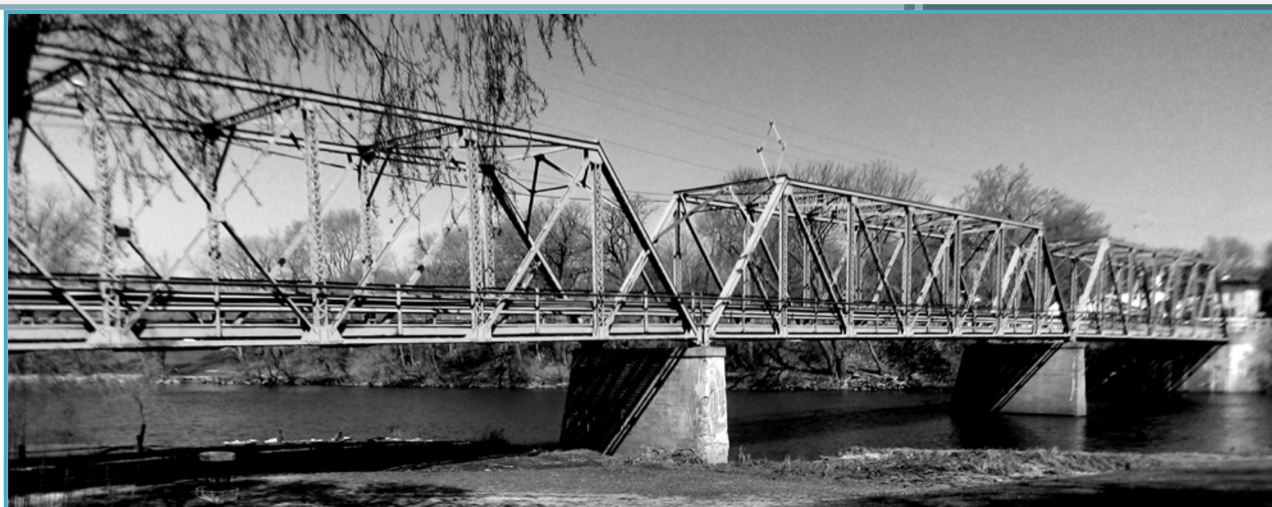
CITY OF WAVERLY

BRIDGE EVALUATION & FEASIBILITY STUDY

3rd STREET SE OVER CEDAR RIVER

FHWA #12250

WAVERLY, IOWA



July 22, 2015

SUBMITTED BY:

VJ Engineering

July 22, 2015

Phil Jones
City Administrator
Waverly, Iowa
(319) 352-9211

Subject: Final Report
3rd St SE Bridge Evaluation & Feasibility Study
City of Waverly, Iowa

Dear Phil Jones,

VJ Engineering (VJE) is pleased to submit this Final Report of the Bridge Evaluation and Feasibility Study performed on the 3rd St. SE Bridge over the Cedar River.

We appreciate you selecting VJ Engineering for this project and look forward to the opportunity to work with you again in the near future. If you have any questions about this report or require additional services, please call me at 319-338-4939.

Sincerely,

Tim McDermott, PE
Structural Engineer/ Project Manager

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TIM J. MCDERMOTT
LICENSED
21396
IOWA
PROFESSIONAL ENGINEER

I HEREBY CERTIFY THAT THIS DOCUMENT WAS PREPARED BY ME OR UNDER MY DIRECT PERSONAL SUPERVISION AND THAT I AM A DULY LICENSED PROFESSIONAL ENGINEER UNDER THE LAWS OF THE STATE OF IOWA.

07/22/15

TIM J. MCDERMOTT, P.E. IOWA REGISTRATION NO. 21396

MY REGISTRATION EXPIRES DECEMBER 31, 2015

PAGES OR DRAWINGS COVERED BY THIS SEAL: ALL

PROJECT BACKGROUND

Bridge Description

The 3rd Street SE Bridge (originally known as the Harmon Street Bridge) is located in Waverly, Iowa and until 2015 carried vehicular traffic across the Cedar River. The bridge is 360' x 18' (with a 5' cantilevered sidewalk on the west side) and is comprised of (3) 120' steel through truss spans. *Figures 1 & 2* show the elevation views of the 2 different truss types (East and West, respectively) used in each span and *Figure 3*

