slender}_Q = \text{"Q = 1.0"} \]

Slender Element Reduction Factor Calculation (Based on slender section):

For Slender Web, the effective width is the following:

\[ b_e = 1.92 t_w \sqrt{\frac{E}{F_y}} \left( 1 - 0.34 \frac{D}{t_w} \sqrt{\frac{E}{F_y}} \right) = 10.29 \text{in} \]

For Slender Web, the slender element reduction factor is the following:

\[ Q_{\text{slender}} = \frac{b_e}{D_{tw}} = 1.13 \]

\[ Q = \text{if} \left( \text{Slender}_Q = \text{"Q = 1.0"}, 1.0, Q_{\text{slender}} \right) = 1 \]
**Equivalent Nominal Yield Strength:**

\[ P_0 = Q \cdot A_k \cdot F_y = 456.2 \text{ kip} \]

**Nominal Compressive Resistance:**

\[
P_n = \begin{cases} 
\frac{P_e}{P_0} & \text{if } \frac{P_e}{P_0} > 0.44, \\
0.658 \left( \frac{P_0}{P_e} \right) & \text{if } P_0, 0.877 \cdot P_e 
\end{cases} = 293.6 \text{ kip}
\]
**LRFD NONCOMPOSITE GIRDER CAPACITY**

**Structure:** Milo Academy Bridge (Transom Trestle in Approach Spans)

**Location:** Moment - At Midspan (Unbraced), Shear - Near Support

**Beam Dimensions:**
- **Beam Depth:** $d = 10.8125$ in
- **Flange Thickness (w/ Cover Plate):** $t_f = 0.75$ in
- **Flange Width (Average w/ Cover Plate):** $b_f = 5.169$ in
- **Web Depth:** $D = d - t_f - t_e = 9.312$ in
- **Depth of Web in Compression:** $D_c = \frac{D}{2} = 4.6562$ in
- **Distance Between Web Centerlines:** $h = d - t_f = 10.0523$ in
- **Web Width:** $t_w = 0.25$ in
- **Unbraced Length:** $L_u = 20$ ft

**Beam Properties:**
- **Steel Yield & Ultimate Strength:** $F_y = 50$ ksi
- **Compression Flange Stress at Yield Onset:** $F_{y1} = 0.7F_y = 35$ ksi
- **Modulus of Elasticity:** $E = 29000$ ksi
- **Moment of Inertia of Compression/Tension Flange:**
  - $I_{yc} = \frac{t_f b_f^3}{12} = 8.63$ in$^4$
  - $I_{yt} = \frac{t_f (b_f)^2}{12} = 8.63$ in$^4$
- **Major Axis Elastic Section Modulus:** $S_G = 37.3$ in$^3$
- **Major Axis Elastic Section Modulus:** $Z_G = 40.71$ in$^3$
- **Torsional Constant:** $J_{beam} = 0.461$ in$^4$
- **Radius of Gyration:**
  $$ r_f = \sqrt{\frac{\frac{1}{12} \left( 1 + \frac{D_c t_w}{b_f t_f} \right)}{1 + \frac{D_c t_w}{b_f t_f} \left( 1 + \frac{3}{3} \frac{b_f}{t_f} \right)}} = 1.42 \text{ in} $$
Strength Limit State Moment Capacity Calculations:

CHECK_{WS} := \text{if} \left( \frac{2D_c}{t_w} < 5.7 \frac{F_y}{F_y} 1, 0 \right) = 1 \quad \text{AASHTO LRFD 6.10.6.2.3-1}

CHECK_{FS} := \text{if} \left( \frac{b_c}{t_y} \geq 0.3, 1, 0 \right) = 1 \quad \text{AASHTO LRFD 6.10.6.2.3-2}

Capacity := \text{if} (\text{CHECK}_{WS} = 1 \land \text{CHECK}_{FS} = 1, "Appendix A6 Capacity", "6.10.2 Capacity") = "Appendix A6 Capacity"

