MEMORANDUM

DATE: October 26, 2015
TO: Mayor and City Council
FROM: Phil Jones, City Administrator
RE: 3rd Street SE Bridge Status and Timeline

Following up on several questions and comments received over the past week, this memo shares some of the history, actions, and findings on the bridge. During this time, the primary concern of the City has been, and continues to be the safety of the public as it relates to the use of the bridge.

On February 18, 2015, WHKS bridge inspectors and engineers, acting on behalf of the City of Waverly as the City-designated Bridge Program Manager as required by the Iowa Department of Transportation, informed the City that the 3rd Street SE Bridge was in unsafe condition and needed to be closed immediately for both vehicle and pedestrian traffic.

In an effort to gain a better understanding of the extent of the issues, verify the report with an independent third party, and understand the scope necessary for rehabilitation or replacement, VJ Engineering of Coralville, Iowa was retained by the City Council on March 16, 2015 for a not-to-exceed amount of $30,000. Motion was made by Neuendorf, second by Lampe, and the vote was 6-0 with Gade absent. The project had two parts, and, as stated in the feasibility study, "the main goal of this project is to determine the extent of the rehabilitation repairs necessary to put the bridge back into service, either for vehicular or pedestrian use, and to provide the City with a cost estimate for the repairs for the purpose of applying for project funding."

The project was given 120 days for both phases and the report delivered to Council on July 22nd, and a presentation given by Tim McDermott at the July 27, 2015 Study Session. Phase 1 included the assessment of the structure and review of records on the bridge. This phase also included the structural analysis on pedestrian live loads and vehicular live loads. Phase 2 was the cost estimates for the construction of the repair work, methods and schedule for pedestrian and vehicle use, and a lifecycle cost assessment on a 20 year design life and future maintenance costs. Five options for action were presented, along with a "do nothing" option. The options were:

1. Do nothing.
2. Rehabilitate the existing bridge for pedestrian only use.
3. Rehabilitate the existing bridge for vehicular and pedestrian only use.
4. Replace existing bridge with new 3 span, pre-engineered steel pony truss bridge with timber deck for pedestrian only use.
5. Replace the bridge with a 3 span steel truss bridge that replicates the geometry for vehicle and pedestrian use.
6. Replace the existing bridge with a new 3 span concrete beam bridge for vehicle and pedestrian use.

On July 27 at the Study Session, Tim McDermott presented the findings of the report that were in agreement with the WHKS Engineering report from February 18 recommending closing the bridge to vehicle traffic. The presentation agreed with the advanced state of deterioration and reasoning for closing the bridge, and presented estimates for rehabilitating the bridge or replacing the bridge.

One point of difference was the sidewalk condition and level of service. Tim McDermott deemed the sidewalk useable on a short-term basis until a final decision was made on the rehabilitation, replacement, or retirement of the bridge. The staff and I made a mistake by opening the sidewalk based on the presentation, without consulting our bridge program manager first. Our decision was influenced by popular opinion, political discussions, and our own desire to quickly find a way to allow pedestrians to use the bridge. Because we wanted to see some portion of the bridge return to service as soon as possible, the decision was made without that critical conversation with WHKS. This would later influence our actions following Mr. Kehe's opinion that the bridge was safe to reopen to vehicles and pedestrians. The sidewalk opening and then closing decision by staff has resulted in additional confusion and frustration for the City Council and the citizens of Waverly, and we apologize for that. That was not our intent, but an unintended consequence of our rush to conclusions.

At the same meeting on July 27, 2015, Butch Kehe, professional engineer and principle of Cedar Valley Engineering, responded to the presentation by raising points for consideration based on his experience with the bridge. He inspected the bridge in the past, and performed work on the bridge including the sidewalk rehabilitation in 2006, and deck modification and repair work in 1983. Mr. Kehe inspected the bridge in 1997. At about that time the Iowa DOT instituted new requirements for program managers of bridges, and the city transitioned to WHKS & Co. as the designated bridge program manager.

On July 28, 2015 Mr. Kehe submitted a letter to me that I shared with the City Council. This letter included his recommendations for repair of the bridge, rather than rehabilitation, and offered an estimate of cost for repairs based on the VJ report of $288,000 and then offered a plan of repairing the bridge, then extending the Cedar River Parkway over the river to Highway 3, and to consider extending 8th Street SE north across the flow-way and river.

Mr. Kehe shared his ideas from the letter at the August 17, 2015 council meeting. Following that meeting, Mr. Kehe then presented his ideas for repair and improvements at the August 24, 2015 study session. The Council at that time wanted to review Mr. Kehe's report before making any decisions on moving forward.

At the September 14, 2015 meeting, I gave an update that Mr. Kehe would present his findings from an assessment of the bridge in October. An agreement from Mr. Kehe for professional services was received on September 21, 2015 and added to the September 28, 2015 agenda for Council approval. The agreement for engineering inspections and a repair assessment for $10,050 was moved by Reznicek with second by Gade, passing 5-0 with McKenzie and Kangas absent.

Mr. Kehe then returned at the October 5, 2015 meeting and presented his findings, handing out his report at the meeting. The major focus was his opinion that the bridge could be opened immediately to vehicle and pedestrian traffic and that minor repairs could be made to the bridge in the future to extend the life of the bridge up to 5 years. Mr. Kehe also raised some questions about the calculations on the bridge done by VJ engineering and said in his opinion the bridge was safe enough to handle traffic.

Although there were some requests that the bridge be opened immediately, the stark contrast of the findings on condition caused me to pause. I informed the Council that I would not direct staff to open the bridge until the discrepancies in information could be resolved, as there were now two engineering firms saying that the bridge was unsafe for use and one firm saying it was safe, and at the time asked the Council to schedule a vote to open the bridge so that some official action of the body could be recorded in the minutes providing direction because of the difference of opinions in engineering firms.

On Tuesday, October 6, 2015 I reached out to Tim McDermott of VJ Engineering, sharing the information and calculations from Mr. Kehe so he could review and compare with his calculations and asked him to provide information back to the Council. During the week of October 12, Bill Werger researched the Iowa DOT procedures for bridge management, inspections, bridge closing, and bridge opening including who had the authority to open and close bridges and the requirements for doing so. His findings were shared with the City Council on October 19, 2015.

In general:

- Bridge inspections are required by federal law in accordance with the National Bridge Inspection Standards. According to Iowa Code, counties and cities are responsible for the safety, inspection, and evaluation of all highway bridges under their jurisdiction located on public roads.
- The bridge owner is required to have a program manager who is assigned these responsibilities, including the periodic inspection of the bridges under the bridge owner's control. The Iowa DOT has developed criteria for individuals performing the inspections that meet the qualifications required by federal law.
- A person who meets those criteria is certified by the Iowa DOT to perform bridge inspections and to maintain relevant information about those bridges in the state database (SIMMS) inventory of all bridges subject to NBIS.
- Many Bridge Owners retain a consultant to perform the duties of Program Manager. The City of Waverly has retained WHKS to perform these duties for all of its bridges for the last 16 years.
- WHKS has a long history of providing this service to public agencies and for the Iowa DOT
- Casey Faber of WHKS & Co. is listed by the City as the delegated bridge inspection program manager for the City of Waverly. Mr. Faber has met all of the requirements of the National Bridge Inspection Standards (NBIS) to be qualified as a bridge inspection program manager and has been approved by the Iowa DOT
- Mr. Faber is responsible for the bridges in the city of Waverly and should be the final authority on their condition and level of safety for the traveling public.

- After his inspection of the bridge, Casey Faber of WHKS filed a Critical Finding Bridge Report in SIIMS and notified the City of Waverly to close the bridge. The status of the bridge in SIIMS was changed to “closed”.
- The Iowa DOT is the ultimate program manager for all bridges in the State of Iowa based upon the federal statue and if a bridge is opened by a Bridge Owner contrary to the designation of the bridge as closed by the program manager, the DOT has the right to close the bridge again.

Now, Mr. Faber of WHKS and Mr. McDermott of VJ Engineering are working on the status of the sidewalk and bridge and Mr. Faber will provide the Council with a field demonstration of the bridge on November 2 with a presentation at the council meeting that evening.

Mr. McDermott has reviewed Mr. Kehe’s information and his response and findings are below. Mr. Faber has provided additional information and photos of the condition of critical components of the bridge, and his information is also below.

As an illustration of the scope of the issue, the bridge has 24 sidewalk support brackets, 12 bearing pins, 189 stingers with 378 connection points supporting the deck, and if there is one bad stringer, stringer connection, or bearing it means the closing of the bridge. Similarly, one bad sidewalk bracket means the closing of the sidewalk. We now must determine the best path forward for repairing or rehabilitating the bridge in the most feasible, practical, and politically acceptable manner based on information from the City’s bridge program manager, and will be able to continue that discussion on November 2nd.

As a result of all of this, I can say that we have learned that if there are questions on which route to choose or which decision to make on unresolved issues, we will always error in the interest of public safety until we can sort out the information to make the best decision possible

Phil Jones

From: Casey Faber <CFaber@Whks.com>
Sent: Thursday, October 22, 2015 3:33 PM
To: Mike Cherry; W D. Werger
Cc: Phil Jones; Fouad Daoud; Scott Sweet
Subject: RE: 3rd Street Bridge
Attachments: 3rd Street Bridge.pdf

Mike,

As requested, please see the attached report discussing in more details the deficiencies we found in February and why they warranted closure of the bridge.

Also, we would be available to attend the City Council meeting to answer any questions you or other City officials may have, and could even visit the site with the Council and City staff to see the deficiencies up close.

Please let us know if you have any further questions regarding this matter.

Thank you,

Casey V. Faber, P.E.
1421 South Bell, Suite 103 I Ames, IA 50010
Voice: 515.663.9997 I www.whks.com
whks

From: Mike Cherry [mailto:mike@ci.waverly.ia.us]
Sent: Wednesday, October 21, 2015 9:54 AM
To: Casey Faber; W D. Werger
Cc: Phil Jones; Fouad Daoud; Scott Sweet
Subject: RE: 3rd Street Bridge

Casey,

Attached are Mr. Kehe's calculations for the 2006 sidewalk improvements and photos I took this morning of the as-built condition (w=5.8' & h=3")

Mike Cherry
Waverly City Engineer
319-352-9065

From: Casey Faber [mailto:CFaber@Whks.com]
Sent: Tuesday, October 20, 2015 4:56 PM
To: Mike Cherry; W D. Werger

Cc: Phil Jones; Fouad Daoud; Scott Sweet
Subject: RE: 3rd Street Bridge

Hi Mike,

We were not aware the sidewalk is lightweight concrete and our loading does not take that into consideration. We were not able to locate plans of the 2006 improvements. Do you have design plans of the 2006 improvements available so that our analysis could be refined?

Thank you very much,

Casey V. Faber, P.E.
1421 South Bell, Suite 103 I Ames, IA 50010
Voice: 515.663.9997 I www.whks.com

From: Mike Cherry [mailto:mike@ci.waverly.ia.us]
Sent: Tuesday, October 20, 2015 4:01 PM
To: Casey Faber; W D. Werger
Cc: Phil Jones; Fouad Daoud; Scott Sweet
Subject: RE: 3rd Street Bridge

Hi Casey,

Do your calculation utilize the original 1917 sidewalk design or the 2006 sidewalk improvements that consist of a light weight concrete on a corrugated steel pan. Mr. Butch Kehe designed the 2006 sidewalk and said it is about 20% lighter than the original 1917 sidewalk.

Mike Cherry
Waverly City Engineer
319-352-9065

From: Casey Faber [mailto:CFaber@Whks.com]
Sent: Tuesday, October 20, 2015 3:34 PM
To: W D. Werger
Cc: Mike Cherry; Phil Jones; Fouad Daoud; Scott Sweet
Subject: RE: 3rd Street Bridge

Hi William,

First, thank you for contacting us regarding the discussion with the Council and continuing to include WHKS in the discussion of this bridge.

We have reviewed the report by VJ Engineering and it is our opinion that the analysis did not adequately consider the degree of deterioration throughout the structure. We performed an alternate analysis that shows the sidewalk overhang brackets do not have the capacity for the full AASHTO design pedestrian load.

I'm not sure of your familiarity with SIIMS, but as a quick note it provides the owner and inspector a central place to record and manage data concerning structures. Some of this data is required to be reported to the federal government, and is organized on the Structural Inventory and Appraisal (SI&A) form. The items on the SI&A are numbered and describe details about the bridge and it's condition.

On the SI&A Item 42 is coded 5 for the 3rd Street bridge which means it is a vehicle and pedestrian structure. When the bridge was closed in February, Item 41 was changed from P (open, but posted for load) to K which means it is closed to all traffic. Based on our analysis that the structure cannot support legal pedestrian loads and the strict wording of the SI&A items we feel it is in the best interest of the City to keep the bridge closed to all traffic.

Please let us know if you have any questions regarding this information. We appreciate the opportunity to serve you!

Casey V. Faber, P.E.
1421 South Bell, Suite 103 I Ames, IA 50010
Voice: 515.663.9997 I www.whks.com

From: W D. Werger [mailto:wdwerger@ci.waverly.ia.us]
Sent: Tuesday, October 20, 2015 1:11 PM
To: Casey Faber
Cc: Mike Cherry; Phil Jones
Subject: 3rd Street Bridge

Casey: We told the Council last night that the bridge would remain closed until you determined that it could be opened. Based upon an opinion from VJ Engineering, we did in fact open the walkway to pedestrian traffic some time ago. After my explanation of how a bridge is closed, a council person inquired as to whether we are violating your closure of the bridge by permitting pedestrian traffic. If you concur with VJ Engineering and believe that it is safe to allow pedestrian traffic, do you need to file something in SIMMS to reflect this? If you believe that it is not safe to allow pedestrian traffic, we will close the bridge. Let me know either way as soon as possible. Thank you.

William D. Werger, J.D.
Director of Community and Economic
Development/City Attorney
City of Waverly
200 1st Street NE
Waverly, IA 50677
Phone: 319-352-9210
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Website: www.whks.com

whks
engineers + planners + land surveyors

October 21, 2015

Mr. Mike Cherry
City Engineer
City of Waverly
200 First Street NE
Waverly, IA 50677

RE: City of Waverly
3rd Street SE Bridge
Discussion of Bridge Closure

Dear Mr. Cherry:

As you requested, we are providing a more detailed discussion on the closure of the 3rd Street SE Bridge over the Cedar River. The bridge is composed of three 75 foot truss spans. The bridge was inspected by WHKS on February 13, 2015 and it was determined that the bridge should be closed to all traffic at that time because of three serious deficiencies:

- Cracks in the webs of two stringers
- Deteriorated bearings
- Advanced section loss of the sidewalk overhang bracket

The following describes the reasons these specific deficiencies warranted the closure of the bridge. In each case it is clear failure of an individual element can negatively impact other elements and should not be considered acceptable. Photos of these deficiencies are included in the Appendix.

Impacts of Stringer Failure

Failure of one stringer would result in the metal decking spanning a space between the adjacent stringers twice as large as it was designed for. As a result the decking would likely sag, if it could sustain the load at all, which could cause a dangerous driving surface which could cause drivers to lose control and have an accident.

The adjacent stringers would also be subject to greater forces as a result of the failure of one stringer. The general condition of the bridge is poor, and the adjacent stringers may not be able to support the additional load in their deteriorated condition.

Consequences of Bearing Failure

A truss is a fracture critical structure because it lacks redundancy, or the ability to redistribute loads to other members if one should fail. If a single bearing failed and the bridge dropped at one corner there would be significant impact to the entire truss. The entire truss would be subject to loads it was not designed for as the span warps in response to the relative displacement at one corner. If a truss member failed because these additional forces were too large for to sustain, the entire truss system could fail.

If a bearing fails, the bridge would not be able to respond to changes in temperature as designed. Additional forces will be induced in members because thermal movement (expansion and contraction) is restricted. The thermal movement of the bridge is dependent on properly functioning bearings.

The most noticeable impact would be the bump between spans or the approach roadway and the bridge if the bridge to dropped. This bump could cause a motorist to lose control and have an accident. The bridge also supports a natural gas line that could be compromised if a bearing failed causing the bridge to drop.

Sidewalk Closure

Many of the sidewalk overhang brackets have significant section loss and corrosion. The deterioration is worst at the bottom flange near the support. This location is of primary concern because it is where the force in the member is the greatest. At the worst location there are cracks in the welds of the angles that form the bottom flanges, the angles have severe section loss and several through holes, and the web is no longer connected to the bottom flange rendering the section ineffective.