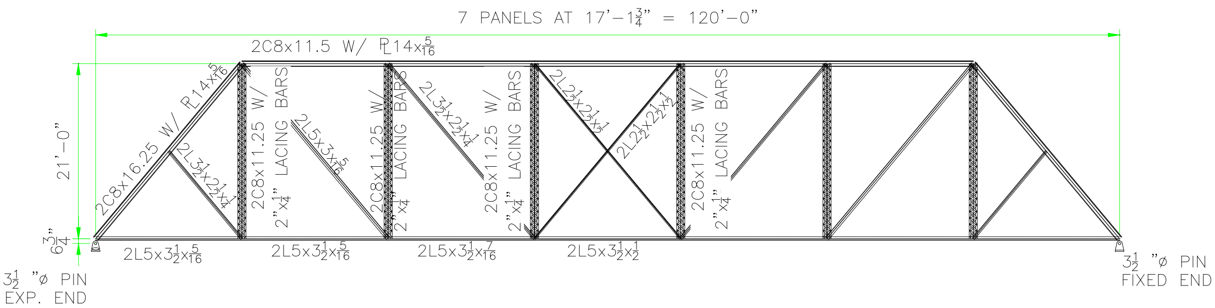


Figure 1 *East Truss Elevation*

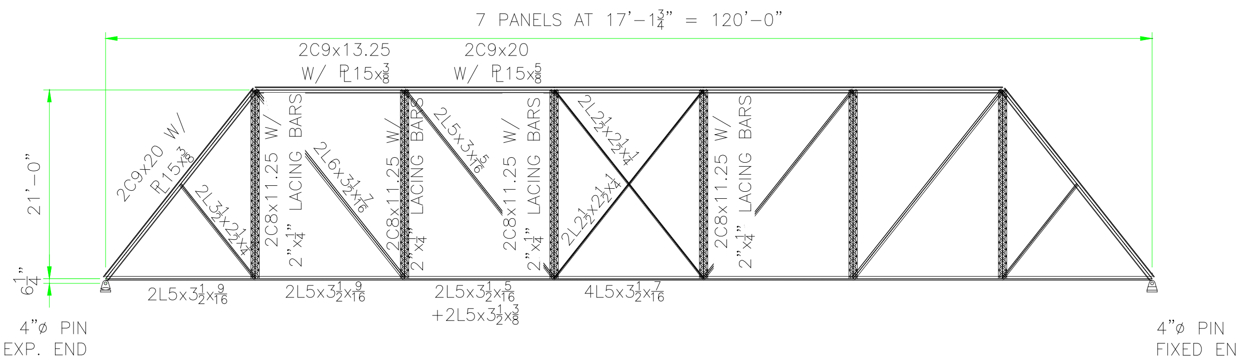


Figure 2 *West Truss Elevation*

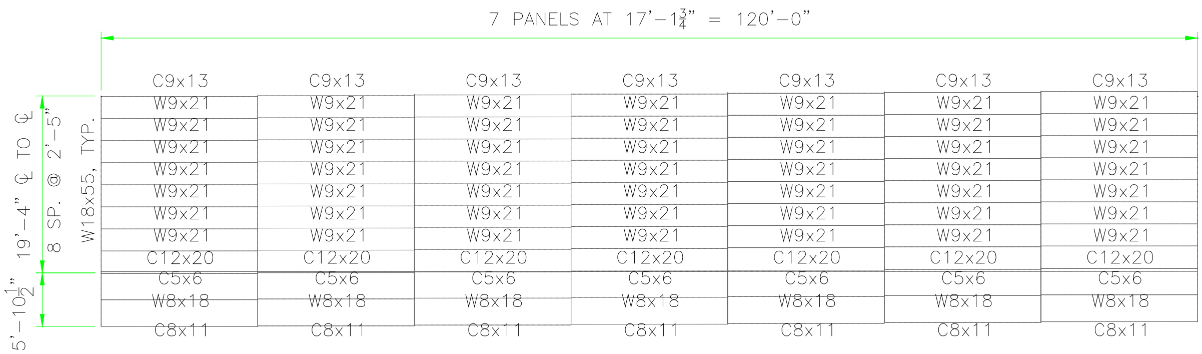


Figure 3 *Floor System Plan*

shows the floor system plan. The deck is open steel grating. The substructure consists of two concrete piers which are founded on spread footings, and concrete abutments at each end founded on spread footings. The bridge was constructed in 1917 and carried vehicular traffic until it was closed in February, 2015 due to advanced deterioration of the superstructure and substructure. The bridge is not currently listed on the National Registry of Historic Places, but is eligible to be listed.

Purpose

The superstructure has significant corrosion and section loss at connection plates and the southwest and northwest bearing connections have failed resulting in settlement of the truss. The bearings were observed to be longitudinally expanded at a low temperature which is the opposite direction they should be which may indicate the bearings have frozen up which prohibits longitudinal movement. Significant section loss was observed on many of the bearing pins. There is heavy pitting and significant section loss on floor beams and stringers and a few crack initiations were observed by the previous inspector. The piers have significant deterioration and spalled areas, particularly near the waterline and pier caps. The abutments are delaminated and have large vertical cracks with efflorescence. The general purpose of this report will be to perform an in depth evaluation and investigate the feasibility of rehabilitating or replacing the structure for either continued vehicular use or to be repurposed as a pedestrian bridge.

In order to determine the extent of the rehabilitation necessary, a structural analysis taking current condition into consideration is required. The structural analysis needs to consider both AASHTO pedestrian design live load and vehicular live loads per AASHTO *Load and Resistance Factor Rating of Highway Bridges (*LRFR). Specific areas which require repair or strengthening shall be noted on a plan and profile drawing.

Measureable section loss, cracking, or other deficiencies which affect load carrying capacities shall be quantified for the purpose of performing the load rating analysis.

To aid in selection of the most appropriate rehabilitation or replacement alternative, the following six alternatives will be investigated:

1. Do nothing.
2. Rehabilitate the bridge and repurpose as a pedestrian bridge.
3. Rehabilitate the bridge for vehicular and pedestrian use.
4. Replace the bridge with a new pedestrian bridge.
5. Replace the bridge "in-kind" for vehicular and pedestrian use.
6. Replace the bridge with a conventional modern bridge for vehicular and pedestrian use.

A cost estimate for the construction of the rehabilitation or replacement and anticipated construction schedule will be prepared for the six alternatives. A lifecycle cost assessment will also be performed for a 20 year design life, taking future maintenance costs into consideration for each alternative.