Web Plastification Factor Calculations:

Beam Yield Moment:
\[ M_y := F_y S_y = 1865 \text{ kip in} \]

Beam Plastic Moment:
\[ M_p := F_y Z_y = 2035.5 \text{ kip in} \]

Hybrid Factor:
\[ \beta := \frac{\left( \frac{2D_c}{t_w} \right)}{\left( \frac{t_f}{b_f} \right)} = 0.601 \quad \text{AASHTO LRFD 6.10.1.10.1-2} \]
\[ \rho = 1 \]
\[ R_h := \frac{12 + \beta \left( 3 - \rho - \rho^2 \right)}{12 + 2 - \beta} = 1 \quad \text{AASHTO LRFD 6.10.1.10.1-1} \]

Depth of Web in Compression at Plastic Moment:
\[ D_{cp} := \frac{D}{2} \left[ \left( \frac{F_y b_f t_f - F_y b_f t_f}{F_y D t_w} \right) + 1 \right] = 4.6562 \text{ in} \quad \text{AASHTO LRFD D6.3.2-1} \]

Limiting Slenderness Ratio for Compact Web:
\[ \lambda_{pw1} := \frac{\sqrt{\frac{E}{F_y}}}{0.54 \left( \frac{M_p}{F_y M_y} \right)} - 0.09 = 48.227 \quad \text{AASHTO LRFD A6.2.1-2} \]

Limiting Slenderness Ratio for Noncompact Web:
\[ \lambda_{TW} := 5.7 \sqrt{\frac{E}{F_y}} = 137.274 \quad \text{AASHTO LRFD A6.2.1-3} \]
\[ \lambda_{pw2} := \max \left( \lambda_{pw1}, \lambda_{pw2} \right) = 137.274 \quad \text{AASHTO LRFD A6.2.1-2} \]
\[ \lambda_{pw} := \text{min} \left( \lambda_{pw1}, \lambda_{pw2} \right) = 48.227 \quad \text{AASHTO LRFD A6.2.1-2} \]
\[ \lambda_{tw} := \frac{\left( \frac{2D_c}{t_w} \right)}{\left( \frac{t_f}{b_f} \right)} = 37.25 \quad \text{AASHTO LRFD A6.2.2-2} \]

Compact Check := \text{if} (\lambda_{pw} \geq \lambda_{tw}, "COMPACT", "NONCOMPACT") = "COMPACT" \quad \text{AASHTO LRFD A6.2.1-1}
Web Plastification Factor: AASHTO LRFD A6.2.2.4

\[ P_p := \begin{cases} \text{Compact Check} = \text{"COMPACT"} \left( \frac{M_p}{M_y} \right), & \min \left[ 1 - \left( 1 - \left( \frac{P_p}{M_p} \right) \right) \left( \frac{M_p}{M_y} \right) \right] = 1.091 \end{cases} \]

Local Buckling Resistance:

Compression Flange Slenderness Ratio:

\[ \lambda_f = \frac{b_f}{2t_f} = 3.446 \quad \text{AASHTO LRFD A6.3.2.3} \]

Compact Flange Limiting Slenderness Ratio:

\[ \lambda_{pf} = 0.38 \frac{E}{F_y} = 9.152 \quad \text{AASHTO LRFD A6.3.2.4} \]

Flange Local Buckling Coefficient:

\[ k_c := \max \left( \frac{4}{\sqrt{t_w}} \right) = 0.655 \quad \text{AASHTO LRFD A6.3.2.6} \]

Noncompact Flange Limiting Slenderness Ratio:

\[ \lambda_{tf} := 0.95 \frac{E}{F_{fy}} = 22.138 \quad \text{AASHTO LRFD A6.3.2.5} \]

Local Buckling Resistance:

\[ M_{ncl} := \max \left( \lambda_f \lambda_{pf} P_p M_y \left[ 1 - \left( \frac{M_p}{M_y} \right) \lambda_{pf} \right] P_p M_y \right) = 2055.5 \text{ kip in} \quad \text{AASHTO LRFD A6.3.2.1.2} \]