These brackets are spaced at each floor beam location, or panel point, just over 17 feet between brackets. Like the truss as a whole, these brackets are considered fracture critical because of the spacing. This means that if one bracket fails the sidewalk is also likely to fail. The sidewalk would fail because the stringers (which rest on top of the brackets beneath the sidewalk) would be spanning over 34 feet, which is a condition beyond their design, and the adjacent support brackets would be subject to more load.

Bridge Inspection and Rating

WHKS understands that the sidewalk has been reopened to pedestrian traffic. However, our analysis shows that the sidewalk overhang bracket does not have the capacity to support the full American Association of State Highway and Transportation Official (AASHTO) design pedestrian load. AASHTO is a federal design code that establishes criteria to ensure safety of the traveling pubic for new designs as well as load rating of existing structure.

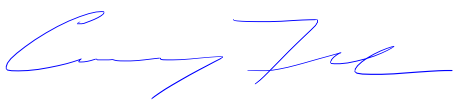
Our rating analysis considers the condition of the worst bracket as described above. The worst case element must be analyzed when rating the bridge because it is the most likely to fail and have negative impacts on other elements.

Our initial inspection report documented in more detail the deterioration at several other areas of the bridge. There are several truss members bent out of plane. The floor beams, stringers, and truss connections have significant section loss and leaf rust in many areas. The concrete piers and abutments are also deteriorating.

As the inspector and bridge program manager public safety is our primary concern. We must consider the ability of the structure to continue to support loads over time before the next inspection. We take in to account the current condition and factor in the historical rate of deterioration to try to predict how much load the structure will continue to safely support over the next inspection cycle (barring unforeseen and unpredictable events). It is our opinion that the current condition of the structure combined with the deterioration that will continue over time presents too great a risk to the public to keep the bridge open.

Sincerely,

***WHKS** & CO.*



Casey V. Faber, P.E.
Bridge Inspection Program Manager

CVF/cvf
cc (w/ enclosures):

APPENDIX: SELECTED PHOTOS

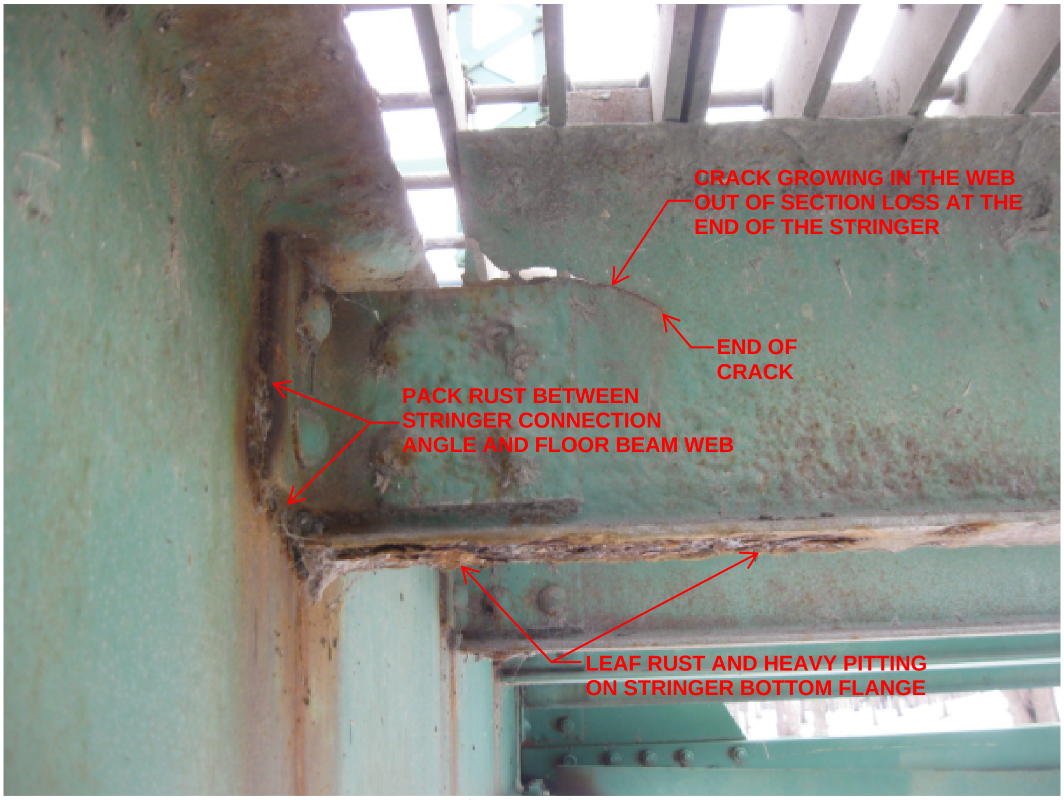


Figure 1: Crack at Stringer End

Figure 2: Crack at Stringer End

Figure 3: Deterioation at Bearing - Truss Gusset Plate Not Connected to Pin

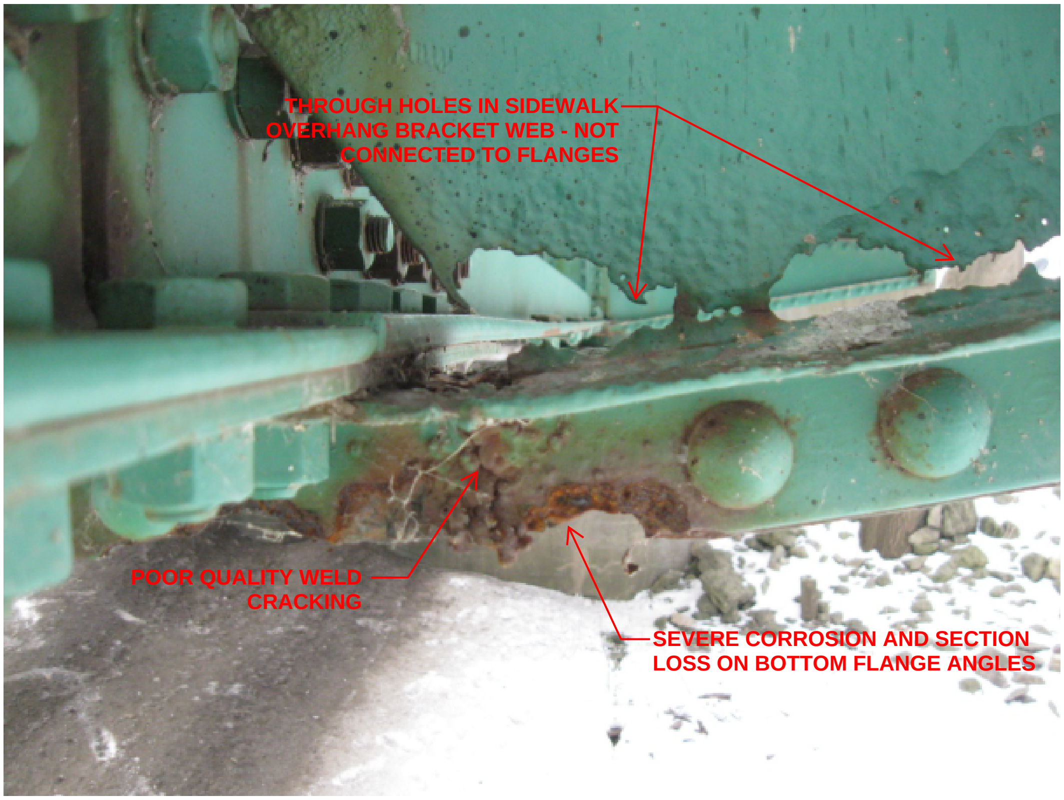


Figure 4: Side View of Sidewalk Overhang Bracket Deterioration

Figure 5: Bottom View of Sidewalk Overhang Bracket

Phil Jones

From: Tim McDermott <tmcdermott@vjengineering.com>
Sent: Friday, October 23, 2015 10:27 AM
To: W D. Werger
Cc: Mike Cherry; Phil Jones; jjacob@vjengineering.com; roneil@vjengineering.com
Subject: RE: 3rd Street Bridge
Attachments: Mathcad - Load Rating_REV 102315.pdf

Gentlemen,
Attached is a revised version of the analysis. Any values that have been altered from the original report are highlighted in red. The dead load acting on the trusses was updated from the values taken from the original bridge plans to reflect the lighter steel grid deck. As you will see throughout the analysis, the changes to the load carrying capacity of the bridge are negligible. The unit weight of the deck used by Mr. Kehe in his analysis was significantly lighter than what is actually on the bridge (11 psf vs. 18-20 psf actual). The 22% and 33% dead load reductions claimed by Mr. Kehe are actually 3% and 7%, respectively.

Regarding the various suggestions and cost estimates provided by Mr. Kehe, none of the **repairs** suggested come close to an actual **rehabilitation**. Reopening the bridge at a limited capacity without truly rehabilitating the structure does not make financial sense (ie. too high of a cost for limited use and service life), and in my opinion is too high of a risk of catastrophic failure.

The scope of VJ's investigation was to evaluate a few replacement options as well as a few rehabilitation options. This is a typical evaluation in a feasibility study and produces a meaningful comparison of bridges functioning at a full capacity (ie. code compliant). When you begin adding alternatives that function at a limited capacity into the comparison, it's like comparing apples to oranges and typically ends up confusing the recipients of the report and the general public.

I hope this clarifies some of the recent discussions. Please let me know if there's any additional information I can provide. We appreciate you choosing VJ for this project and look forward to the opportunity for ongoing involvement with the bridge. Thanks- Tim

Tim McDermott, P.E.
Structural Engineer

VJ Engineering
2570 Holiday Road, Suite 10
Coralville, IA 52241
Cell: (319) 540-6956
Phone: (319) 338-4939
Fax: (319) 338-9457

Subject: 3rd Street Bridge
Date: Wed, 21 Oct 2015 14:41:50 -0500
From: wdwerger@ci.waverly.ia.us

To: tmcdermott@vjengineering.com
CC: mike@ci.waverly.ia.us; philj@ci.waverly.ia.us

Tim: The Council reviewed the status of the bridge again on Monday. I informed them that WHKS is the current program manager for the city's bridges and is ultimately responsible for the closure or opening of the bridge to vehicles or pedestrians. We are still very interested in having you respond to the findings and conclusions in the Kehe report, especially to the extent that his opinion indicates that some of your calculations included some discrepancies. I hope you can address those matters as soon as possible. We look forward to your responses.

William D. Werger, J.D.
Director of Community and Economic
Development/City Attorney
City of Waverly
200 1st Street NE
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Phone: 319-352-9210
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Email: wdwerger@ci.waverly.ia.us

RED HIGHLIGHT= REVISIONS MADE ON 10-23-15

PEDESTRIAN ONLY USE

Span length: $L_{span\_1} := 120ft$ — $W_{deck} := 18ft$ Deck width

Panel length: $L_{panel\_1} := 17.15ft$ — $F_y := 30ksi$ per IDOT HR-239

Dead Load

Diff. between wood and steel deck (east truss)

$$DL\_corrected := 18psf \cdot 18ft \cdot .5 - [50pcf \cdot (3in + 2in) \cdot 18ft \cdot .5 + 2psf \cdot 18ft \cdot .5] = -43.5$$

Corrected DL (east truss)

$$w_{DL\_1} := 575 \frac{lb}{ft} - DL\_corrected = 618.5 \frac{lb}{ft}$$

%Difference (east truss)

$$Difference\_\%_1 := \frac{|DL\_corrected|}{w_{DL\_1}} = 7.03 \cdot \%$$

Corrected DL (west truss)

$$w_{DL\_2} := 1275 \frac{lb}{ft} - DL\_corrected = 1318.5 \frac{lb}{ft}$$

%Difference (west truss)

$$Difference\_\%_2 := \frac{|DL\_corrected|}{w_{DL\_2}} = 3.3 \cdot \%$$

DL Panel Point Load

$$DL_1 := L_{panel\_1} \cdot w_{DL\_1} = 10.61 \cdot k \qquad DL_2 := L_{panel\_1} \cdot w_{DL\_2} = 22.61 \cdot k$$

Pedestrian Live Load

AASHTO pedestrian LL per LRFD 3.6.1.6

$$w_{ped\_LL} := 90psf$$

LL Panel Point Load (east)

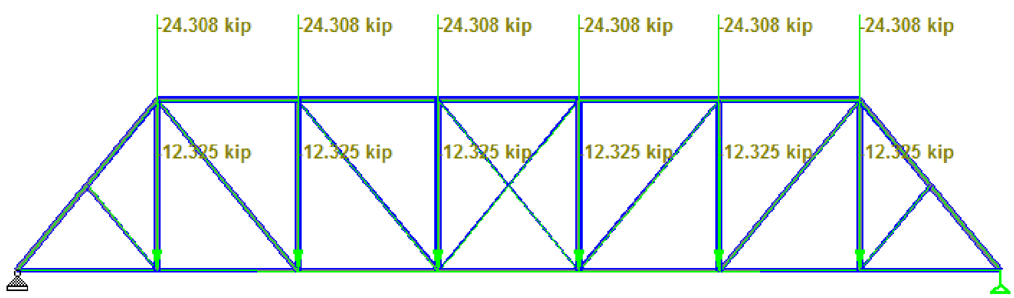
$$LL_{ped\_1} := \frac{W_{deck}}{2} \cdot L_{panel\_1} \cdot w_{ped\_LL} = 13.89 \cdot k$$

LL Panel Point Load (west)

$$LL_{ped\_2} := \left(\frac{W_{deck}}{2} + 5ft\right) \cdot L_{panel\_1} \cdot w_{ped\_LL} = 21.61 \cdot k$$

East Truss

Truss forces analyzed in STAAD per AASHTO LRFD Strength I (factored loads shown)



Analysis Results per AASHTO LRFR:

			A_g (in^2)	L (in)	r (in)	KL/r	Allowable Comp. $\phi_c P_n$ k	Allowable Tension $\phi_y F_y A_g$ k	ACTUAL LOAD $?_i P_u$ k	S.R.
2C8x16.25 (w/ 5/16x14 PL)	L0-U1	TC	14	162.66	3.16	51.47	336.48	399.00	141.9	0.42
2C8x11.5 (w/ 5/16x14 PL)	U1-U2	TC	11.1	205.75	3.16	65.11	249.24	316.92	149.5	0.60
2C8x11.5 (w/ 5/16x14 PL)	U2-U3	TC	11.1	205.75	3.16	65.11	249.24	316.92	179.4	0.72
2C8x11.5 (w/ 5/16x14 PL)	U3-U4	TC	11.1	205.75	3.16	65.11	249.24	316.92	177.8	0.71
2L5x3.5x5/16	L0-L1	BC	5.12	205.75	1.02	201.72	28.40	145.92	-89.7	0.61
2L5x3.5x5/16	L1-L2	BC	5.12	205.75	1.02	201.72	28.40	145.92	-89.7	0.61
2L5x3.5x7/16	L2-L3	BC	8.01	205.75	1.00	205.75	42.70	228.29	-149.5	0.65
2L5x3.5x1/2	L2-L3	BC	8.01	205.75	1.00	205.75	42.70	228.29	-181	0.79
2C8x11.25 (w/ 2"x1/4" lacing)	L1-U1	vert	6.61	252.00	3.16	79.75	134.98	188.39	-36.6	0.19
2C8x11.25 (w/ 2"x1/4" lacing)	L2-U2	vert	6.61	252.00	3.16	79.75	134.98	188.39	36.6	0.27
2C8x11.25 (w/ 2"x1/4" lacing)	L3-U3	vert	6.61	252.00	3.16	79.75	134.98	188.39	2	0.01
2L5x3x5/16	U1-L2	diag.	4.81	325.31	0.85	382.72	7.41	137.09	-94.6	0.69
2L3.5x2.5x1/4	U2-L3	diag.	2.9	325.31	0.73	445.63	3.30	82.65	-47.3	0.57
2L2.5x2.5x1/2	U3-L4	diag.	4.5	325.31	0.74	439.61	5.26	128.25	2.6	0.49

Member L2-L3 controls

AASHTO LRFR Pedestrian Load Rating:

Condition Factor $\varphi_{c\_truss\_1} := 0.85$

System Factor $\varphi_{s\_truss\_1} := 0.9$

L2-L3 Capacity $C_{truss\_1} := \min(0.85, \varphi_{c\_truss\_1} \cdot \varphi_{s\_truss\_1}) \cdot 228.3\mathbf{k}$