RECORDS REVIEW

Inspection Reports

Previous Inspection Reports were reviewed and are summarized below. The number in parenthesis is the condition rating given by the previous inspector to each component on the 0-9 NBIS (National Bridge Inspection Standards) rating scale. A rating of 9 is excellent condition and 0 is failed condition.

Deck (7): The deck is in satisfactory condition with some areas showing minor deterioration. The south pier joint cover plate on the top of the deck is loose and is vibrating the deck when traffic crosses.

Superstructure (3): Significant pack rust typical at many connections. Pack rust is causing distortion of plates built up near bearings and bulging of pins. Significant section loss (including through holes) of plates adjacent to the pins, and the connection has failed at the southeast and northwest bearings of the south truss resulting in some settlement of the truss. The southwest bearing is near failure. Two additional plates were welded to the gusset plates directly above the bearing pin, at the east side of the south abutment during the 2006 repair in order to temporarily alleviate the potential for failure. Member U1:L2 on the east side of the south truss has slight sweep (out of plane bending) that is likely due to differential settlement of the truss at the failed bearings. There is section loss on some anchor bolts and nuts are not tight at several locations. The bearings are also tipped outward which is the opposite direction based on the current temperature. At the bottom of the diagonals, pack rust is causing distortion of up to approx. 3/8” of the connection angles and up to approx. 1/8” section loss. Pack rust is causing up to approx. 1/4” distortion of the tie plates on the diagonal members. The repair performed at several verticals along the west side is deteriorating. There is pack rust between the original and repair materials indicating failure of the welds. There is pack rust between the angles in the west bottom chord between panel points two and five causing distortion and section loss. The overhead bracing members have minor pack rust as well. There is a loose bolt at the bottom chord connection to vertical six in the center truss, west side. Several other bottom chord connections have heavy pitting including on the fasteners. There is impact damage to diagonal L4-U5 on the west side of the center truss, diagonal L3-U4 on the east side of the center truss, and minor impact damage to tie plates at other locations. There are several discrete locations of leaf rust and other deformation to tie plates. There is heavy pitting and significant section loss on floor beams and stringers. The flanges of the floor beams have the heaviest loss at the connections to the truss, but much of the section loss is not active and has been painted over. The webs have heavy pack rust and section loss at the connection angles to the stringers. The stringers have significant section loss in the flanges with some through holes. The webs have significant section loss especially at the connections to the floor beams.

There are two stringers in the south truss that have serious section loss at the web connection to the floor beam, one that is cracked and the other with a crack initiating. Many of the locations that were repaired have pack rust between the original repair materials indicating failure of the webs and new section loss. Significant deterioration of the stringer to floor beam connection angles, especially those with fasteners replaced by welds.

Substructure (4): Both abutments have vertical cracks with leaching. The north abutment has a large area that has been previously repaired, but is cracked and leaching again. There is significant delamination and spalling with some reinforcing exposed and corroded. The north back wall is cracked at the roadway adjacent to the bridge and appears to be crumbling. Areas of both piers near the waterline have large spalls, including a large spall in the north pier on the west end below the ice guard. The south pier has significant map cracking with leaching and the east end is spalling. The bridge seats are deteriorating especially on the south pier at the west bearing.

Bridge Plans & Repair History

The bridge plans and repair records were reviewed and determined to provide adequate dimensioning and member details to develop the structural models to be used for the load rating analysis. A site visit was still required to field measure deformations, section loss, and cracking. These deficiencies directly compromise the load carrying capacity of the bridge and were required to be quantified for use in the load rating analysis.

FEMA Flood Insurance Study (FIS)

A FEMA Flood Insurance Study (FIS) for Bremer County performed in 2008 was obtained and reviewed to determine if the bridge currently meets the Iowa DNR's criteria for minimum freeboard (vertical clearance to the low point of the bridge superstructure) of 3 feet above the design flood having a 2% chance of being exceeded in any given year (Q_{50}). Relevant data from the FIS is included in Appendix D. Figure 4 shows the design flood elevations for various design flows. The design elevation, which is based on the Q_{50}, is 907.1 feet.