Lateral Torsional Buckling Resistance:

Limiting Lengths:

\[ L_t := 1.95 t_f \frac{E}{F_{fy}} \sqrt{\frac{E I}{E I_{beam}}} \sqrt{1 + 6.76 \left( \frac{(F_{fy} S_{xh})^2}{E I_{beam}} \right)} = 12.989 \text{ ft} \quad \text{AASHTO LRFD A6.3.3.5} \]

\[ L_p := 1.0 t_f \frac{E}{F_{fy}} = 2.855 \text{ ft} \quad \text{AASHTO LRFD A6.3.3.4} \]

\[ L_5 < L_p: \quad M_{nc2a} := P_p M_y = 2055.5 \text{ kip in} \quad \text{AASHTO LRFD A6.3.3.1} \]

\[ L_p < L_5 < L_t: \quad M_{nc2b} := 1 - \left[ 1 - \left( \frac{F_{fy} S_{xh}}{P_p M_y} \right) \left( \frac{L_p - L_5}{L_t - L_p} \right) \right] P_p M_y = 800.43 \text{ kip in} \quad \text{AASHTO LRFD A6.3.3.2} \]
Bridle, 2013
Calc’d By: BAC
Checked By: KNR

\[ L_b > L_y: \quad F_{cr} = \left[ \frac{\pi^2 E}{L_b} \right] ^{1/2} \left[ 1 + 0.078 \left( \frac{h}{s_x h} \right) \left( \frac{L}{t} \right) \right]^{1/2} = 19.41 \text{ kN} \quad \text{AASHTO LRFD A6.3.3.3} \]

\[ M_{nc2e} = F_{cr} S_x = 724.16 \text{ kip-in} \quad \text{AASHTO LRFD A6.3.3.3} \]

**Lateral Torsional Buckling Controlling Resistance:**
\[ M_{nc2} = \text{if}(L_b < I_p, M_{nc2a}, \text{if}(L_b > I_p, M_{nc2e}, M_{nc2b})) = 724.16 \text{ kip-in} \]

**Tension Flange Yielding Resistance:**
\[ Tension \text{ Flange Yield Moment:} \quad M_{lf} = P_y M_y = 2035.5 \text{ kip-in} \quad \text{AASHTO LRFD A6.4.1} \]

**Nominal Moment Strength:**
\[ M_n = \text{min}(M_{nc1}, M_{nc2}, M_{lf}) = 724.8 \text{ kip-in} \]

**Shear Strength Calculations:**
\[ \text{Web Depth assuming a 4'' hole:} \quad D_w = D - 4 \text{ in} \quad \text{AASHTO LRFD 6.10.9.2} \]
\[ \text{Shear Buckling Coefficient:} \quad k = 5 \quad \text{AASHTO LRFD 6.10.9.2} \]
\[ \text{Depth to Thickness Ratio Limits:} \]
\[ \lim_1 = 1.12 \sqrt{\frac{E (k)}{F_y}} = 60.31 \text{ AASHTO LRFD 6.10.9.3.2.4} \]
\[ \lim_2 = 1.40 \sqrt{\frac{E (k)}{F_y}} = 75.39 \text{ AASHTO LRFD 6.10.9.3.2.5} \]
\[ \text{Depth to Thickness Ratio:} \]
\[ \text{web ratio} = \frac{D}{t_w} = 37.25 \quad \text{AASHTO LRFD 6.10.9.3.2} \]

**Web Shear Coefficient:**
\[ C_v = \begin{cases} 1 & \text{if } \text{web ratio} < \lim_1 \text{ or } \lim_2 \text{ kip-in} \\ \frac{1.57 \left( \frac{D}{t_w} \right)^2 \sqrt{\frac{E (k)}{F_y}}}{1.2} & \text{if } \text{web ratio} > \lim_2 \end{cases} \quad \text{AASHTO LRFD 6.10.9.3.2} \]