L2-L3 DL $\gamma DL_{truss\_1} := 1.25 \cdot 40.2\mathbf{k}$

L2-L3 LL $\gamma LL_{truss\_1} := 1.75 \cdot 56.7\mathbf{k}$

Inventory Rating

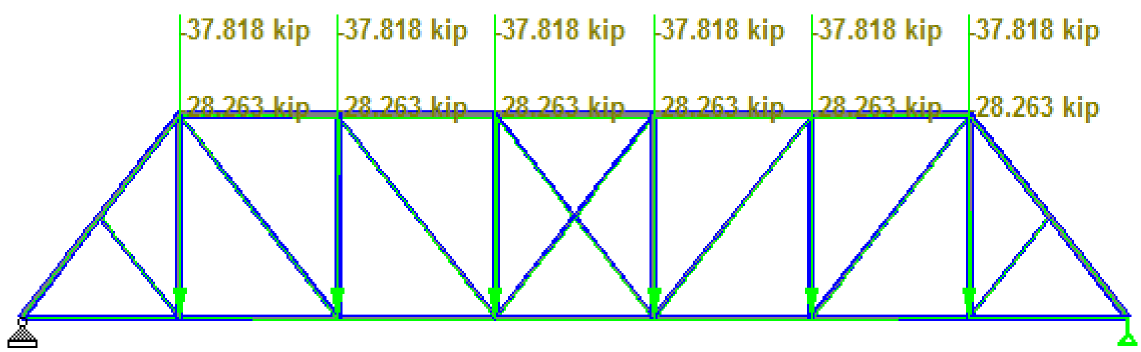
$$INV_{truss\_1} := w_{ped\_LL} \cdot \frac{C_{truss\_1} - \gamma DL_{truss\_1}}{\gamma LL_{truss\_1}} = 112.83 \cdot \mathbf{psf}$$

Operating Rating

$$OPR_{truss\_1} := w_{ped\_LL} \cdot \frac{C_{truss\_1} - \gamma DL_{truss\_1}}{\gamma LL_{truss\_1} \cdot \left(\frac{1.35}{1.75}\right)} = 146.27 \cdot \mathbf{psf}$$

West Truss

Truss forces analyzed in STAAD per AASHTO LRFD Strength I (factored loads shown)



Analysis Results per AASHTO LRFR:

			A_g	L	r		$\phi_c P_n$	$\phi_y F_y A_g$	?$_i P_u$	
			(in^2)	(in)	(in)	KL/r	k	k	k	S.R.
2C9x20 (w/ 3/8x15 PL)	L0-U1	TC	16.4	162.66	3.44	47.28	402.13	468.26	255.91	0.64
2C9x13.25 (w/ 3/8x15 PL)	U1-U2	TC	12.6	205.75	3.56	57.79	293.09	358.25	269.71	0.92
2C9x13.25 (w/ 3/8x15 PL)	U2-U3	TC	12.6	205.75	3.56	57.79	293.09	358.25	323.7	1.10
2C9x13.25 (w/ 3/8x15 PL)	U3-U4	TC	12.6	205.75	3.56	57.79	293.09	358.25	320.8	1.09
2L5x3.5x9/16	L0-L1	BC	11.6	205.75	0.97	212.11	58.19	330.60	-161.8	0.49
2L5x3.5x9/16	L1-L2	BC	11.6	205.75	0.97	212.11	58.19	330.60	-161.8	0.49
2L5x3.5x5/16 + 2L5x3.5x3/8	L2-L3	BC	11.2	205.75	1.02	201.72	62.23	319.77	-269.7	0.84
4L5x3.5x7/16	L3-L4	BC	10.2	205.75	1.02	201.72	56.80	291.84	-326.5	1.12
2C8x11.25 (w/ 2"x1/4" lacing)	L1-U1	vert	6.61	252.00	3.16	79.75	134.98	188.39	-66.1	0.35
2C8x11.25 (w/ 2"x1/4" lacing)	L2-U2	vert	6.61	252.00	3.16	79.75	134.98	188.39	66.1	0.49
2C8x11.25 (w/ 2"x1/4" lacing)	L3-U3	vert	6.61	252.00	3.16	79.75	134.98	188.39	3.5	0.03
2L6x3.5x7/16	U1-L2	diag.	9.04	325.31	0.97	335.37	18.14	257.64	-170.6	0.66
2L5x3x5/16	U2-L3	diag.	4.81	325.31	0.85	382.72	7.41	137.09	-85.3	0.62
2L2.5x2.5x1/4	U3-L4	diag.	2.37	162.66	0.76	214.02	11.68	67.55	4.6	0.39

Member L3-L4 controls

AASHTO LRFR Pedestrian Load Rating:

L3-L4 Capacity
$$C_{truss\_2} := \min\left(0.85, \varphi_{c\_truss\_1} \cdot \varphi_{s\_truss\_1}\right) \cdot 291.8k$$

L3-L4 DL
$$\gamma DL_{truss\_2} := 1.25 \cdot 111.7k$$

L3-L4 LL
$$\gamma LL_{truss\_2} := 1.75 \cdot 106.8k$$

Inventory Rating
$$INV_{truss\_2} := w_{ped\_LL} \cdot \frac{C_{truss\_2} - \gamma DL_{truss\_2}}{\gamma LL_{truss\_2}} = 40.26 \cdot psf$$

Operating Rating
$$OPR_{truss\_2} := w_{ped\_LL} \cdot \frac{C_{truss\_2} - \gamma DL_{truss\_2}}{\gamma LL_{truss\_2} \cdot \left(\frac{1.35}{1.75}\right)} = 52.19 \cdot psf$$

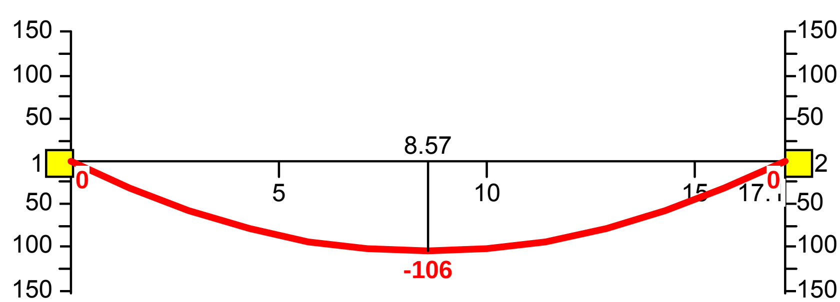
Floor System

Exterior Stringers:

Distributed DL
$$w_{DL\_1\_ext\_strngr} := \frac{w_{DL\_1}}{W_{deck}} \cdot \frac{(2ft + 5in)}{2} = 41.52 \frac{lb}{ft}$$

Distributed LL
$$w_{LL\_1\_ext\_strngr} := w_{ped\_LL} \cdot \frac{(2ft + 5in)}{2} = 108.75 \frac{lb}{ft}$$

Stringer forces analyzed in STAAD per AASHTO LRFR Pedestrian Strength I



East exterior stringer channels are C9x13.

$$Z_{x\_C9x13} := 12.6in^3$$

Nominal flexural resistance
$$\phi M_{n\_ext\_strngr\_1} := F_y \cdot Z_{x\_C9x13} = 31.5 \cdot ft \cdot k$$

AASHTO LRFR Pedestrian Load Rating:

Condition Factor $\varphi_{c\_ext\_strngr\_1} := 0.85$

System Factor $\varphi_{s\_ext\_strngr\_1} := 1$

Exterior Stringer Capacity $C_{ext\_strngr\_1} := \varphi_{c\_ext\_strngr\_1} \cdot \varphi_{s\_ext\_strngr\_1} \cdot \phi M_{n\_ext\_strngr\_1}$

Exterior Stringer DL moment $\gamma DL_{ext\_strngr\_1} := 1.25 \cdot 1.43 \mathbf{ft} \cdot \mathbf{k}$

Exterior Stringer LL moment $\gamma LL_{ext\_strngr\_1} := 1.75 \cdot 4.01 \mathbf{k} \cdot \mathbf{ft}$

Inventory Rating $INV_{ext\_strngr\_1} := w_{ped\_LL} \cdot \frac{C_{ext\_strngr\_1} - \gamma DL_{ext\_strngr\_1}}{\gamma LL_{ext\_strngr\_1}} = 320.47 \cdot \mathbf{psf}$

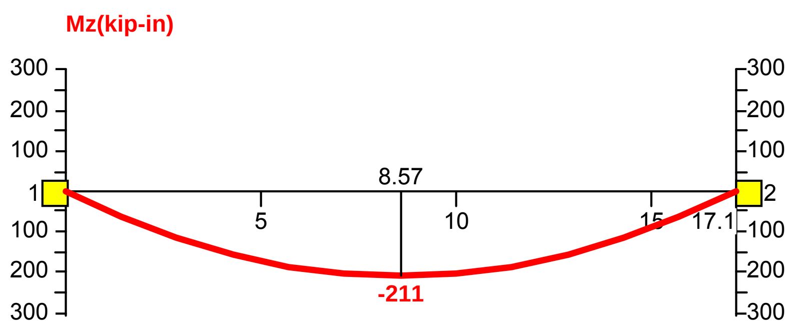
Operating Rating $OPR_{ext\_strngr\_1} := w_{ped\_LL} \cdot \frac{C_{ext\_strngr\_1} - \gamma DL_{ext\_strngr\_1}}{\gamma LL_{ext\_strngr\_1} \cdot \left(\frac{1.35}{1.75}\right)} = 415.42 \cdot \mathbf{psf}$

Interior Stringers:

Distributed DL $w_{DL\_1\_strngr} := \frac{w_{DL\_1}}{W_{deck}} \cdot (2\mathbf{ft} + 5\mathbf{in}) = 83.04 \frac{\mathbf{lb}}{\mathbf{ft}}$

Distributed LL $w_{LL\_1\_strngr} := w_{ped\_LL} \cdot (2\mathbf{ft} + 5\mathbf{in}) = 217.5 \frac{\mathbf{lb}}{\mathbf{ft}}$

Stringer forces analyzed in STAAD per AASHTO LRFR Pedestrian Strength I



Interior stringers are I9x21

$Z_{x\_I9x21} := 21.7 \mathbf{in}^3$

Nominal flexural resistance $\phi M_{n\_strngr\_1} := F_y \cdot Z_{x\_I9x21} = 54.25 \cdot \mathbf{ft} \cdot \mathbf{k}$

AASHTO LRFR Load Rating:

Stringer Capacity $C_{strngr\_1} := \varphi_{c\_ext\_strngr\_1} \cdot \varphi_{s\_ext\_strngr\_1} \cdot \phi M_{n\_strngr\_1}$

Stringer DL moment $\gamma DL_{strngr\_1} := 1.25 \cdot 2.83 \mathbf{ft} \cdot \mathbf{k}$

Stringer LL moment $\gamma LL_{strngr\_1} := 1.75 \cdot 8.02 \mathbf{k} \cdot \mathbf{ft}$

Inventory Rating $INV_{strngr\_1} := w_{ped\_LL} \cdot \frac{C_{strngr\_1} - \gamma DL_{strngr\_1}}{\gamma LL_{strngr\_1}} = 273.01 \cdot \mathbf{psf}$

Operating Rating $OPR_{strngr\_1} := w_{ped\_LL} \cdot \frac{C_{strngr\_1} - \gamma DL_{strngr\_1}}{\gamma LL_{strngr\_1} \cdot \left(\frac{1.35}{1.75}\right)} = 353.91 \cdot \mathbf{psf}$

Interior Stringers (at C.L. of west truss):

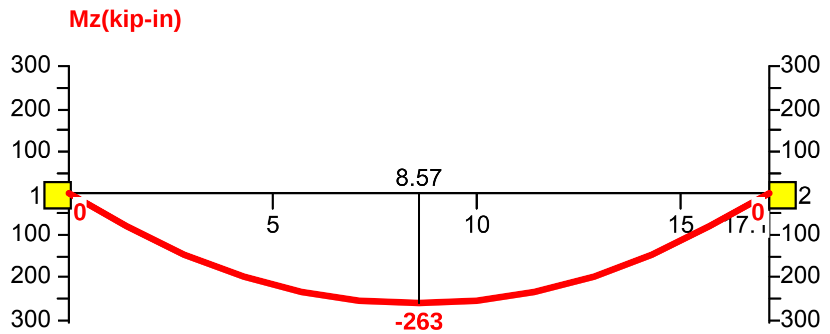
Distributed DL

$$w_{DL\_2\_ext\_strngr} := \frac{w_{DL\_2}}{W_{deck}} \cdot (2\mathbf{ft} + 5\mathbf{in}) = 177.02\,\frac{\mathbf{lb}}{\mathbf{ft}}$$

Distributed LL

$$w_{LL\_2\_ext\_strngr} := w_{ped\_LL} \cdot (2\mathbf{ft} + 5\mathbf{in}) = 217.5\,\frac{\mathbf{lb}}{\mathbf{ft}}$$

Stringer forces analyzed in STAAD per AASHTO LRFR Pedestrian Strength I



Stringers are C12x20

$$Z_{x\_C12x20} := 25.6\mathbf{in}^3$$

Nominal flexural resistance

$$\phi M_{n\_ext\_strngr\_2} := F_y \cdot Z_{x\_C12x20} = 64 \cdot \mathbf{ft} \cdot \mathbf{k}$$

AASHTO LRFR Load Rating:

Stringer Capacity

$$C_{ext\_strngr\_2} := \varphi_{c\_ext\_strngr\_1} \cdot \varphi_{s\_ext\_strngr\_1} \cdot \phi M_{n\_ext\_strngr\_2}$$

Stringer DL moment

$$\gamma DL_{ext\_strngr\_2} := 1.25 \cdot 6.28\mathbf{ft} \cdot \mathbf{k}$$

Stringer LL moment

$$\gamma LL_{ext\_strngr\_2} := 1.75 \cdot 8.02\mathbf{k} \cdot \mathbf{ft}$$

Inventory Rating

$$INV_{ext\_strngr\_2} := w_{ped\_LL} \cdot \frac{C_{ext\_strngr\_2} - \gamma DL_{ext\_strngr\_2}}{\gamma LL_{ext\_strngr\_2}} = 298.5 \cdot \mathbf{psf}$$

Operating Rating

$$OPR_{ext\_strngr\_2} := w_{ped\_LL} \cdot \frac{C_{ext\_strngr\_2} - \gamma DL_{ext\_strngr\_2}}{\gamma LL_{ext\_strngr\_2} \cdot \left(\frac{1.35}{1.75}\right)} = 386.95 \cdot \mathbf{psf}$$

Sidewalk Bracket (tapered I beam):

To account for the bracket that has significant web section loss and is disjointed from the bottom angles (bottom flange), the bottom 2" of the tapered I beam are excluded from the capacity calculation.