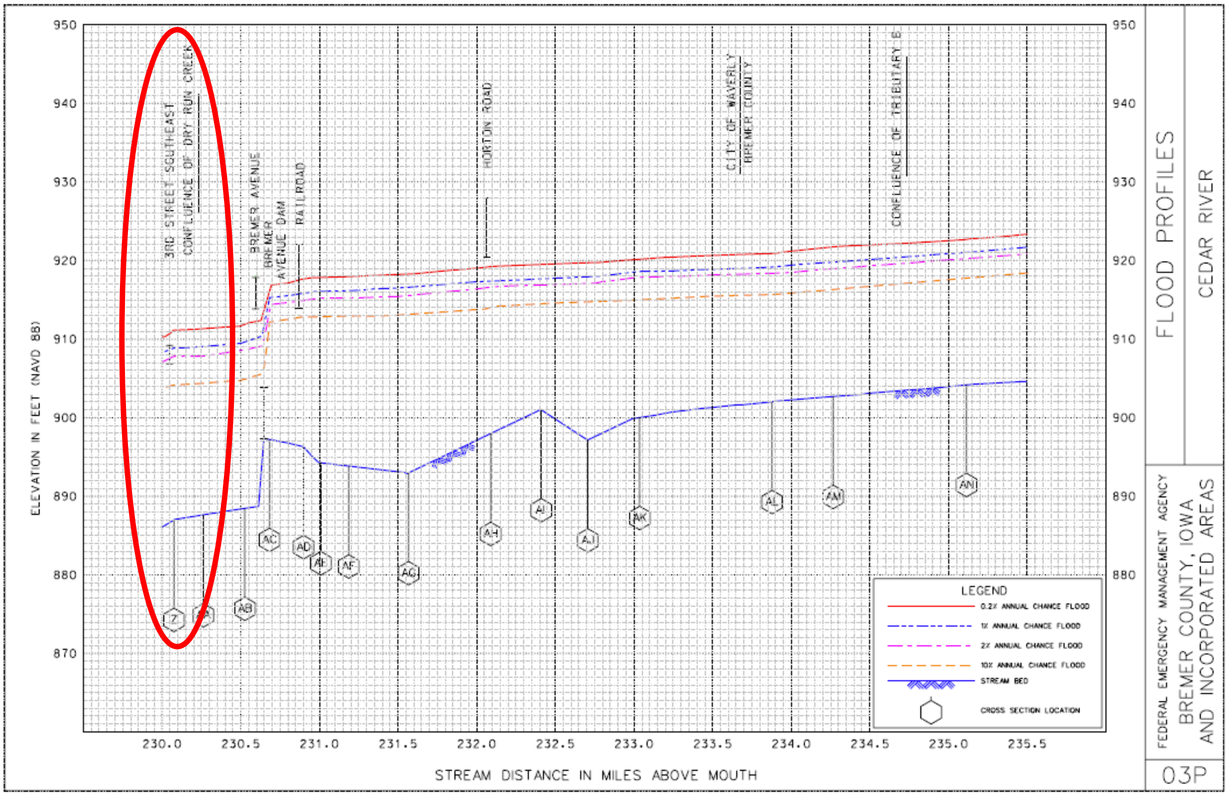


Figure 4 *Flood Profiles at Bridge*

The low structure elevation is 906.7 which does not provide any freeboard above the design elevation. To meet the DNR's freeboard criteria, the bridge would need to be raised 3'-5".

ANALYSIS

Procedure

The structural analysis was performed according to *AASHTO LRFR.* Using the geometry, member size data, and measured deficiencies, the trusses, stringers, and floor beams were modeled in the structural analysis software STAAD. The Ratings were calculated and the controlling ratings were taken as the minimums. The bearing pins and sidewalk brackets were also analyzed due to their deteriorated conditions.

For pedestrian use, the Inventory Rating represents the maximum pedestrian live load that the bridge can safely support for an indefinite period of time. The Operating Rating represents the absolute maximum pedestrian live load that the bridge can support for a short period of time. To meet pedestrian design live load criteria the Inventory Rating should be at least 90 psf. For vehicular use, the Inventory Rating Factor represents the proportion of design vehicular live load that the bridge can safely support for an indefinite period of time. A Rating Factor greater than or equal to 1 means the bridge is sufficient for design vehicular live loads. If the Rating Factor is less than 1, the legal live loads need to be evaluated according to IDOT criteria to determine the appropriate weight restrictions. Analysis calculations are included in Appendix B.

Pedestrian Only Use

For the truss analysis, dead loads were taken from the plans. The design pedestrian live load per *AASHTO LRFD Bridge Design Specifications* is 90 psf distributed over the entire deck area. Load factors were applied according to *AASHTO LRFD* for the Strength I load combination which produces the maximum member stresses. The factored loads were then distributed evenly and applied to the bottom chord panel points. The analysis was then run in the truss model to determine the maximum axial forces in the truss members. The axial capacities were calculated for the truss members per *AASHTO LRFD* and compared to the maximum member forces from the truss model. From that comparison, the controlling member(s) was chosen as the one with the highest ratio of maximum force to axial capacity. Additional factors were then applied to the controlling member capacity per *AASHTO LRFR* to account for the condition and importance of that particular member. The Inventory and Operating Ratings were then calculated and reported in pounds per square foot.

The dead load acting on the stringers includes the deck, railing (external stringers), and the stringer self weight. The pedestrian live load is distributed across the stringer tributary area. The loads were factored for the Strength I load combination and the load applied uniformly along the stringer. The maximum bending moment was then calculated and compared to the bending capacity per *AASHTO LRFD* with the additional factors from *AASHTO LRFR* to account for the condition and importance of the member. The Inventory and Operating Ratings were then calculated and reported in pounds per square foot.

The dead load acting on the floor beams includes the deck, stringer weight, and self weight. The pedestrian live load is transferred from the stringers to the floor beams. The loads were factored for the Strength I load combination and applied as point loads at the stringer to floor beam connections. The maximum bending moment was then calculated and compared to the bending capacity per *AASHTO LRFD* with the additional factors from *AASHTO LRFR* to account for the condition and importance of the member. The Inventory and Operating Ratings were then calculated and reported in pounds per square foot.