**Plastic Shear Force:**
\[ V_p h = 0.38 F_y D_w t_w = 38.52 \text{ kip} \quad \text{AASHTO LRFD 6.10.9.2.2} \]
Plastic Shear Force: 
(without 4" hole)

\[ V_{pnh} = 0.58 \cdot E_y \cdot D \cdot t_w = 67.52 \text{ kip} \]

Nominal Shear Strength: 
(with 4" hole)

\[ V_{nh} = C_v \cdot V_{pnh} = 38.52 \text{ kip} \]

Nominal Shear Strength: 
(without 4" hole)

\[ V_{n nh} = C_i \cdot V_{p nh} = 67.52 \text{ kip} \]
LRFD NONCOMPOSITE GIRDER CAPACITY

Structure: Milo Academy Bridge (Transom Trestle in Approach Spans)
Location: Moment - At Midspan (Braced at Quarter Points), Shear - Near Support

Beam Dimensions:
- Beam Depth: \( d = 10.8125 \text{ in} \)
- Flange Thickness (w/ Cover Plate): \( t_f = 0.75 \text{ in} \)
- Flange Width (Average w/ Cover Plate): \( b_f = 5.169 \text{ in} \)
- Web Depth: \( D = d - t_f - t_f = 9.312 \text{ in} \)
- Depth of Web in Compression: \( D_c = \frac{D}{2} = 4.6562 \text{ in} \)
- Distance Between Web Centerlines: \( h = d - t_f = 10.0625 \text{ in} \)
- Web Width: \( t_w = 0.25 \text{ in} \)
- Unbraced Length: \( L_b = 5 \text{ ft} \)

Beam Properties:
- Steel Yield & Ultimate Strength: \( F_y = 50 \text{ ksi} \)
- Compression Flange Stress at Yield Onset: \( F_{yt} = 0.7F_y = 35 \text{ ksi} \)
- Modulus of Elasticity: \( E = 29000 \text{ ksi} \)
- Moment of Inertia of Compression/Tension Flange:
  - \( I_{yc} = \frac{(t_f b_f^2)}{12} = 8.63 \text{ in}^4 \)
  - \( I_{yt} = \frac{(t_f b_f^3)}{12} = 8.63 \text{ in}^4 \)

- Major Axis Elastic Section Modulus:
  - (Assuming 4" hole at center of web) \( S_X = 37.3 \text{ in}^3 \)
- Major Axis Elastic Section Modulus:
  - (Assuming 4" hole at center of web) \( Z_X = 40.71 \text{ in}^3 \)
- Torsional Constant:
  - \( J_{beam} = 0.461 \text{ in}^4 \)
- Radius of Gyration:
  - \( r_t = \sqrt{\frac{b_f}{D_c t_w b_f + t_f}} = 1.42 \text{ in} \)
Strength Limit State Moment Capacity Calculations:

\[
\text{CHECK}_{WS} = \begin{cases} 
\left( \frac{2}{D_c} \right) \leq 5.7, \sqrt{\frac{F}{F_y}} \leq 1, 0 \right) = 1 & \text{AASHTO LRFD 6.10.6.2.1-1} \\
\text{CHECK}_{PS} = \begin{cases} 
\frac{h_y}{t_w} \geq 0.3, 1, 0 \right) = 1 & \text{AASHTO LRFD 6.10.6.2.3-2} 
\end{cases}
\]

Capacity := if(\(\text{CHECK}_{WS} = 1 \land \text{CHECK}_{PS} = 1\), "Appendix A6 Capacity", "6.10.2 Capacity") = "Appendix A6 Capacity"

Web Plastification Factor Calculations:

Beam Yield Moment: 
\(M_y = F_y S_y = 1885\text{-kip-in}\)

Beam Plastic Moment: 
\(M_p = F_y Z_y = 2035.3\text{-kip-in}\)