Section properties at truss end

$$A_{sw\_bracket} := 16\mathbf{in} \cdot 0.25\mathbf{in} + 2 \cdot 2.37\mathbf{in}^2 = 8.74 \cdot \mathbf{in}^2 \quad a_{sw\_bracket} := \frac{10.7232}{2}\mathbf{in} + 2.19\mathbf{in}$$

$$Z_{sw\_bracket} := \frac{A_{sw\_bracket}}{2} \cdot a_{sw\_bracket} = 33 \cdot \mathbf{in}^3$$

Flexural resistance at truss end

$$\phi M_{sw\_bracket} := F_y \cdot Z_{sw\_bracket} = 82.5 \cdot \mathbf{ft} \cdot \mathbf{k}$$

AASHTO LRFR Load Rating:

Bracket Capacity

$$C_{sw\_bracket} := 0.85 \cdot \phi M_{sw\_bracket} = 70.126 \cdot \mathbf{ft} \cdot \mathbf{k}$$

DL moment at truss end

$$\gamma DL_{sw\_bracket} := 1.25 \cdot \frac{\left(2.4\,\frac{\mathbf{k}}{\mathbf{ft}}\right)}{2} \cdot (5\mathbf{ft})^2$$

LL moment at truss end

$$\gamma LL_{sw\_bracket} := 1.75\cdot\frac{\left(1.54\frac{k}{ft}\right)}{2}\cdot(5ft)^2$$

Inventory Rating

$$INV_{sw\_bracket} := w_{ped\_LL}\cdot\frac{C_{sw\_bracket} - \gamma DL_{sw\_bracket}}{\gamma LL_{sw\_bracket}} = 87.16\cdot psf$$

Operating Rating

$$OPR_{sw\_bracket} := w_{ped\_LL}\cdot\frac{C_{sw\_bracket} - \gamma DL_{sw\_bracket}}{\gamma LL_{sw\_bracket}\cdot\left(\frac{1.35}{1.75}\right)} = 112.99\cdot psf$$

Floor Beams:

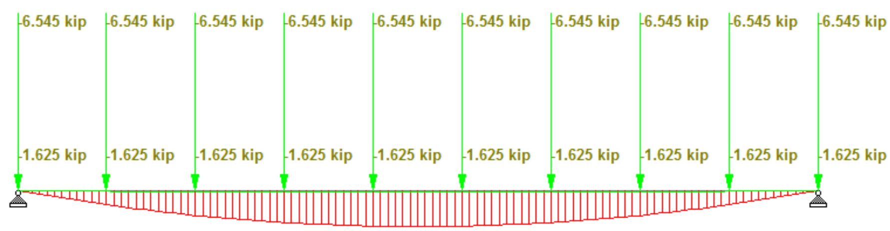
DL from stringers

$$P_{DL\_1\_2\_FB} := 1.3k$$

LL from stringers

$$P_{LL\_1\_2\_FB} := 3.74k$$

Floor beam forces analyzed in STAAD per AASHTO Pedestrian LRFD Strength I (factored loads shown)



Floor beams are W18x55

$$Z_{x\_W18x55} := 112in^3$$

Nominal flexural resistance

$$\phi M_{n\_FB\_1} := F_y\cdot Z_{x\_W18x55} = 280\cdot ft\cdot k$$

AASHTO LRFR Load Rating:

System Factor

$$\varphi_{s\_FB\_1} := 0.85$$

Floor Beam Capacity

$$C_{FB\_1} := \varphi_{s\_FB\_1}\cdot\phi M_{n\_FB\_1}$$

Floor Beam DL moment

$$\gamma DL_{FB\_1} := 1.25\cdot 27.9ft\cdot k$$

Floor Beam LL moment

$$\gamma LL_{FB\_1} := 1.75\cdot 80.3k\cdot ft$$

Inventory Rating

$$INV_{FB\_1} := w_{ped\_LL}\cdot\frac{C_{FB\_1} - \gamma DL_{FB\_1}}{\gamma LL_{FB\_1}} = 130.09\cdot psf$$

Operating Rating

$$OPR_{FB\_1} := w_{ped\_LL}\cdot\frac{C_{FB\_1} - \gamma DL_{FB\_1}}{\gamma LL_{FB\_1}\cdot\left(\frac{1.35}{1.75}\right)} = 168.64\cdot psf$$

Truss Bearings:

The 4"ϕ pin at the SW bearing has 2.5" of remaining section.

Pin & Bearing plates

$$D_{SW} := 2.5in \qquad t_{SW} := 1.75in$$

Nominal bearing resistance

$$\phi P_{n\_SW} := F_y\cdot 2\cdot D_{SW}\cdot t_{SW} = 262.5\cdot k$$

Strength I truss reactions (from STAAD analysis)

$$V_{u\_SW} := 189.3k$$

AASHTO LRFR Load Rating:

Bearing Capacity $C_{SW} := 0.85 \cdot \phi P_{n\_SW}$

Bearing DL reaction $\gamma DL_{SW} := 1.25 \cdot 65.7 \cdot k$

Bearing LL reaction $\gamma LL_{SW} := 1.75 \cdot 64.8 k$

Inventory Rating $INV_{SW\_brg} := w_{ped\_LL} \cdot \dfrac{C_{SW} - \gamma DL_{SW}}{\gamma LL_{SW}} = 111.9 \cdot psf$

Operating Rating $OPR_{SW\_brg} := w_{ped\_LL} \cdot \dfrac{C_{SW} - \gamma DL_{SW}}{\gamma LL_{SW} \cdot \left(\dfrac{1.35}{1.75}\right)} = 145.06 \cdot psf$

BRIDGE RATINGFOR PEDESTRIAN USE:

$INV_{ped} := \min(INV_{SW\_brg}, INV_{FB\_1}, INV_{strngr\_1}, INV_{ext\_strngr\_1}, INV_{ext\_strngr\_2}, INV_{truss\_1}, INV_{truss\_2}, INV_{sw\_}$

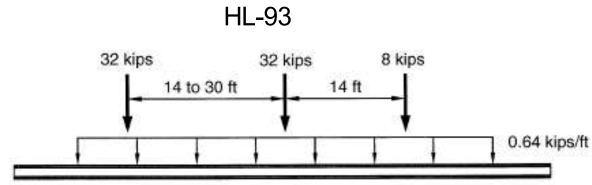
Inventory Rating $INV_{ped} = 40.3 \cdot psf$

$OPR_{ped} := \min(OPR_{SW\_brg}, OPR_{FB\_1}, OPR_{strngr\_1}, OPR_{ext\_strngr\_1}, OPR_{ext\_strngr\_1}, OPR_{truss\_1}, OPR_{truss\_2}, OPR_{s}$

Operating Rating $OPR_{ped} = 52.2 \cdot psf$

This rating is 55% below AASHTO pedestrian design live load (90 psf).

VEHICULAR USE



Design Live Load

Stringer spacing

$S_{stringer} := 2ft + 5in$

Live load distribution factor per AASHTO 4.6.2.2.2b-1

$DF_{interior} := \frac{S_{stringer}}{8ft} = 0.3021$

Live load impact factor

$IM := 0.33$

Wheel loads

$P_1 := 0.5 \cdot 8k = 4000\,lb$ $P_2 := 0.5 \cdot 32k = 16000\,lb$

Lane load

$LL_{lane} := DF_{interior} \cdot 640 \frac{lb}{ft} = 193.33 \frac{lb}{ft}$

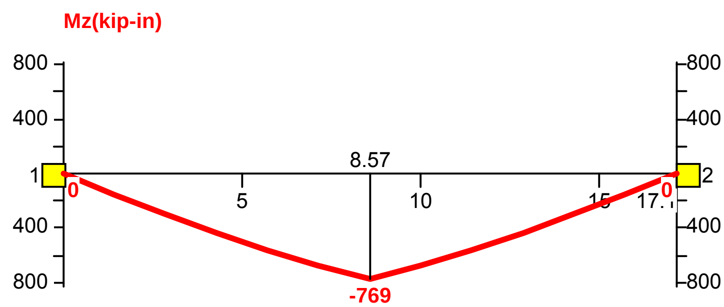
Floor System

Interior

Point loads on stringers

$LL_{stringer} := (1 + IM) \cdot DF_{interior} \cdot P_2 = 6.43 \cdot k$ (Worst case when back axle is at stringer mid-span)

Stringer forces analyzed in STAAD per AASHTO LRFR Strength I



AASHTO LRFR Load Rating:

Stringer LL moment

$\gamma LL_{strngr\_2} := 1.75 \cdot 34.6k \cdot ft$

Inventory Rating Factor

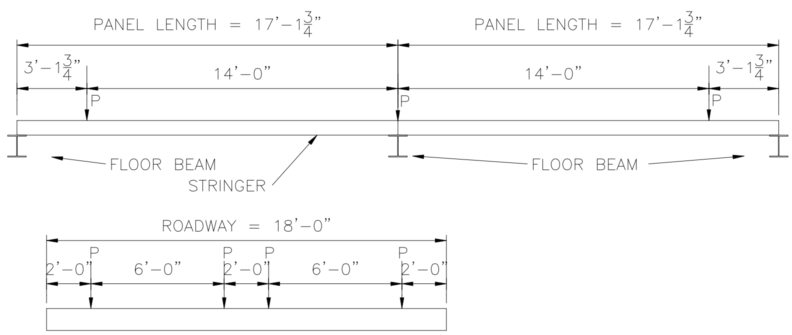
$INV_{strngr\_2} := 1 \cdot \frac{C_{strngr\_1} - \gamma DL_{strngr\_1}}{\gamma LL_{strngr\_2}} = 0.7$

Operating Rating Factor

$OPR_{strngr\_2} := 1 \cdot \frac{C_{strngr\_1} - \gamma DL_{strngr\_1}}{\gamma LL_{strngr\_2} \cdot \left(\frac{1.35}{1.75}\right)} = 0.91$

Floor Beams:

Worst case LL configuration for floor beams (trucks centered on floor beam):



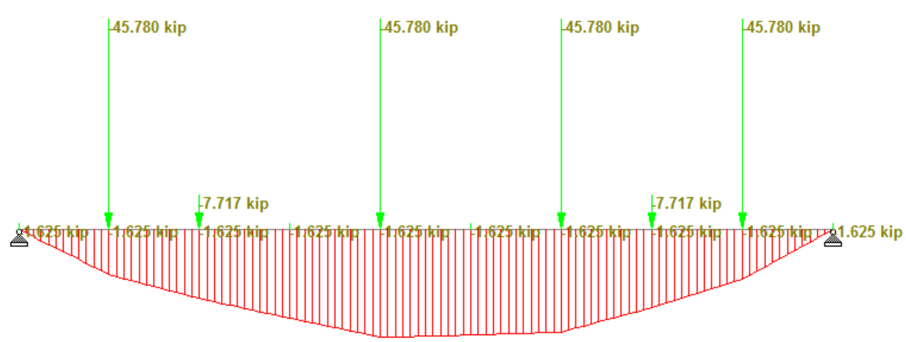
Point loads on floor beams

$$\mathbf{LL_{fb}} := (1 + \mathbf{IM})\cdot\left[\mathbf{P_2}\cdot\left[1 + 1\cdot\left(\frac{3\mathbf{ft} + 1.75\mathbf{in}}{17\mathbf{ft} + 1.75\mathbf{in}}\right)\right] + \mathbf{P_1}\cdot\left(\frac{3\mathbf{ft} + 1.75\mathbf{in}}{17\mathbf{ft} + 1.75\mathbf{in}}\right)\right] = 26.16\cdot\mathbf{k}$$

Lane loads on floor beams

$\mathbf{LL_{fb\_lane}} := (1 + \mathbf{IM})\cdot\mathbf{LL_{lane}}\cdot\mathbf{L_{panel\_1}} = 4.41\cdot\mathbf{k}$ (Applied at 5 ft from each end)

Floor beam forces analyzed in STAAD per AASHTO LRFR Strength I



AASHTO LRFR Load Rating:

Floor beam live load moment

$$\gamma\mathbf{LL_{FB\_2}} := 1.75\cdot274.9\mathbf{k}\cdot\mathbf{ft}$$

Inventory Rating Factor

$$\mathbf{INV_{FB\_2}} := 1\cdot\frac{\mathbf{C_{FB\_1}} - \gamma\mathbf{DL_{FB\_1}}}{\gamma\mathbf{LL_{FB\_2}}} = 0.42$$

Operating Rating Factor

$$\mathbf{OPR_{FB\_2}} := 1\cdot\frac{\mathbf{C_{FB\_1}} - \gamma\mathbf{DL_{FB\_1}}}{\gamma\mathbf{LL_{FB\_2}}\cdot\left(\frac{1.35}{1.75}\right)} = 0.55$$

East Truss

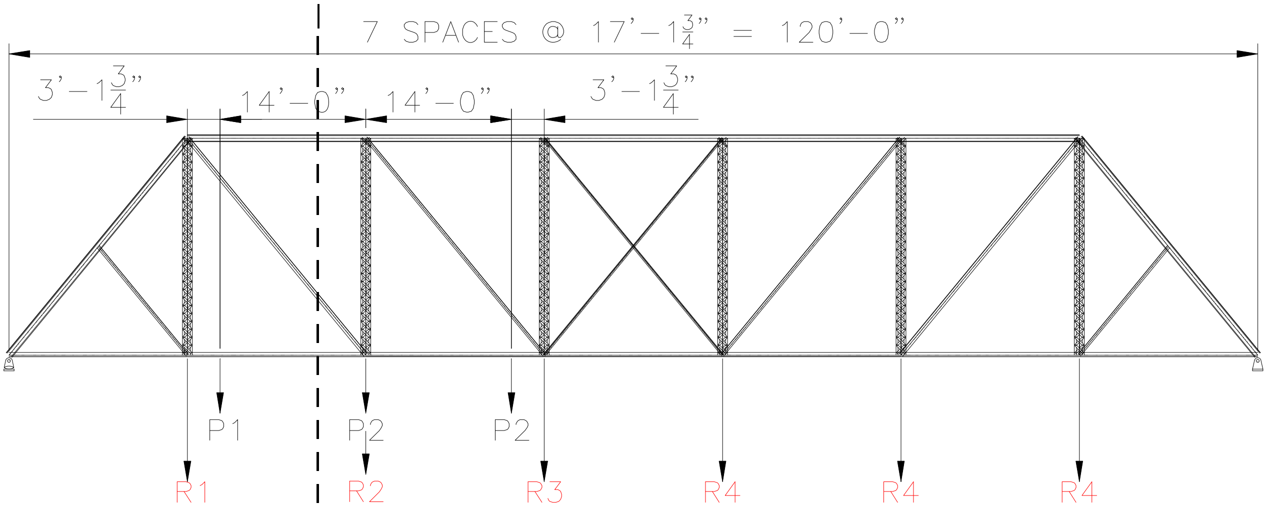
$$R_1 := (1 + IM)\cdot 2P_1\cdot\left(\frac{L_{panel\_1} - 3ft - 1.75in}{L_{panel\_1}}\right) + (1 + IM)\cdot LL_{lane}\cdot L_{panel\_1} = 13.1\cdot k$$

$$R_2 := (1 + IM)\cdot 2P_2\cdot\left(\frac{3ft + 1.75in}{L_{panel\_1}} + 1\right) + (1 + IM)\cdot 2P_1\cdot\left(\frac{3ft + 1.75in}{L_{panel\_1}}\right) + (1 + IM)\cdot LL_{lane}\cdot L_{panel\_1} = 56.73\cdot$$

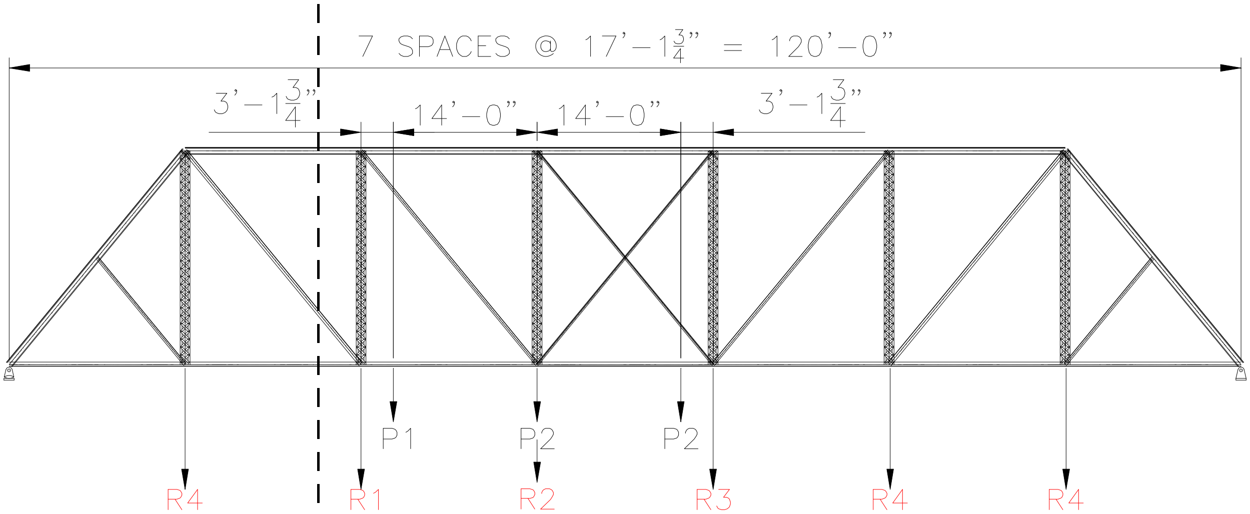
$$R_3 := (1 + IM)\cdot 2P_2\cdot\left(\frac{L_{panel\_1} - 3ft - 1.75in}{L_{panel\_1}}\right) + (1 + IM)\cdot LL_{lane}\cdot L_{panel\_1} = 39.16\cdot k$$

$$R_4 := (1 + IM)\cdot LL_{lane}\cdot L_{panel\_1} = 4.41\cdot k$$

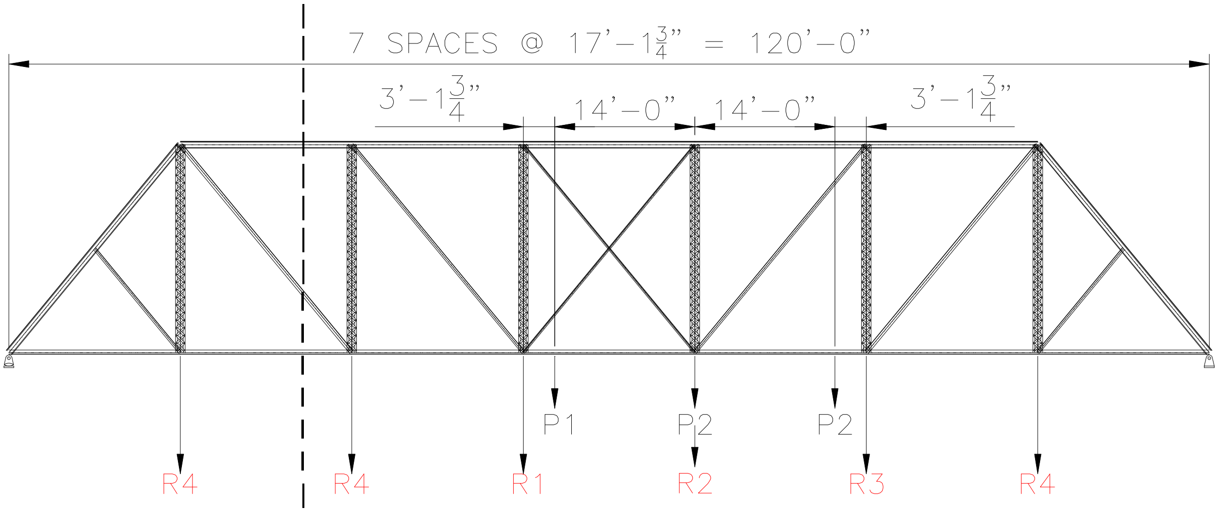
Case 1:



Case 2:



Case 3:



Truss forces analyzed in STAAD per AASHTO LRFR Strength I

							Allowable Comp.	Allowable Tension	ACTUAL LOAD	
			A_g	L	r		$\phi_c P_n$	$\phi_y F_y A_g$	?$_i$P$_u$	
			(in^2)	(in)	(in)	KL/r	k	k	k	S.R.
2C8x16.25 (w/ 5/16x14 PL)	L0-U1	TC	14	162.66	3.16	51.47	336.48	399.00	223.7	0.66
2C8x11.5 (w/ 5/16x14 PL)	U1-U2	TC	11.1	205.75	3.16	65.11	249.24	316.92	254.2	1.02
2C8x11.5 (w/ 5/16x14 PL)	U2-U3	TC	11.1	205.75	3.16	65.11	249.24	316.92	304.3	1.22
2C8x11.5 (w/ 5/16x14 PL)	U3-U4	TC	11.1	205.75	3.16	65.11	249.24	316.92	294.5	1.18
2L5x3.5x5/16	L0-L1	BC	5.12	205.75	1.02	201.72	28.40	145.92	-141.5	0.97
2L5x3.5x5/16	L1-L2	BC	5.12	205.75	1.02	201.72	28.40	145.92	-141.5	0.97
2L5x3.5x7/16	L2-L3	BC	8.01	205.75	1.00	205.75	42.70	228.29	-254.2	1.11
2L5x3.5x1/2	L2-L3	BC	8.01	205.75	1.00	205.75	42.70	228.29	-299.8	1.31
2C8x11.25 (w/ 2"x1/4" lacing)	L1-U1	vert	6.61	252.00	3.16	79.75	134.98	188.39	-35.3	0.19
2C8x11.25 (w/ 2"x1/4" lacing)	L2-U2	vert	6.61	252.00	3.16	79.75	134.98	188.39	98.4	0.73
2C8x11.25 (w/ 2"x1/4" lacing)	L3-U3	vert	6.61	252.00	3.16	79.75	134.98	188.39	24.4	0.18
2L5x3x5/16	U1-L2	diag.	4.81	325.31	0.85	382.72	7.41	137.09	-183.8	1.34
2L3.5x2.5x1/4	U2-L3	diag.	2.9	325.31	0.73	445.63	3.30	82.65	-127	1.54
2L2.5x2.5x1/2	U3-L4	diag.	4.5	325.31	0.74	439.61	5.26	128.25	-38.7	0.30

Member U2-L3 controls

AASHTO LRFR Load Rating:

U2-L3 Capacity

$$C_{truss\_3} := \min\left(0.85, \varphi_{c\_truss\_1} \cdot \varphi_{s\_truss\_1}\right) \cdot 82.65\mathbf{k}$$

U2-L3 DL

$$\gamma DL_{truss\_3} := 1.25 \cdot 12.7\mathbf{k}$$

U2-L3 LL

$$\gamma LL_{truss\_3} := 1.75 \cdot 63.5\mathbf{k}$$

Inventory Rating Factor

$$INV_{truss\_3} := 1 \cdot \frac{C_{truss\_3} - \gamma DL_{truss\_3}}{\gamma LL_{truss\_3}} = 0.43$$

Operating Rating Factor

$$OPR_{truss\_3} := 1 \cdot \frac{C_{truss\_3} - \gamma DL_{truss\_3}}{\gamma LL_{truss\_3} \cdot \left(\frac{1.35}{1.75}\right)} = 0.55$$

West Truss

$$R_5 := (1 + IM) \cdot 2P_1 \cdot \left(\frac{L_{panel\_1} - 3ft - 1.75in}{L_{panel\_1}} \right) + (1 + IM) \cdot LL_{lane} \cdot L_{panel\_1} + w_{ped\_LL} \cdot L_{panel\_1} \cdot 5ft = 2($$

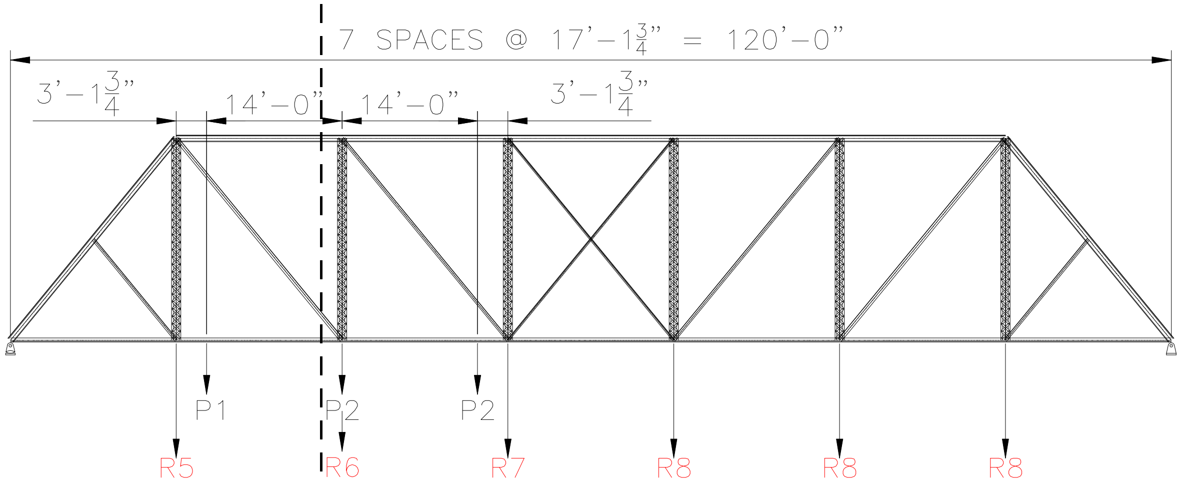
$$R_6 := (1 + IM) \cdot 2P_2 \cdot \left(\frac{3ft + 1.75in}{L_{panel\_1}} + 1 \right) + (1 + IM) \cdot 2P_1 \cdot \left(\frac{3ft + 1.75in}{L_{panel\_1}} \right) + (1 + IM) \cdot LL_{lane} \cdot L_{panel\_1} + w_{ped\_LL} \cdot L_{pan}$$

$$R_6 = 64.45 \cdot k$$

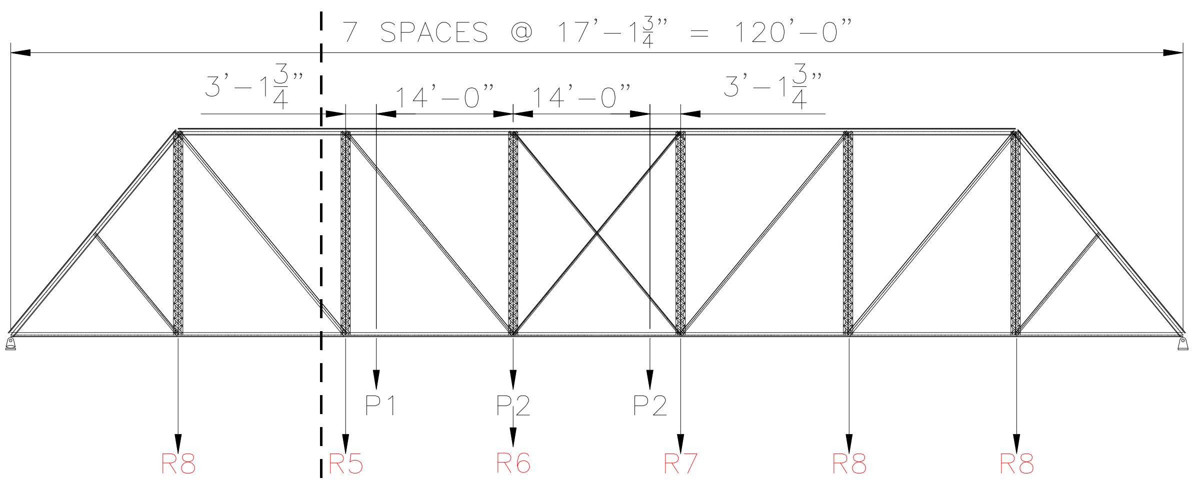
$$R_7 := (1 + IM) \cdot 2P_2 \cdot \left(\frac{L_{panel\_1} - 3ft - 1.75in}{L_{panel\_1}} \right) + (1 + IM) \cdot LL_{lane} \cdot L_{panel\_1} + w_{ped\_LL} \cdot L_{panel\_1} \cdot 5ft = 46.$$

$$R_8 := (1 + IM) \cdot LL_{lane} \cdot L_{panel\_1} + w_{ped\_LL} \cdot L_{panel\_1} \cdot 5ft = 12.13 \cdot k$$

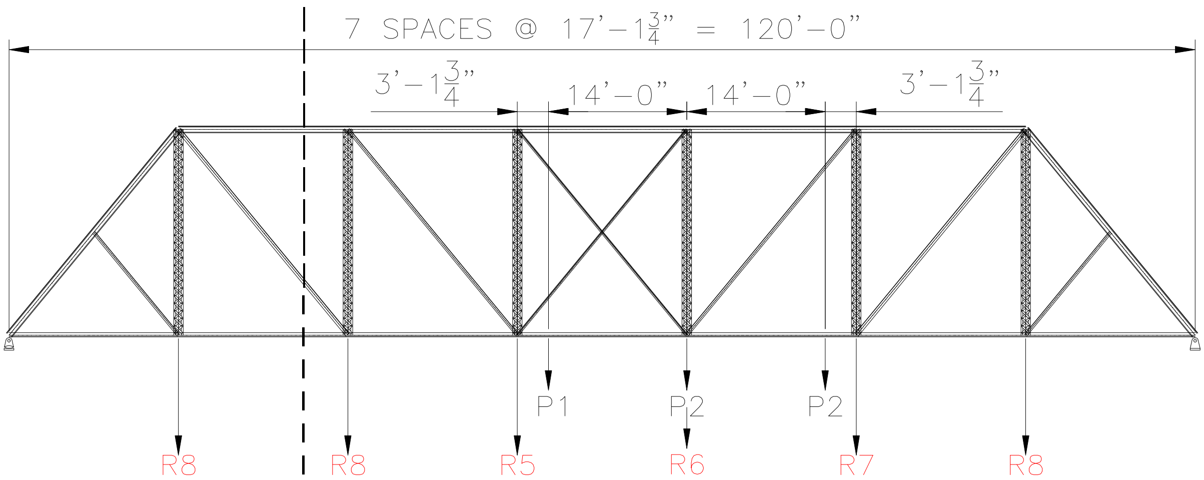
Case 1:



Case 2:



Case 3:



Truss forces analyzed in STAAD per AASHTO LRFR Strength I

							Allowable Comp.	Allowable Tension	ACTUAL LOAD	
			A_g	L	r		$\phi_c P_n$	$\phi_y F_y A_g$	$?_i P_u$	
			(in^2)	(in)	(in)	KL/r	k	k	k	S.R.
2C9x20 (w/ 3/8x15 PL)	L0-U1	TC	16.4	162.66	3.44	47.28	402.13	468.26	337.9	0.84
2C9x13.25 (w/ 3/8x15 PL)	U1-U2	TC	12.6	205.75	3.56	57.79	293.09	358.25	374.5	1.28
2C9x13.25 (w/ 3/8x15 PL)	U2-U3	TC	12.6	205.75	3.56	57.79	293.09	358.25	448.7	1.53
2C9x13.25 (w/ 3/8x15 PL)	U3-U4	TC	12.6	205.75	3.56	57.79	293.09	358.25	437.6	1.49
2L5x3.5x9/16	L0-L1	BC	11.6	205.75	0.97	212.11	58.19	330.60	-213.7	0.65
2L5x3.5x9/16	L1-L2	BC	11.6	205.75	0.97	212.11	58.19	330.60	-213.7	0.65
2L5x3.5x5/16 + 2L5x3.5x3/8	L2-L3	BC	11.2	205.75	1.02	201.72	62.23	319.77	-374.5	1.17
4L5x3.5x7/16	L3-L4	BC	10.2	205.75	1.02	201.72	56.80	291.84	-445.5	1.53
2C8x11.25 (w/ 2"x1/4" lacing)	L1-U1	vert	6.61	252.00	3.16	79.75	134.98	188.39	-64.7	0.34
2C8x11.25 (w/ 2"x1/4" lacing)	L2-U2	vert	6.61	252.00	3.16	79.75	134.98	188.39	128	0.95
2C8x11.25 (w/ 2"x1/4" lacing)	L3-U3	vert	6.61	252.00	3.16	79.75	134.98	188.39	22.9	0.17
2L6x3.5x7/16	U1-L2	diag.	9.04	325.31	0.97	335.37	18.14	257.64	-259.9	1.01
2L5x3x5/16	U2-L3	diag.	4.81	325.31	0.85	382.72	7.41	137.09	-165.1	1.20
2L2.5x2.5x1/4	U3-L4	diag.	2.37	162.66	0.76	214.02	11.68	67.55	-40.7	0.60

Member L3-L4 controls

AASHTO LRFR Design Load Rating:

L3-L4 LL

$$\gamma LL_{truss\_4} := 1.75 \cdot 174.8\mathbf{k}$$

Inventory Rating Factor

$$INV_{truss\_4} := 1 \cdot \frac{C_{truss\_2} - \gamma DL_{truss\_2}}{\gamma LL_{truss\_4}} = 0.27$$

Operating Rating Factor

$$OPR_{truss\_4} := 1 \cdot \frac{C_{truss\_2} - \gamma DL_{truss\_2}}{\gamma LL_{truss\_4} \cdot \left(\frac{1.35}{1.75}\right)} = 0.35$$

L

BRIDGE RATINGFOR VEHICULAR USE:

$\mathbf{INV_{vehicle}} := \mathbf{min}\left(\mathbf{INV_{FB\_2}}, \mathbf{INV_{strngr\_2}}, \mathbf{INV_{truss\_3}}, \mathbf{INV_{truss\_4}}\right)$

Inventory Rating Factor

$\mathbf{INV_{vehicle}} = 0.27$

$\mathbf{OPR_{vehicle}} := \mathbf{min}\left(\mathbf{OPR_{FB\_2}}, \mathbf{OPR_{strngr\_2}}, \mathbf{OPR_{truss\_3}}, \mathbf{OPR_{truss\_4}}\right)$

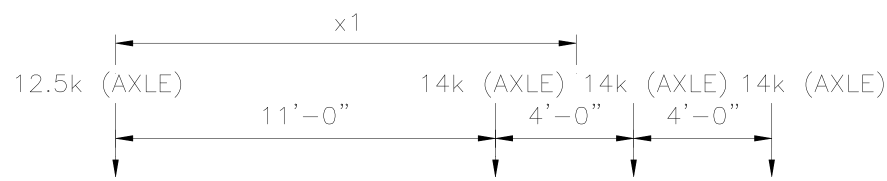
Operating Rating Factor

$\mathbf{OPR_{vehicle}} = 0.35$

This rating is 73% below AASHTO LRFD design live load at the Inventory level. Legal loads should be evaluated.