Vehicular Use

For the truss analysis, dead loads were taken from the plans. The design live load per *AASHTO LRFD Bridge Design Specifications* is HL-93 which includes an evenly distributed lane load of 640 pounds per foot, an 8,000 pound axle, and two 32,000 pound axles spaced as shown in Figure 5. Load factors were applied according to *AASHTO LRFD* for the Strength I load combination which produces the maximum member stresses. The factored loads were then applied to the bottom chord panel points. The analysis was then run in the truss model to determine the maximum axial forces in the truss members. The axial capacities were calculated for the truss members per *AASHTO LRFD* and compared to the maximum member forces from the truss model. From that comparison, the controlling member(s) was chosen as the one with the highest ratio of maximum force to axial capacity. Additional factors are then applied to the controlling member capacity per *AASHTO LRFR* to account for the condition and importance of that particular member. The Inventory and Operating Rating Factors were then calculated and reported as unit-less proportions of the HL-93 live load. If the controlling Inventory Rating Factor is less than 1, the analysis should examine the load carrying capacity for Legal Loads which are shown in Figure 6. The load application is performed similarly to the Design Load and the results are reported in tons.

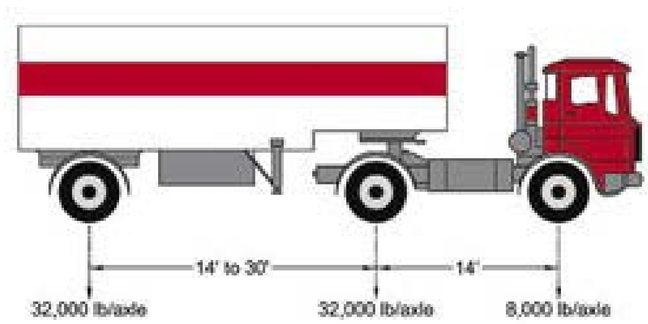


Figure 5 *AASHTO HL-93 Axle Loads*

The dead load acting on the stringers includes the deck, railing (external stringers), and the stringer self weight. The HL-93 design live load is positioned such that it produces the maximum bending stress in the stringer being analyzed. The loads were factored for the Strength I load combination and the load applied uniformly along the stringer. The maximum bending moment was then calculated and compared to the bending capacity per *AASHTO LRFD* with the additional factors from *AASHTO LRFR* to account for the condition and importance of the member. The Inventory and Operating Rating Factors were then calculated and reported as unit-less proportions of the HL-93 live load. If the controlling Inventory Rating

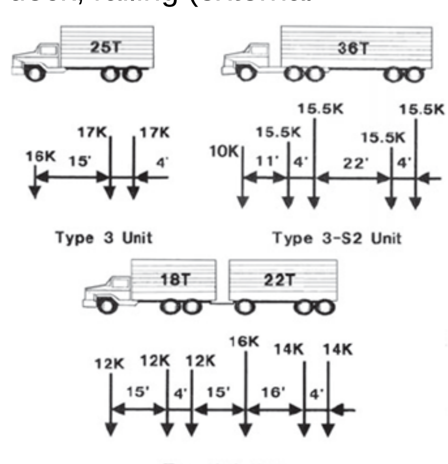


Figure 6 *AASHTO Legal Loads*

Factor is less than 1 the load carrying capacity for Legal Loads is then analyzed and reported in tons.

The dead load acting on the floor beams includes the deck, stringer weight, and self weight. The HL-93 design live load is positioned such that it produces the maximum bending stress in the floor beam. The loads were factored for the Strength I load combination and applied as point loads at the stringer to floor beam connections. The maximum bending moment was then calculated and compared to the bending capacity per *AASHTO LRFD* with the additional factors from *AASHTO LRFR* to account for the condition and importance of the member. The Inventory and Operating Rating Factors were then calculated and reported as unit-less proportions of the HL-93 live load. If the controlling Inventory Rating Factor is less than 1 the load carrying capacity for Legal Loads is then analyzed and reported in tons.

Results

Analysis calculations are shown in Appendix B. The following results summary were obtained from the analysis:

Pedestrian Only Use

East Truss -	The Inventory and Operating Ratings were calculated as **112 psf** and **146 psf** respectively. The analysis was controlled by truss member L2-L3 in tension.
West Truss -	The Inventory and Operating Ratings were calculated as **42 psf** and **54 psf** respectively. The analysis was controlled by truss member L3-L4 in tension.
Stringers - (interior)	The Inventory and Operating Ratings were calculated as **273 psf** and **353 psf** respectively.
Floor Beams -	The Inventory and Operating Ratings were calculated as **130 psf** and **168 psf** respectively.
Sidewalk Beam -	The Inventory and Operating Ratings were calculated as **87 psf** and **112 psf** respectively.
Truss Bearing - (southwest)	The Inventory and Operating Ratings were calculated as **111 psf** and **145 psf** respectively.

Taking the minimum Inventory Rating, the bridge rating is **42 psf** controlled by the west truss. This is 53% below the AASHTO pedestrian design live load of 90 psf. If the bridge is restricted to just the roadway (ie. blocking off the cantilevered sidewalk), the bridge rating is increased to 69 psf which is 23% below the design live load. To increase the bridge rating to the required 90 psf, in addition to repairs needed to the damaged and significantly deteriorated bridge elements, the west truss members circled in Figure 7 require strengthening on each of the three spans. The total number of members that require strengthening is 12.