Hybrid Factor: 
\(\beta = \left( \frac{2}{D_c t_w} \right) = 0.601 \quad \text{AASHTO LRFD 6.10.1.101-2} \)

\(\rho = 1 \quad \text{AASHTO LRFD 6.10.1.101-1} \)

\(R_b = \left( \frac{12 + \beta \cdot \left( 3 \cdot \rho - \rho^3 \right)}{12 + 2 \cdot \beta} \right) = 1 \quad \text{AASHTO LRFD 6.10.1.101-1} \)

Depth of Web in Compression at Plastic Moment:
\(D_{cp} = \left( \frac{D}{7} \right) \left[ \left( \frac{F_y b_f t_f - F_y b_f t_f}{F_y D_c t_w} \right) + 1 \right] = 4.6582\text{-in} \quad \text{AASHTO LRFD D6.3.2.1-1} \)

Limiting Slenderness Ratio for Compact Web:
\(\lambda_{pw1} := \frac{\sqrt{\frac{E}{F_y}}}{0.54 \left( \frac{M_p}{R_b M_y} \right) - 0.09} = 48.227 \quad \text{AASHTO LRFD A6.2.1-2} \)

Limiting Slenderness Ratio for Noncompact Web:
\(\lambda_{nw} := \frac{5.7}{\sqrt{\frac{F_y}{F_y}}} \quad \text{AASHTO LRFD A6.2.1-3} \)
\(\lambda_{pw2} := \lambda_{nw} \left( \frac{D_{cp}}{D_c} \right) = 137.274 \quad \text{AASHTO LRFD A6.2.1-2} \)
\(\lambda_{pw} := \min(\lambda_{pw1}, \lambda_{pw2}) = 48.227 \quad \text{AASHTO LRFD A6.2.1-2} \)
\(\lambda_{w} = \frac{\left( \frac{2}{D_c} \right)}{t_w} = 37.25 \quad \text{AASHTO LRFD A6.2.2.2} \)

CompactCheck = if(\(\lambda_{pw} > \lambda_{w}\), "COMPACT", "NONCOMPACT") = "COMPACT"  
\(\text{AASHTO LRFD A6.2.1-1} \)

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Web Plastification Factor: AASHTO LRFD A6.2.2-4
\[ P_p = \begin{cases} \text{CompactCheck = "COMPACT",} & \min \left[ \frac{M_p}{M_y} \right], \min \left[ 1 - \left( \frac{R_p M_y}{M_p} \right) \left( \frac{M_p}{M_y} \right) \right] = 0.90 \end{cases} \]

**Local Buckling Resistance:**

**Compression Flange Slenderness Ratio:**
\[ \lambda_f = \frac{b_f}{2 t_f} = 3.446 \quad \text{AASHTO LRFD A6.3.2-3} \]

**Compact Flange Limiting Slenderness Ratio:**
\[ \lambda_{pf} = 0.38 \sqrt{\frac{F_y}{F_{yr}}} = 9.152 \quad \text{AASHTO LRFD A6.3.2-4} \]

**Flange Local Buckling Coefficient:**
\[ k_f = \max \left( \frac{0.35}{\sqrt{\frac{D}{w}}} \right) = 0.655 \quad \text{AASHTO LRFD A6.3.2-6} \]

**Noncompact Flange Limiting Slenderness Ratio:**
\[ \lambda_{nf} = 0.95 \sqrt{\frac{F_{yr} k_f}{F_y}} = 22.138 \quad \text{AASHTO LRFD A6.3.2-5} \]

**Local Buckling Resistance:**
\[ M_{nc1} = 1 \left[ \lambda_f < \lambda_{pf}, \frac{R_p M_y}{M_p} \left( 1 - \left( \frac{F_y S_d}{R_p M_y} \right) \left( \lambda_f - \lambda_{pf} \right) \right) \right] R_p M_y = 2035.3 \text{-kip in} \quad \text{AASHTO LRFD A6.3.2-1.2} \]