<u>Legal Loads</u>

Type 4 Truck

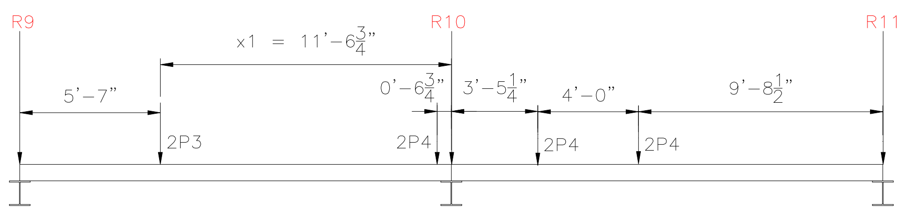


Wheel loads

$\mathbf{P_3} := 0.5 \cdot 12.5\mathbf{k} = 6250\,\mathbf{lb} \qquad \mathbf{P_4} := 0.5 \cdot 14\mathbf{k} = 7000\,\mathbf{lb}$

Resultant distance

$$\mathbf{x_1} := \frac{\mathbf{P_3} \cdot 0\mathbf{ft}}{27.25\mathbf{k}} + \frac{\mathbf{P_4} \cdot 11\mathbf{ft}}{27.25\mathbf{k}} + \frac{\mathbf{P_4} \cdot 15\mathbf{ft}}{27.25\mathbf{k}} + \frac{\mathbf{P_4} \cdot 19\mathbf{ft}}{27.25\mathbf{k}} = 11.5596\,\mathbf{ft}$$



$$\mathbf{R_9} := (1 + \mathbf{IM}) \cdot \left[2\mathbf{P_3} \cdot \left(\frac{\mathbf{L_{panel\_1}} - 5\mathbf{ft} - 7\mathbf{in}}{\mathbf{L_{panel\_1}}}\right) + 2\mathbf{P_4} \cdot \left(\frac{6.75\mathbf{in}}{\mathbf{L_{panel\_1}}}\right)\right] + \mathbf{w_{ped\_LL}} \cdot \mathbf{L_{panel\_1}} \cdot 5\mathbf{ft} = 1$$

$$\mathbf{R_{10}} := (1 + \mathbf{IM}) \cdot \left[2\mathbf{P_3} \cdot \left(\frac{\mathbf{x_1}}{\mathbf{L_{panel\_1}}}\right) + 2\mathbf{P_4} \cdot \left[\left(\frac{\mathbf{L_{panel\_1}} - 6.75\mathbf{in}}{\mathbf{L_{panel\_1}}}\right) + \left(\frac{\mathbf{L_{panel\_1}} - 3\mathbf{ft} - 5.25\mathbf{in}}{\mathbf{L_{panel\_1}}}\right) + \left(\frac{9\mathbf{ft} + 8.5\mathbf{in}}{\mathbf{L_{panel\_1}}}\right)\right]\right] + \mathbf{w_{ped}}$$

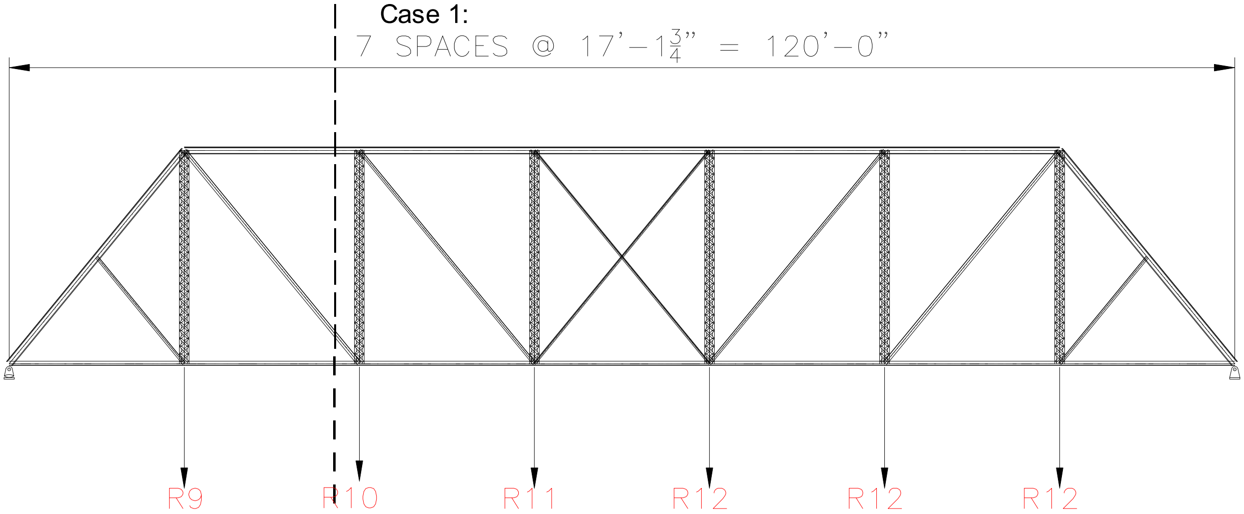
$\mathbf{R_{10}} = 62.36 \cdot \mathbf{k}$

$$\mathbf{R_{11}} := (1 + \mathbf{IM}) \cdot \left[2\mathbf{P_4} \cdot \left(\frac{\mathbf{L_{panel\_1}} - 9\mathbf{ft} - 8.5\mathbf{in}}{\mathbf{L_{panel\_1}}}\right) + 2\mathbf{P_4} \cdot \left(\frac{3\mathbf{ft} + 5.25\mathbf{in}}{\mathbf{L_{panel\_1}}}\right)\right] + \mathbf{w_{ped\_LL}} \cdot \mathbf{L_{panel\_1}} \cdot 5\mathbf{ft} = 19.53 \cdot \mathbf{l}$$

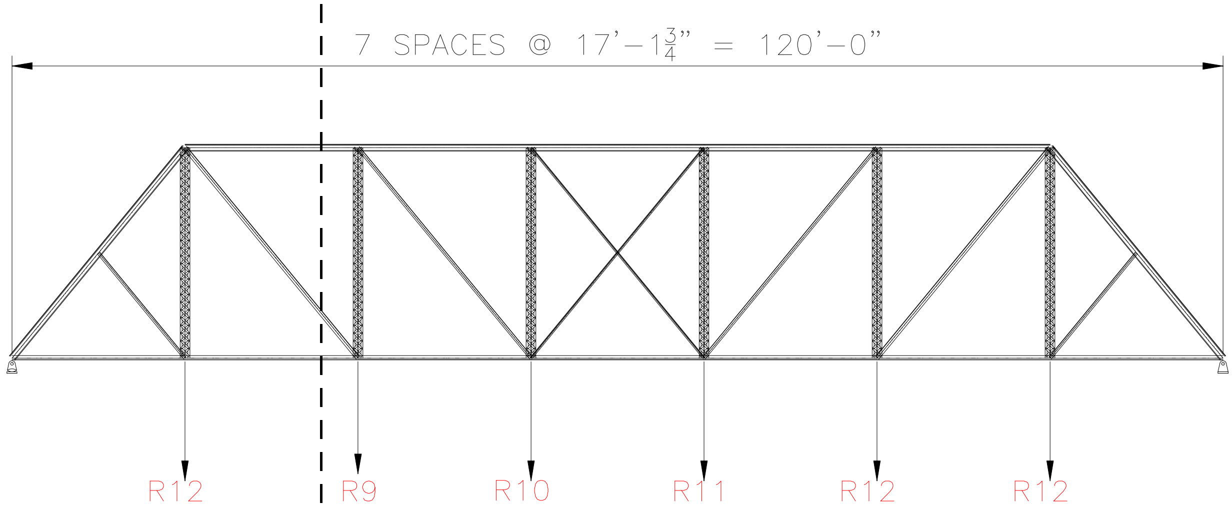
$\mathbf{R_{12}} := \mathbf{w_{ped\_LL}} \cdot \mathbf{L_{panel\_1}} \cdot 5\mathbf{ft} = 7.72 \cdot \mathbf{k}$

West Truss

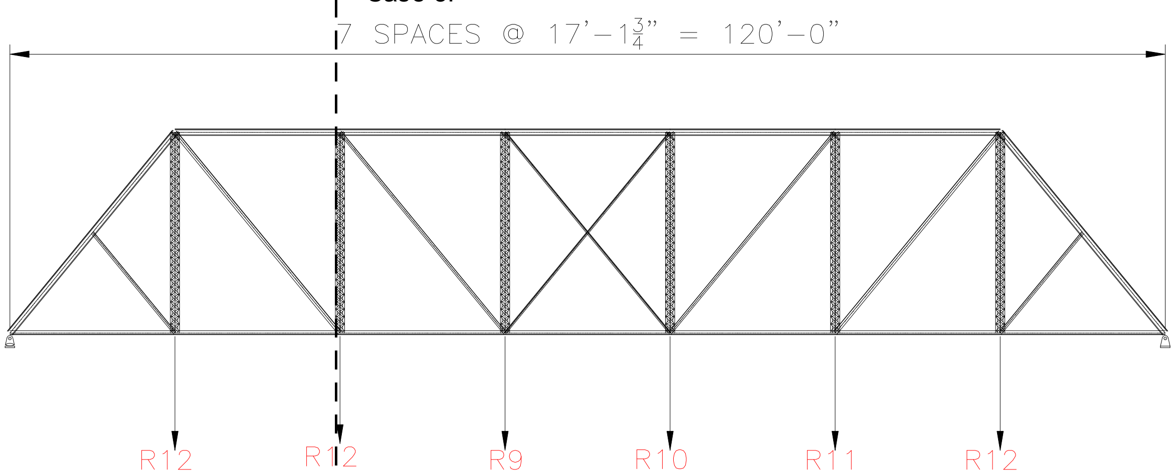
Case 1:



Case 2:



Case 3:



Truss forces analyzed in STAAD per AASHTO LRFR Strength I

							Allowable Comp.	Allowable Tension	ACTUAL LOAD	
			A_g	L	r		$\phi_c P_n$	$\phi_y F_y A_g$	?$_i$P$_u$	
			(in^2)	(in)	(in)	KL/r	k	k	k	S.R.
2C9x20 (w/ 3/8x15 PL)	L0-U1	TC	16.4	162.66	3.44	47.28	402.13	468.26	288.1	0.72
2C9x13.25 (w/ 3/8x15 PL)	U1-U2	TC	12.6	205.75	3.56	57.79	293.09	358.25	313.4	1.07
2C9x13.25 (w/ 3/8x15 PL)	U2-U3	TC	12.6	205.75	3.56	57.79	293.09	358.25	379.4	1.29
2C9x13.25 (w/ 3/8x15 PL)	U3-U4	TC	12.6	205.75	3.56	57.79	293.09	358.25	360.6	1.23
2L5x3.5x9/16	L0-L1	BC	11.6	205.75	0.97	212.11	58.19	330.60	-182.2	0.55
2L5x3.5x9/16	L1-L2	BC	11.6	205.75	0.97	212.11	58.19	330.60	-182.2	0.55
2L5x3.5x5/16 + 2L5x3.5x3/8	L2-L3	BC	11.2	205.75	1.02	201.72	62.23	319.77	-313.4	0.98
4L5x3.5x7/16	L3-L4	BC	10.2	205.75	1.02	201.72	56.80	291.84	-367.1	1.26
2C8x11.25 (w/ 2"x1/4" lacing)	L1-U1	vert	6.61	252.00	3.16	79.75	134.98	188.39	-62.5	0.33
2C8x11.25 (w/ 2"x1/4" lacing)	L2-U2	vert	6.61	252.00	3.16	79.75	134.98	188.39	100.5	0.74
2C8x11.25 (w/ 2"x1/4" lacing)	L3-U3	vert	6.61	252.00	3.16	79.75	134.98	188.39	16.2	0.12
2L6x3.5x7/16	U1-L2	diag.	9.04	325.31	0.97	335.37	18.14	257.64	-208.9	0.81
2L5x3x5/16	U2-L3	diag.	4.81	325.31	0.85	382.72	7.41	137.09	-129.7	0.95
2L2.5x2.5x1/4	U3-L4	diag.	2.37	162.66	0.76	214.02	11.68	67.55	-29.7	0.44

Member U2-U3 controls

AASHTO Type 4 Truck Load Rating:

U2-U3 Capacity

$$C_{truss\_5} := 0.85 \cdot 293.09\mathbf{k}$$

U2-U3 DL

$$\gamma DL_{truss\_5} := 1.25 \cdot 110.7\mathbf{k}$$

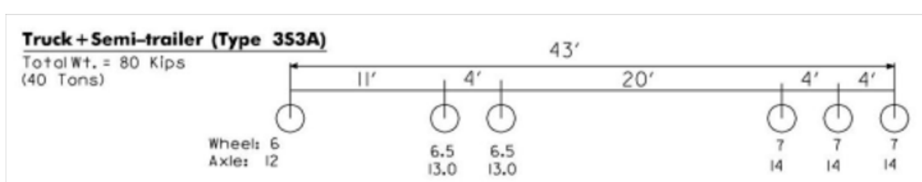
U2-U3 LL

$$\gamma LL_{truss\_5} := 1.75 \cdot 137.7\mathbf{k}$$

Type 4 Truck Rating

$$Type\_4 := 27.25\,ton \cdot \frac{C_{truss\_5} - \gamma DL_{truss\_5}}{\gamma LL_{truss\_5}} = 12.5 \cdot ton$$

Type 3S3 Truck

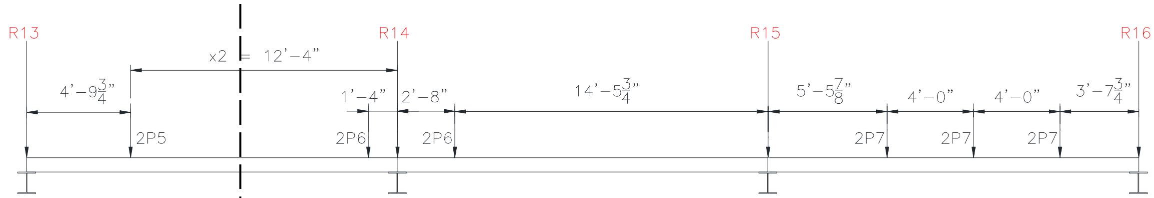


Wheel loads

$$P_5 := 0.5 \cdot 12\mathbf{k} = 6 \cdot \mathbf{k} \qquad P_6 := 0.5 \cdot 13\mathbf{k} = 6.5 \cdot \mathbf{k} \qquad P_7 := 0.5 \cdot 14\mathbf{k} = 7 \cdot \mathbf{k}$$

Resultant distance

$$x_2 := \frac{P_5 \cdot 0ft}{80\mathbf{k}} + \frac{P_6 \cdot 11ft}{80\mathbf{k}} + \frac{P_6 \cdot 15ft}{80\mathbf{k}} + \frac{P_7 \cdot (35ft + 39ft + 43ft)}{80\mathbf{k}} = 12.35\,ft$$



$$R_{13} := (1 + IM)\cdot\left[2P_5\cdot\left(\frac{x_2}{L_{panel\_1}}\right) + 2P_6\cdot\left(\frac{6.75in}{L_{panel\_1}}\right)\right] + w_{ped\_LL}\cdot L_{panel\_1}\cdot 5ft = 19.78\cdot k$$

$$R_{14} := (1 + IM)\cdot\left[2P_5\cdot\left(\frac{x_2}{L_{panel\_1}}\right) + 2P_6\cdot\left[\left(\frac{L_{panel\_1} - 16in}{L_{panel\_1}}\right) + \left(\frac{L_{panel\_1} - 2ft - 8in}{L_{panel\_1}}\right)\right]\right] + w_{ped\_LL}\cdot L_{panel\_1}\cdot 5ft = 49.$$