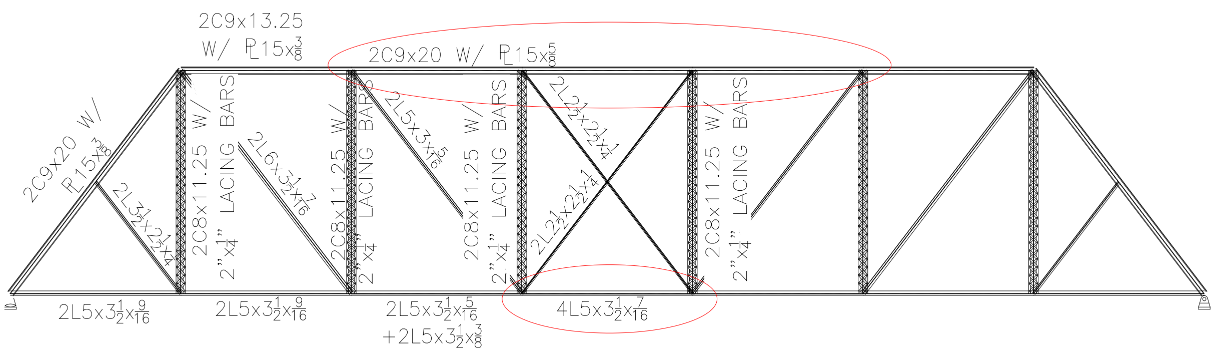


Figure 7 *West Truss Members Requiring Strengthening for Pedestrian Live Load*

Vehicular Use – Design Live Load

East Truss - The Inventory and Operating Rating Factor Factors were calculated as **0.43** and **0.55** respectively. The analysis was controlled by truss member U2-L3 in tension.

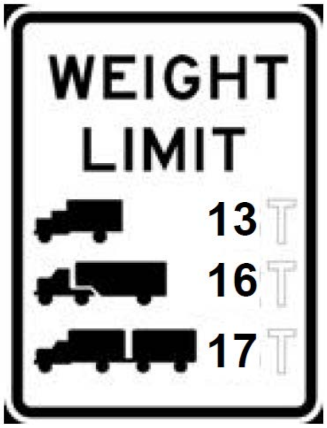
West Truss - The Inventory and Operating Rating Factors were calculated as **0.29** and **0.37** respectively. The analysis was controlled by truss member L3-L4 in tension.

Stringers - (interior) The Inventory and Operating Rating Factors were calculated as **0.7** and **0.91** respectively.

Floor Beams - The Inventory and Operating Rating Factors were calculated as **0.42** and **0.55** respectively.

Taking the minimum Inventory Rating Factor, the bridge rating factor is **0.29** controlled by the west truss. Because this is 71% below the AASHTO LRFD design live load, Legal Loads were required to be analyzed to determine the appropriate weight restrictions. The following summarizes the results of the Legal Load Analysis:

Vehicular Use – Legal Live Loads



Type 4 Truck - The Type 4 Rating was calculated as **13 tons.** The analysis was controlled by west truss member U2-U3 in compression.

Type 3S3 Truck - The Type 3S3 Rating was calculated as **16 tons.** The analysis was controlled by west truss member L3-L4 in tension.

Type 3-3 Truck - The Type 3-3 Rating was calculated as **17 tons.** The analysis was controlled by west truss member L3-L4 in tension.

To increase the bridge capacity to meet current legal and design live load criteria, in addition to repairs needed to the damaged and significantly deteriorated bridge elements, the following members require strengthening:

1. All interior stringers – 147 total
2. All floor beams – 18 total
3. East Truss members shown in Figure 8 – 36 total
4. West Truss members shown in Figure 9 – 36 total