**Lateral Torsional Buckling Resistance:**

**Limiting Lengths:**
\[ L_f = 1.95 \sqrt{\frac{E}{F_{yr}}} \sqrt{\frac{1}{1 + 6.76 \left( \frac{F_{yr} S_d b_f}{E I_{beam}} \right)^{\frac{3}{2}}} = 12.989 \text{ ft} \quad \text{AASHTO LRFD A6.3.3-5} \]
\[ L_p = 1.04 \sqrt{\frac{E}{F_y}} = 2.855 \text{ ft} \quad \text{AASHTO LRFD A6.3.3-4} \]

\[ L_{b} < L_p: \quad M_{nc2a} = R_p M_y = 2035.5 \text{ kip in} \quad \text{AASHTO LRFD A6.3.3-1} \]

\[ L_p < L_b < L_f: \quad M_{nc2b} = \begin{cases} 1 - \left[ 1 - \left( \frac{F_y S_d}{R_p M_y} \right) \left( \frac{b - L_p}{L_f - L_p} \right) \right] R_p M_y = 1880.99 \text{ kip in} \end{cases} \quad \text{AASHTO LRFD A6.3.3-2} \]
\[ L_d > L_f : \quad F_{cr} = \frac{\left( \frac{n - \frac{E}{L}}{n} \right)^2}{1 + 0.078 \left( \frac{I_{bem}}{S_{x,h}} \right) \left( \frac{I_b}{h_1} \right)} = 174.09 \text{kN} \quad \text{AASHTO LRFD A6.3.3-8} \]

\[ M_{nc2} = F_{cr} S_{y} = 6493.46 \text{kip in} \quad \text{AASHTO LRFD A6.3.3-3} \]

**Lateral Torsional Buckling Controlling Resistance:**
\[ M_{nc2} = \text{if} (L_b < L_f, M_{nc2}, \text{if} (L_b > L_f, M_{nc1}, M_{nc2})) = 1880.99 \text{kip in} \]

**Tension Flange Yielding Resistance:**
\[ M_{n1} = P_{pf} M_{y} = 2035.5 \text{kip in} \quad \text{AASHTO LRFD A6.4-1} \]

**Nominal Moment Strength:**
\[ M_{n} = \min(M_{nc1}, M_{nc2}, M_{n1}) = 1881 \text{kip in} \]

**Shear Strength Calculations:**

**Web Depth assuming a 4\text{"} hole:**
\[ D_w = D - 4 \text{ in} \quad \text{AASHTO LRFD 6.10.9.2} \]

**Shear Buckling Coefficient:**
\[ k = \frac{5}{9} \quad \text{AASHTO LRFD 6.10.9.2} \]

**Depth to Thickness Ratio Limits:**
\[ \lim_1 = 1.12 \sqrt{\frac{E \times k}{F_y}} = 60.31 \quad \text{AASHTO LRFD 6.10.9.3.2-4} \]
\[ \lim_2 = 1.40 \sqrt{\frac{E \times k}{F_y}} = 75.39 \quad \text{AASHTO LRFD 6.10.9.3.2-5} \]

**Depth to Thickness Ratio:**
\[ \text{web ratio} = \frac{D}{t_w} = 37.25 \quad \text{AASHTO LRFD 6.10.9.3.2} \]

**Web Shear Coefficient:**
\[ C_y = \begin{cases} \text{if} \text{web ratio} < \lim_1, 1.0, \text{if} \text{web ratio} > \lim_2, \left[ \frac{1.57 (E_k)}{D \left( \frac{t_w}{D} \right)^2 F_y} \right] \left[ \frac{1.12 (E_k)}{D \left( \frac{t_w}{D} \right)^2 F_y} \right] = 1 \quad \text{AASHTO LRFD 6.10.9.3.2} \end{cases} \]