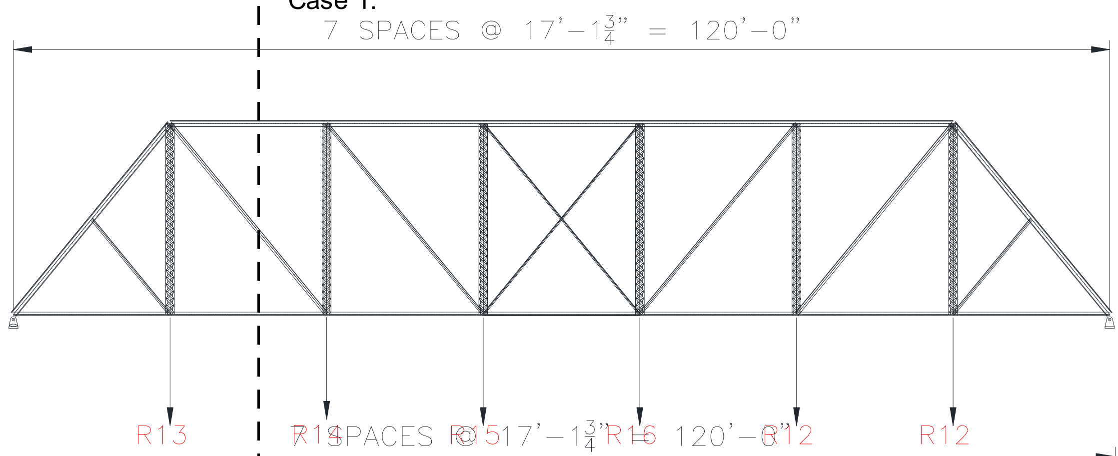
$$R_{15} := (1 + IM)\cdot\left[2P_6\cdot\left(\frac{2ft + 8in}{L_{panel\_1}}\right) + 2P_7\cdot\left[\left(\frac{L_{panel\_1} - 13ft - 5.875in}{L_{panel\_1}}\right) + \left(\frac{L_{panel\_1} - 5ft - 5.875in}{L_{panel\_1}}\right) + \left(\frac{L_{panel\_1} -}{L_p}\right.\right.\right.$$

$$R_{15} = 35.36\cdot k$$

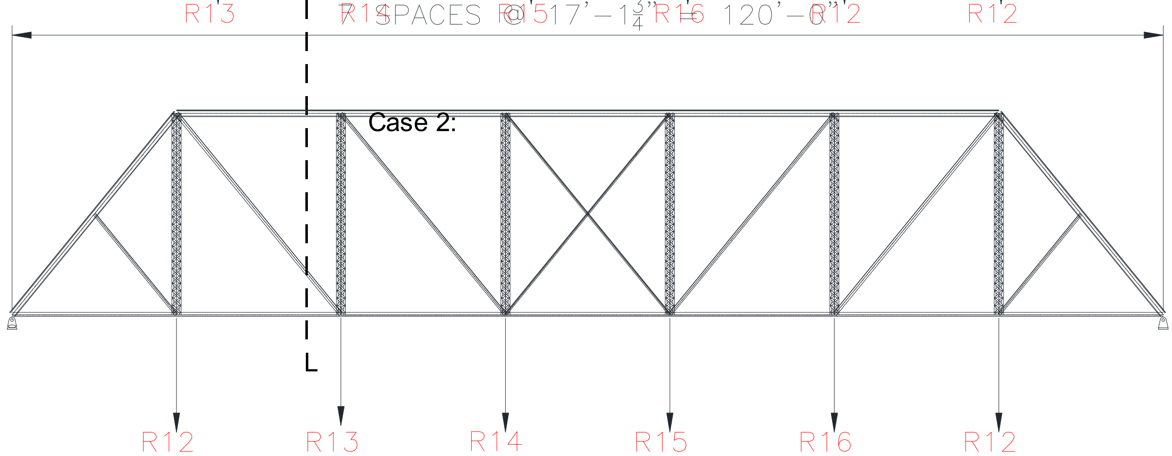
$$R_{16} := (1 + IM)\cdot\left[2P_7\cdot\left[\left(\frac{13ft + 5.875in}{L_{panel\_1}}\right) + \left(\frac{5ft + 5.875in}{L_{panel\_1}}\right) + \left(\frac{9ft + 5.875in}{L_{panel\_1}}\right)\right]\right] + w_{ped\_LL}\cdot L_{panel\_1}\cdot 5ft = 38.63\cdot k$$

<u>West Truss</u>

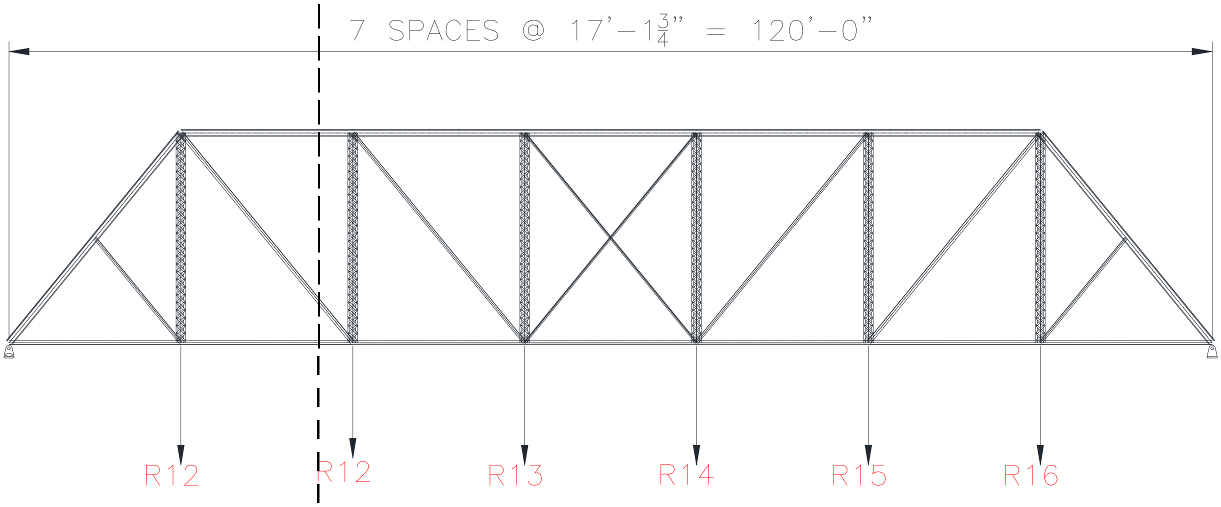
Case 1:



Case 2:



Case 3:



Truss forces analyzed in STAAD per AASHTO LRFR Strength I

			A_g (in^2)	L (in)	r (in)	KL/r	Allowable Comp. $\phi_c P_n$ k	Allowable Tension $\phi_y F_y A_g$ k	ACTUAL LOAD ?$_i P_u$ k	S.R.
2C9x20 (w/ 3/8x15 PL)	L0-U1	TC	16.4	162.66	3.44	47.28	402.13	468.26	318.6	0.79
2C9x13.25 (w/ 3/8x15 PL)	U1-U2	TC	12.6	205.75	3.56	57.79	293.09	358.25	351.6	1.20
2C9x13.25 (w/ 3/8x15 PL)	U2-U3	TC	12.6	205.75	3.56	57.79	293.09	358.25	415.9	1.42
2C9x13.25 (w/ 3/8x15 PL)	U3-U4	TC	12.6	205.75	3.56	57.79	293.09	358.25	411.6	1.40
2L5x3.5x9/16	L0-L1	BC	11.6	205.75	0.97	212.11	58.19	330.60	-201.5	0.61
2L5x3.5x9/16	L1-L2	BC	11.6	205.75	0.97	212.11	58.19	330.60	-201.5	0.61
2L5x3.5x5/16 + 2L5x3.5x3/8	L2-L3	BC	11.2	205.75	1.02	201.72	62.23	319.77	-351.6	1.10
4L5x3.5x7/16	L3-L4	BC	10.2	205.75	1.02	201.72	56.80	291.84	-419	1.44
2C8x11.25 (w/ 2"x1/4" lacing)	L1-U1	vert	6.61	252.00	3.16	79.75	134.98	188.39	-94.9	0.50
2C8x11.25 (w/ 2"x1/4" lacing)	L2-U2	vert	6.61	252.00	3.16	79.75	134.98	188.39	116.6	0.86
2C8x11.25 (w/ 2"x1/4" lacing)	L3-U3	vert	6.61	252.00	3.16	79.75	134.98	188.39	17.9	0.13
2L6x3.5x7/16	U1-L2	diag.	9.04	325.31	0.97	335.37	18.14	257.64	-239.6	0.93
2L5x3x5/16	U2-L3	diag.	4.81	325.31	0.85	382.72	7.41	137.09	-150.5	1.10
2L2.5x2.5x1/4	U3-L4	diag.	2.37	162.66	0.76	214.02	11.68	67.55	-33.8	0.50

Member L3-L4 controls

AASHTO Type 3S3 Truck Load Rating:

L3-L4 Capacity
$$C_{truss\_6} := 0.85 \cdot 291.84\mathbf{k}$$

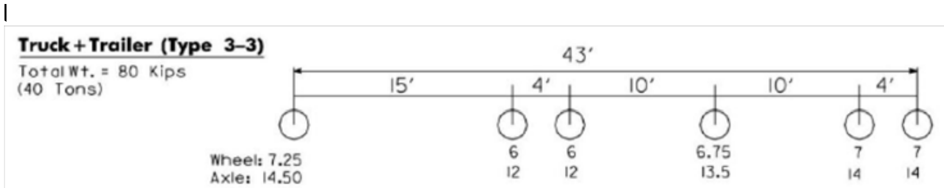
L3-L4 DL
$$\gamma DL_{truss\_6} := 1.25 \cdot 111.7\mathbf{k}$$

L3-L4 LL
$$\gamma LL_{truss\_6} := 1.75 \cdot \mathbf{159.6k}$$

Type 3S3 Truck Rating
$$Type\_3S3 := 40\mathbf{ton} \cdot \frac{C_{truss\_6} - \gamma DL_{truss\_6}}{\gamma LL_{truss\_6}} = 15.5 \cdot \mathbf{ton}$$

L

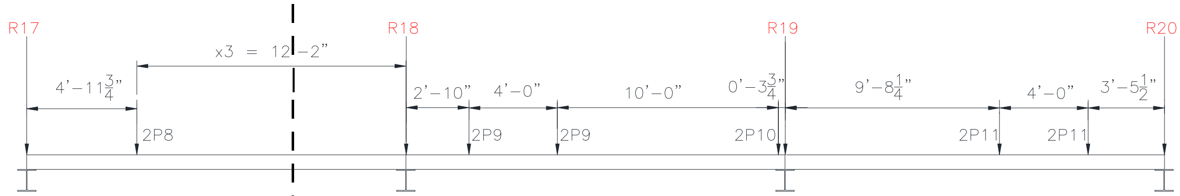
Type 3-3 Truck



Wheel loads $\mathbf{P_8} := 0.5 \cdot 14.5\mathbf{k} = 7.25 \cdot \mathbf{k}$ $\mathbf{P_9} := 0.5 \cdot 12\mathbf{k} = 6 \cdot \mathbf{k}$ $\mathbf{P_{10}} := 0.5 \cdot 13.5\mathbf{k} = 6.75 \cdot \mathbf{k}$ $\mathbf{P_{11}} := 0.5 \cdot 14\mathbf{k} = 7 \cdot \mathbf{k}$

Resultant distance

$$\mathbf{x_3} := \frac{\mathbf{P_8} \cdot 0\mathbf{ft}}{80\mathbf{k}} + \frac{\mathbf{P_9} \cdot (15\mathbf{ft} + 19\mathbf{ft})}{80\mathbf{k}} + \frac{\mathbf{P_{10}} \cdot 29\mathbf{ft}}{80\mathbf{k}} + \frac{\mathbf{P_{11}} \cdot (39\mathbf{ft} + 43\mathbf{ft})}{80\mathbf{k}} = 12.17\,\mathbf{ft}$$



$$\mathbf{R_{17}} := (1 + \mathbf{IM}) \cdot \left[2\mathbf{P_8} \cdot \left(\frac{\mathbf{x_3}}{\mathbf{L_{panel\_1}}} \right) \right] + \mathbf{w_{ped\_LL}} \cdot \mathbf{L_{panel\_1}} \cdot 5\mathbf{ft} = 21.4 \cdot \mathbf{k}$$

$$\mathbf{R_{18}} := (1 + \mathbf{IM}) \cdot \left[2\mathbf{P_8} \cdot \left(\frac{\mathbf{L_{panel\_1}} - \mathbf{x_3}}{\mathbf{L_{panel\_1}}} \right) + 2\mathbf{P_9} \cdot \left[\left(\frac{\mathbf{L_{panel\_1}} - 34\mathbf{in}}{\mathbf{L_{panel\_1}}} \right) + \left(\frac{\mathbf{L_{panel\_1}} - 6\mathbf{ft} - 10\mathbf{in}}{\mathbf{L_{panel\_1}}} \right) \right] + 2\mathbf{P_{10}} \cdot \left(\frac{3.75\mathbf{in}}{\mathbf{L_{panel\_1}}} \right) \right] + \cdots$$

$$\mathbf{R_{18}} = 36.57 \cdot \mathbf{k}$$

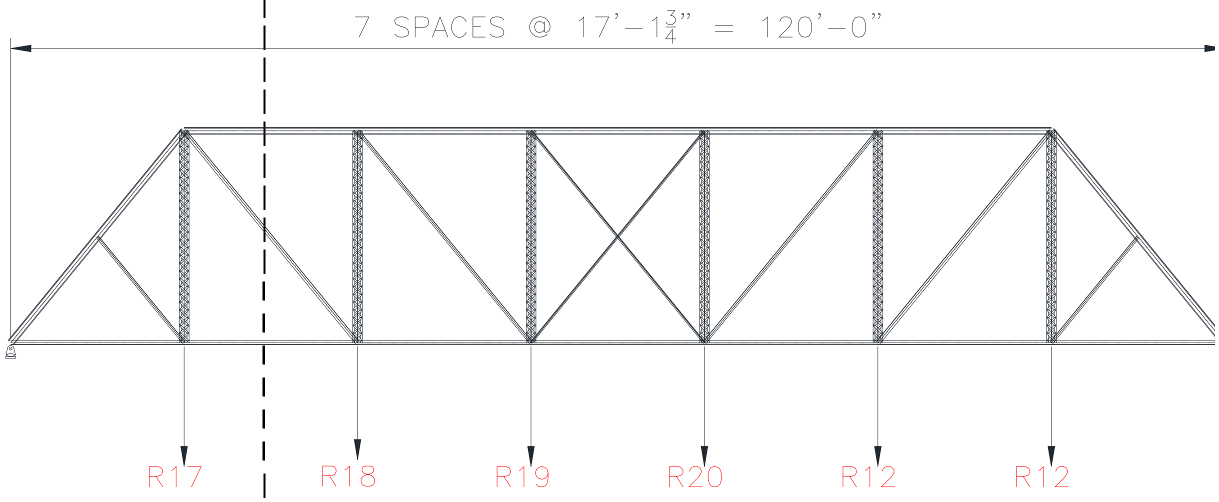
$$\mathbf{R_{19}} := (1 + \mathbf{IM}) \cdot \left[2\mathbf{P_{11}} \cdot \left[\left(\frac{\mathbf{L_{panel\_1}} - 9\mathbf{ft} - 8.25\mathbf{in}}{\mathbf{L_{panel\_1}}} \right) + \left(\frac{\mathbf{L_{panel\_1}} - 13\mathbf{ft} - 8.25\mathbf{in}}{\mathbf{L_{panel\_1}}} \right) \right] + 2\mathbf{P_9} \cdot \left[\left(\frac{34\mathbf{in}}{\mathbf{L_{panel\_1}}} \right) + \left(\frac{6\mathbf{ft} + 10\mathbf{in}}{\mathbf{L_{panel\_1}}} \right) \cdots \right.\right.$$

$$\mathbf{R_{19}} = 46.2 \cdot \mathbf{k}$$

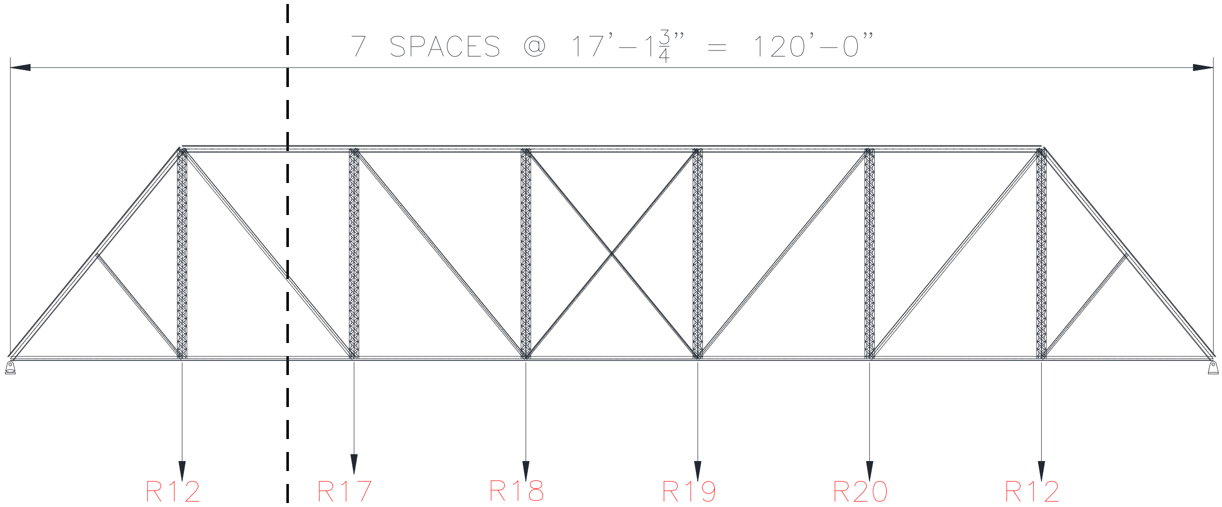
$$\mathbf{R_{20}} := (1 + \mathbf{IM}) \cdot \left[2\mathbf{P_{11}} \cdot \left[\left(\frac{9\mathbf{ft} + 8.25\mathbf{in}}{\mathbf{L_{panel\_1}}} \right) + \left(\frac{13\mathbf{ft} + 8.25\mathbf{in}}{\mathbf{L_{panel\_1}}} \right) \right] \right] + \mathbf{w_{ped\_LL}} \cdot \mathbf{L_{panel\_1}} \cdot 5\mathbf{ft} = 3$$