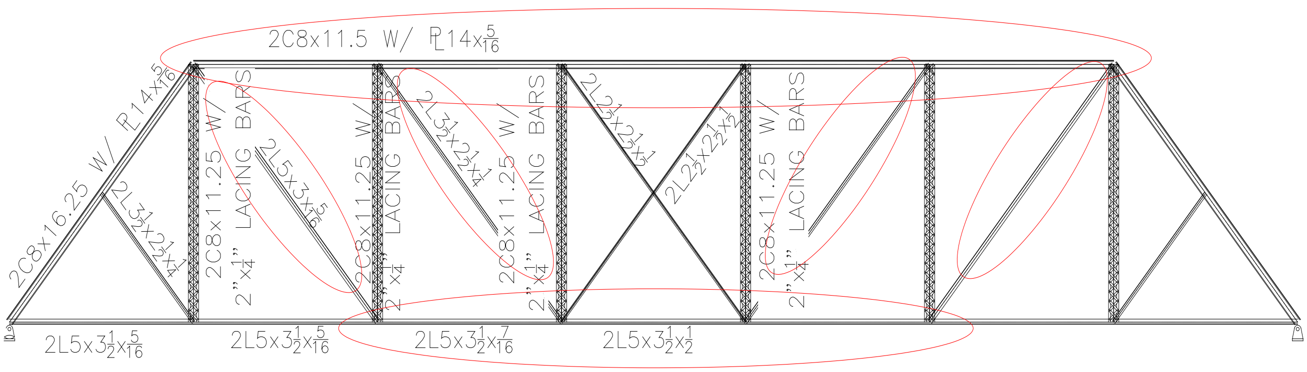


Figure 8 *East Truss Members Requiring Strengthening for Vehicular Live Load*

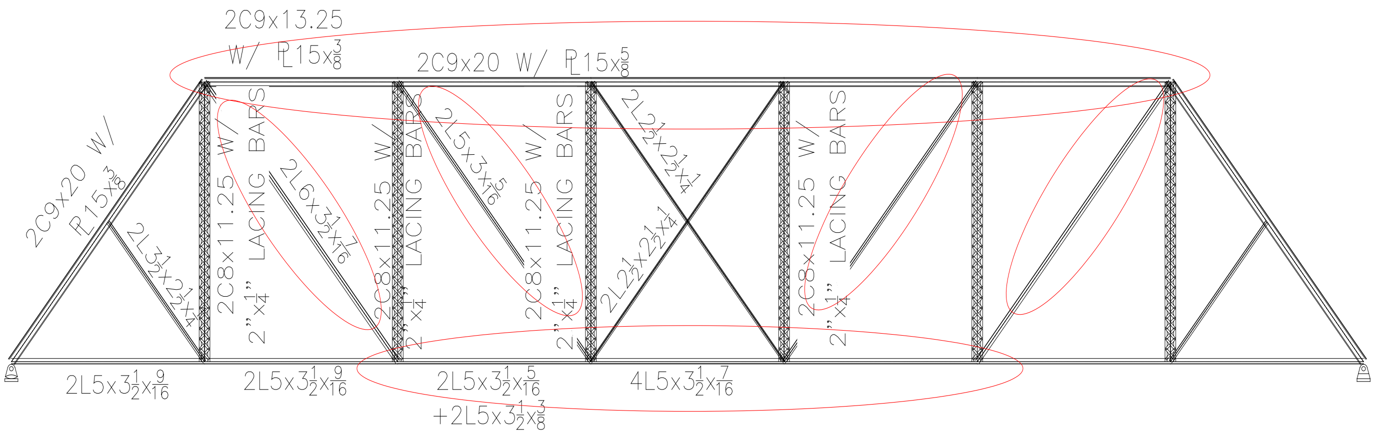


Figure 9 *West Truss Members Requiring Strengthening for Vehicular Live Load*

REHABILITATION ALTERNATIVES

Rehabilitation Items

The following rehabilitation items are required to bring the bridge up to code and mitigate the current unsafe conditions for the two rehabilitation alternatives (Options 2 & 3 in next section).

1. Substructure Rehabilitation
    a. Pier Repairs
        i. Replace pier caps to raise bridge profile by 3'-5".
        ii. Repair spalled and delaminated areas on both piers.
        iii. Add revetment at both piers.
    b. Abutment Repairs
        i. Repair spalled and delaminated areas on both abutments.
        ii. Raise abutment seats to raise bridge profile.
2. Superstructure Rehabilitation
    a. Truss Repairs/ Strengthening
        i. Reinforce overstressed truss members.
        ii. Repair/ reinforce Gusset plate connections that are distorted or have measureable section loss.

iii. Heat straighten member U1-L2 on the south span, east truss, member L4-U5 on the center span, west truss, member L3-U4 on the center span, east truss, and any other diagonal members distorted ½” or more.

iv. Replace southeast, southwest, and northwest bearings on south span.

b. Floor System Repairs

i. Reinforce floor beam flanges in areas of measureable section loss.

ii. Install new stringers adjacent to stringers with significant section loss, and the two south span stringers with cracks observed.

iii. Replace the stringer to floor beam connections where previous repairs replaced fasteners with welds.

iv. Reinforce sidewalk bracket flanges in areas of measureable section loss OR remove sidewalk.

v. Replace sidewalk bracket to floor beam connections in areas of measureable section loss OR remove sidewalk.

3. Bridge Approaches

a. Regrade and pave approaches to raise bridge profile.

Options

To aid in selection of the most appropriate rehabilitation or replacement alternative, the following six options were considered:

Option 1: Do nothing. This option entails leaving the bridge as-is and keeping it closed to vehicular and pedestrian traffic indefinitely.

Option 2: Rehabilitate the bridge and repurpose as a pedestrian bridge. This option entails all of the rehabilitation items in the previous section. Additionally, the entire bridge deck and railings need replacement to accommodate pedestrian use.

Option 3: Rehabilitate the bridge for vehicular and pedestrian use. This option entails all of the rehabilitation items in the previous section. The amount of required rehabilitation items is significantly great for Option 3 than Option 2.

Option 4: Replace the bridge with a new pedestrian bridge. This option entails removing the existing bridge and substructure, and constructing a new 3 span, 360’x14’ pre-engineered steel pony truss bridge for pedestrian use. The deck will be timber plank and the substructure will be reinforced concrete founded on steel piles. Similarly to the rehabilitation options (Option 2 & 3), the profile grade will be raised by 3’-5” to meet freeboard requirements.

Option 5: Replace the bridge “in-kind” for vehicular and pedestrian use. This option entails removing the existing bridge and substructure, and constructing a new geometrically similar bridge for vehicular and pedestrian use. The bridge will be a 3 span, 360’x40’ steel truss bridge with similar panel spacing and height as the existing bridge. The bridge will be significantly wider than the existing bridge and have more substantial truss members and floor system beams. The roadway deck will be steel grating and the sidewalk will be concrete. The substructure will be reinforced concrete on steel piles and the profile grade will be raised similarly to the previous options.

Option 6: Replace the bridge with a conventional modern bridge for vehicular and pedestrian use. This option entails removing the existing bridge and substructure, and constructing a new 3 span, 360’x40’ prestressed concrete beam bridge with sidewalk, for vehicular and pedestrian use. The substructure will be reinforced concrete on steel piles. The profile will be raised an additional 2’ more than the previous options (5’-8” total) due to the depth of the beams.