**Plastic Shear Force:**
\[ V_{p,h} = 0.58 F_y D_w t_w = 38.52 \text{ kip} \quad \text{AASHTO LRFD 6.10.9.2-2} \]
<table>
<thead>
<tr>
<th>Plastic Shear Force: (without 4&quot; hole)</th>
<th>( V_{p,nh} = 0.58 F_y D t_w = 67.52 \text{ kip} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nominal Shear Strength: (with 4&quot; hole)</td>
<td>( V_{n,h} = C_v V_{p,h} = 38.52 \text{ kip} )</td>
</tr>
<tr>
<td>Nominal Shear Strength: (without 4&quot; hole)</td>
<td>( V_{n,nh} = C_v V_{p,nh} = 67.52 \text{ kip} )</td>
</tr>
</tbody>
</table>
### TABLE 1. Breaking strength of turnbuckles (complete with end pulls)

<table>
<thead>
<tr>
<th>Size, nominal outside diameter of thread</th>
<th>Strength breaking, minimum</th>
<th>Recommended working loads</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Jaw, eye, or stub end pulls</td>
<td>Hook, eye, or stub end pulls</td>
</tr>
<tr>
<td></td>
<td>Pounds</td>
<td>Pounds</td>
</tr>
<tr>
<td>1/4</td>
<td>2,500</td>
<td>1,500</td>
</tr>
<tr>
<td>5/32</td>
<td>3,500</td>
<td>2,500</td>
</tr>
<tr>
<td>3/32</td>
<td>5,200</td>
<td>3,500</td>
</tr>
<tr>
<td>1/8</td>
<td>9,000</td>
<td>5,500</td>
</tr>
<tr>
<td>5/32</td>
<td>13,500</td>
<td>8,000</td>
</tr>
<tr>
<td>3/16</td>
<td>20,000</td>
<td>10,000</td>
</tr>
<tr>
<td>1/4</td>
<td>20,000</td>
<td>12,000</td>
</tr>
<tr>
<td>1/2</td>
<td>38,000</td>
<td>14,500</td>
</tr>
<tr>
<td>9/32</td>
<td>60,000</td>
<td>23,000</td>
</tr>
<tr>
<td>5/16</td>
<td>72,000</td>
<td>29,000</td>
</tr>
<tr>
<td>1/2</td>
<td>85,000</td>
<td>36,000</td>
</tr>
</tbody>
</table>

3.2 Construction.

3.2.1 **Forged.** Each forged turnbuckle body and each forged end pull shall be forged at elevated temperature to final shape and size.

3.2.2 **Spread.** For each spread turnbuckle body, one piece of material shall be cut lengthwise from near one end to the other end by any suitable means, such as an oxy-acetylene cutting torch; the resulting reams then shall be spread apart at elevated temperatures to final shape and size.

3.2.3 **Resistance welded.** Each resistance-welded turnbuckle body shall be fabricated by joining two formed pieces of material by either the flash or upset welding process. The welds shall be parallel to the long axis of the piece. The surfaces to be joined shall be held in intimate contact by external forces, an electric current passed through the surfaces, and the weld consolidated by the forces.

3.2.4 **Arc or gas welded.** The welds shall be either electric arc or oxy-acetylene welds at the option of the contractor.

3.2.4.1 All arc-welded bodies, eyes, and jaws shall be welded with type MIL.7015, MIL.7016, MIL.8015 or MIL.8016 electrodes of MIL.E.18035.

3.2.4.2 All gas-welded bodies, eyes, and jaws shall be welded with class 1, type A welding rods of MIL.R.903.

3.3 **Size.** Turnbuckles covered by this specification shall be furnished in the sizes shown in tables II and III, as specified (see 6.1). The size of turnbuckle bodies and turnbuckles shall be the nominal major diameter of the threads in the heads and the clear opening between heads (which is approximately equal to the take up); thus, for a ¾ by 6-inch turnbuckle body, the heads shall be threaded for a ¾-inch nominal major-diameter end pull, and the clear opening between heads shall be 6 inches. The difference
APPENDIX B - REFERENCES

For all references to BDI’s equipment, services, and analysis/rating methods please go to our website:
www.bridgetest.com