West Truss

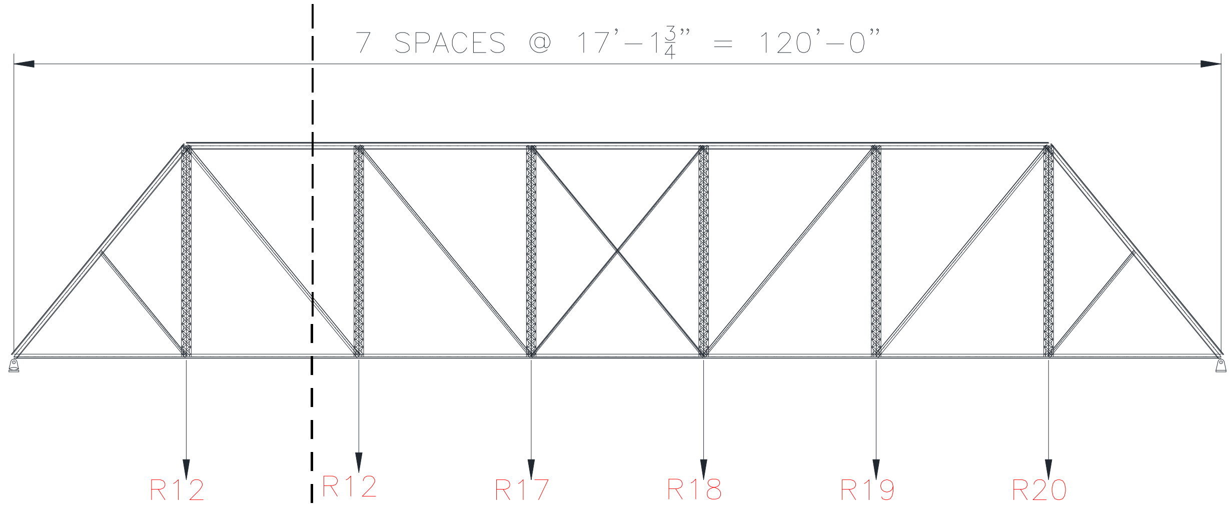
Case 1:



Case 2:



Case 3:



Truss forces analyzed in STAAD per AASHTO LRFR Strength I

							Allowable Comp.	Allowable Tension	ACTUAL LOAD	
			A_g	L	r		$\phi_c P_n$	$\phi_y F_y A_g$	?$_i$P$_u$	
			(in^2)	(in)	(in)	KL/r	k	k	k	S.R.
2C9x20 (w/ 3/8x15 PL)	L0-U1	TC	16.4	162.66	3.44	47.28	402.13	468.26	320.3	0.80
2C9x13.25 (w/ 3/8x15 PL)	U1-U2	TC	12.6	205.75	3.56	57.79	293.09	358.25	335	1.14
2C9x13.25 (w/ 3/8x15 PL)	U2-U3	TC	12.6	205.75	3.56	57.79	293.09	358.25	405	1.38
2C9x13.25 (w/ 3/8x15 PL)	U3-U4	TC	12.6	205.75	3.56	57.79	293.09	358.25	397.1	1.35
2L5x3.5x9/16	L0-L1	BC	11.6	205.75	0.97	212.11	58.19	330.60	-202.3	0.61
2L5x3.5x9/16	L1-L2	BC	11.6	205.75	0.97	212.11	58.19	330.60	-202.3	0.61
2L5x3.5x5/16 + 2L5x3.5x3/8	L2-L3	BC	11.2	205.75	1.02	201.72	62.23	319.77	-335	1.05
4L5x3.5x7/16	L3-L4	BC	10.2	205.75	1.02	201.72	56.80	291.84	-408.6	1.40
2C8x11.25 (w/ 2"x1/4" lacing)	L1-U1	vert	6.61	252.00	3.16	79.75	134.98	188.39	-85.3	0.45
2C8x11.25 (w/ 2"x1/4" lacing)	L2-U2	vert	6.61	252.00	3.16	79.75	134.98	188.39	113.1	0.84
2C8x11.25 (w/ 2"x1/4" lacing)	L3-U3	vert	6.61	252.00	3.16	79.75	134.98	188.39	14.6	0.11
2L6x3.5x7/16	U1-L2	diag.	9.04	325.31	0.97	335.37	18.14	257.64	-233	0.90
2L5x3x5/16	U2-L3	diag.	4.81	325.31	0.85	382.72	7.41	137.09	-146	1.07
2L2.5x2.5x1/4	U3-L4	diag.	2.37	162.66	0.76	214.02	11.68	67.55	-29.1	0.43

Member L3-L4 controls

AASHTO Type 3-3 Truck Load Rating:

L3-L4 Capacity

$$C_{truss\_7} := 0.85 \cdot 291.84\mathbf{k}$$

L3-L4 DL

$$\gamma DL_{truss\_7} := 1.25 \cdot 111.7\mathbf{k}$$

L3-L4 LL

$$\gamma LL_{truss\_7} := 1.75 \cdot \mathbf{153.7k}$$

Type 3-3 Truck Rating

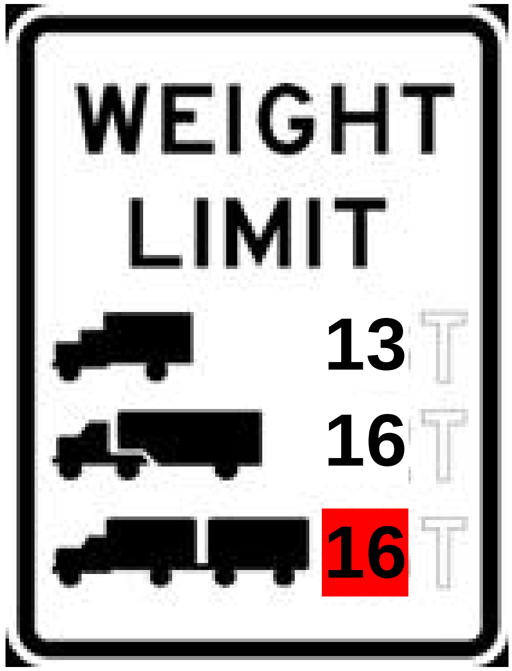
$$Type\_3\_3 := 40\mathbf{ton} \cdot \frac{C_{truss\_7} - \gamma DL_{truss\_7}}{\gamma LL_{truss\_7}} = 16.1 \cdot \mathbf{ton}$$

Legal Load Rating Summary

Type_4 = 13·**ton**

Type_3S3 = 16·**ton**

Type_3_3 = 16·**ton**



SIDEWALK ONLY

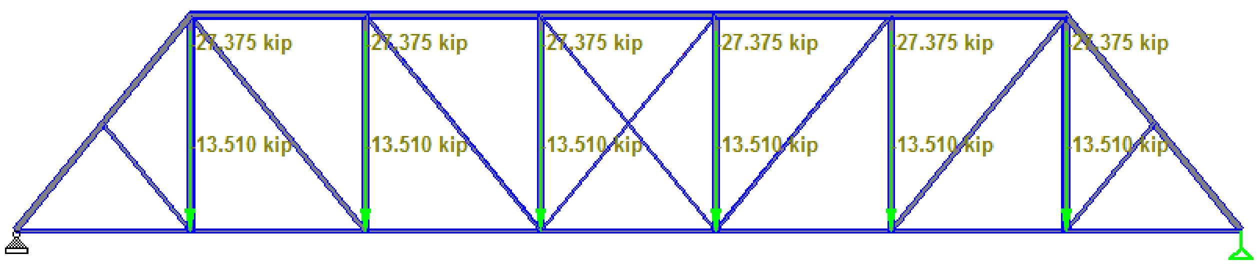
Pedestrian Live Load

LL Panel Point Load (west)

$LL_{ped\_3} := 5ft \cdot L_{panel\_1} \cdot w_{ped\_LL} = 7.72 \cdot k$

West Truss

Truss forces analyzed in STAAD per AASHTO LRFD Strength I (factored loads shown)



							Allowable Comp.	Allowable Tension	ACTUAL LOAD	
			A_g (in^2)	L (in)	r (in)	KL/r	$\phi_c P_n$ k	$\phi_y F_y A_g$ k	$?_i P_u$ k	S.R.
2C9x20 (w/ 3/8x15 PL)	L0-U1	TC	16.4	162.66	3.44	47.28	402.13	468.26	161.8	0.40
2C9x13.25 (w/ 3/8x15 PL)	U1-U2	TC	12.6	205.75	3.56	57.79	293.09	358.25	170.5	0.58
2C9x13.25 (w/ 3/8x15 PL)	U2-U3	TC	12.6	205.75	3.56	57.79	293.09	358.25	204.6	0.70
2C9x13.25 (w/ 3/8x15 PL)	U3-U4	TC	12.6	205.75	3.56	57.79	293.09	358.25	202.8	0.69
2L5x3.5x9/16	L0-L1	BC	11.6	205.75	0.97	212.11	58.19	330.60	-102.3	0.31
2L5x3.5x9/16	L1-L2	BC	11.6	205.75	0.97	212.11	58.19	330.60	-102.3	0.31
2L5x3.5x5/16 + 2L5x3.5x3/8	L2-L3	BC	11.2	205.75	1.02	201.72	62.23	319.77	-170.5	0.53
4L5x3.5x7/16	L3-L4	BC	10.2	205.75	1.02	201.72	56.80	291.84	-206.4	0.71
2C8x11.25 (w/ 2"x1/4" lacing)	L1-U1	vert	6.61	252.00	3.16	79.75	134.98	188.39	-40.9	0.22
2C8x11.25 (w/ 2"x1/4" lacing)	L2-U2	vert	6.61	252.00	3.16	79.75	134.98	188.39	40.9	0.30
2C8x11.25 (w/ 2"x1/4" lacing)	L3-U3	vert	6.61	252.00	3.16	79.75	134.98	188.39	2.2	0.02
2L6x3.5x7/16	U1-L2	diag.	9.04	325.31	0.97	335.37	18.14	257.64	-105.6	0.41
2L5x3x5/16	U2-L3	diag.	4.81	325.31	0.85	382.72	7.41	137.09	-52.8	0.39
2L2.5x2.5x1/4	U3-L4	diag.	2.37	162.66	0.76	214.02	11.68	67.55	-2.8	0.04

Member L3-L4 controls

AASHTO LRFR Pedestrian Load Rating:

L3-L4 Capacity

$C_{truss\_8} := \min(0.85, \varphi_{c\_truss\_1} \cdot \varphi_{s\_truss\_1}) \cdot 291.8k$

L3-L4 DL

$\gamma DL_{truss\_8} := 1.25 \cdot 111.7k$

L3-L4 LL

$\gamma LL_{truss\_8} := 1.75 \cdot 38.1k$

Inventory Rating

$$INV_{truss\_8} := w_{ped\_LL} \cdot \frac{C_{truss\_8} - \gamma DL_{truss\_8}}{\gamma LL_{truss\_8}} = 112.85 \cdot psf$$

Operating Rating

$$OPR_{truss\_8} := w_{ped\_LL} \cdot \frac{C_{truss\_8} - \gamma DL_{truss\_8}}{\gamma LL_{truss\_8} \cdot \left(\frac{1.35}{1.75}\right)} = 146.29 \cdot psf$$

Phil Jones

From: Tim McDermott <tmcdermott@vjengineering.com>
Sent: Friday, October 23, 2015 9:42 AM
To: Mike Cherry; CFaber@Whks.com; Phil Jones
Subject: RE: Waverly - 3rd Street SE Bridge

Casey,
Mike wanted me to touch base with you regarding the sidewalk on the 3rd St. Bridge. I called your office this morning and understand that you are gone today but I wanted send this to open a dialogue so we can all get on the same page and give the City some definitive answers.
As I stressed at the Council Meeting following the completion of our report, the widespread deterioration throughout the structure requires mitigation before the roadway portion of the bridge is reopened to vehicular traffic or re-purposed as a pedestrian bridge. This is not necessarily reflected in the analysis portion of our report due to the most advanced deterioration occurring on non-controlling elements (even after taking section loss into account), such as the stringers and floor beams. Each rehabilitation alternative includes addressing areas of measurable section loss on any primary load carrying member.

Regarding the sidewalk being temporarily reopened, this was never intended as a long term solution. In my opinion, keeping the bridge open in this limited capacity for the short term (<1 year) poses little concern for failure. I'm aware that the transverse sidewalk beams are fracture critical members and have advanced section loss, but they do not undergo cyclic loading and most of the section loss is occurring outside of the tensile regions of the beams. The worst case scenario I observed was about 2" of loss at the truss connection; this was analyzed and found to have adequate capacity. I'm certainly open to reviewing your analysis for comparison.

Thanks- Tim

Tim McDermott, P.E.
Structural Engineer

VJ Engineering
2570 Holiday Road, Suite 10
Coralville, IA 52241
Cell: (319) 540-6956
Phone: (319) 338-4939
Fax: (319) 338-9457

Subject: Waverly - 3rd Street SE Bridge
Date: Thu, 22 Oct 2015 10:50:27 -0500
From: mike@ci.waverly.ia.us
To: tmcdermott@vjengineering.com

Tim,

Below is the email we received from Casey Faber and I have highlighted their concern regarding the sidewalk.

Mike Cherry
Waverly City Engineer
319-352-9065

From: Casey Faber [mailto:CFaber@Whks.com]
Sent: Tuesday, October 20, 2015 3:34 PM
To: W D. Werger
Cc: Mike Cherry; Phil Jones; Fouad Daoud; Scott Sweet
Subject: RE: 3rd Street Bridge

Hi William,

First, thank you for contacting us regarding the discussion with the Council and continuing to include WHKS in the discussion of this bridge.

We have reviewed the report by VJ Engineering and it is our opinion that the analysis did not adequately consider the degree of deterioration throughout the structure. We performed an alternate analysis that shows the sidewalk overhang brackets do not have the capacity for the full AASHTO design pedestrian load.

I'm not sure of your familiarity with SIIMS, but as a quick note it provides the owner and inspector a central place to record and manage data concerning structures. Some of this data is required to be reported to the federal government, and is organized on the Structural Inventory and Appraisal (SI&A) form. The items on the SI&A are numbered and describe details about the bridge and it's condition.

On the SI&A Item 42 is coded 5 for the 3rd Street bridge which means it is a vehicle and pedestrian structure. When the bridge was closed in February, Item 41 was changed from P (open, but posted for load) to K which means it is closed to all traffic. Based on our analysis that the structure cannot support legal pedestrian loads and the strict wording of the SI&A items we feel it is in the best interest of the City to keep the bridge closed to all traffic.

Please let us know if you have any questions regarding this information. We appreciate the opportunity to serve you!

Casey V. Faber, P.E.
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Voice: 515.663.9997 I www.whks.com
whks