Drawings for Options 2-6 are shown in Appendix E and itemized Cost Opinions for Options 2-6 are shown in Appendix C.

Feasibility

The cost opinions shown in Appendix C were estimated using data from recent bid letting items from similar projects, current Iowa DOT bid item averages, and contractor input. Due to the age of the existing bridge, there is limited service life remaining even after rehabilitating the bridge from its’ current condition. That said, the cost opinion of Options 2 & 3 in comparison with the replacement options do not give a comprehensive cost analysis without considering the life-cycle cost. The current bridge design life, per *AASHTO LRFD,* is 75 years which shall be applicable to Options 4-6. It is anticipated that the remaining service life for Options 2 & 3 is 20 years, at which point it would be impractical to perform any further major rehabilitations as it was already rehabilitated in the 1970s.

The following table shows a more meaningful cost comparison of options by evaluating the life-cycle costs of a 20 year period. Also shown in the table are the historical implications and impact on the existing bridge for each of the options.

OPTION	DESCRIPTION	HISTORICAL IMPLICATION	IMPACT ON EXISTING STRUCTURE	DESIGN LIFE OR REMAINING SERVICE LIFE	COST OF WORK [1]	FUTURE VALUE[2]	LIFE-CYCLE COSTS[3]
1	Do nothing.	Existing structure will be neglected.	None.	NA	$0	$0	**$0**
2	Rehabilitate the existing bridge for pedestrian only use.	All historically significant elements of the bridge will be preserved.	This option will have minimal impact on the existing structure. Strengthening is minimal due to removal of sidewalk.	20 years	$1,045,000	$0	**$1,045,000**
3	Rehabilitate the existing bridge for vehicular and pedestrian only use.	All historically significant elements of the bridge will be preserved.	Due to the large amount of strengthening required, the appearance of the existing structure will be altered significantly.	20 years	$1,730,000	$0	**$1,730,000**
4	Replace existing bridge with a new 3 span, 360'x14' pre-engineered steel pony truss bridge with timber deck for pedestrian only use. Substructure will be reinforced concrete on steel piles.	All of the historical elements will be lost.	All existing bridge components will be lost.	75 years	$1,711,000	$1,254,733	**$1,118,169**
5	Replace existing bridge with a new 3 span, 360'x40' steel truss bridge that replicates some of the geometry of the existing bridge, for vehicular and pedestrian use. The roadway deck will be steel grating and the sidewalk will be concrete. Substructure will be reinforced concrete on steel piles.	This option tries to replicate the existing structure. For replacement options, it is the most true to the original structure.	All existing bridge components will be lost.	75 years	$2,961,000	$2,171,400	**$1,936,099**
6	Replace existing bridge with a new 3 span, 360'x40' prestressed, precast concrete beam bridge for vehicular and pedestrian use. Substructure will be reinforced concrete on steel piles.	All of the historical elements will be lost.	All existing bridge components will be lost.	75 years	$2,446,000	$1,793,733	**$1,599,358**

1. Includes engineering and construction management.
2. Value of remaining service life after 20 years.
3. CURENT VALUE = FUTURE VALUE x $1/(1+r)^n$
   Current State & Local bonds interest rate of 3.82% used for r, 20 years used for n.
   LIFE CYCLE COSTS = COST OF WORK – CURRENT VALUE

CONCLUSION & RECOMENDATIONS

The 3rd Street Bridge should be considered a bridge of high historical significance being one of Iowa's few remaining major bridges of an archaic design that was dominant in the era of its' construction. The bridge is too iconic of a structure to neglect so Option 1 is not recommended. According to *Guidelines for Historic Bridge Rehabilitation and Replacement,* which is an AASHTO requested study as part of the National Cooperative Highway Research Program; none of the proposed rehabilitation items for Option 2 would negatively impact the historical significance of the bridge therefore it can retain its' eligibility to be listed on the National Register of Historic Places. Also, the comparatively low cost of construction make Option 2 a feasible alternative. Due to the significant amount of reinforcing required for Option 3, the historically significant components of the bridge would be altered such that it could potentially become ineligible to be listed on the National Register of Historic Places. For this reason and the high cost for a limited service life, Option 3 should not be considered feasible.

Options 4, 5, and 6 are all acceptable alternatives from an engineering standpoint as they are entirely new construction, but with the exception of Option 5 being an homage to the original structure, all of the historic elements of the original bridge would be lost. The high construction costs of Options 5 & 6 make them cost prohibitive alternatives. Although Option 4 has a comparatively high construction cost, when the life-cycle costs are considered it becomes a feasible alternative and is life-cycle cost competitive with Option 2.

Between the two feasible alternatives, Option 2 & 4, the decision of whether to rehabilitate or replace depends on the priorities of the City. Both options adhere to current design criteria for a pedestrian bridge. Option 2 has a lower construction cost but within its' anticipated 20 year remaining service life the life-cycle costs become very close. Option 2 would provide a wider deck, 19 ft. versus 14 ft., of the two options. If the City prefers constructing a bridge with a substantial design life over preservation of the historical aspects or sentimental value of the bridge, clearly Option 4 is the prevailing alternative. If historical preservation weighs heavier than the longer design life then Option 2 should be chosen.

APPENDIX A

Bridge Photos

East Elevation from South Embankment

Roadway Looking North

East Elevation – South Span

East Elevation – Center Span

East Elevation – North Span

South Span Floor System Looking North

Southwest Bearing at South Abut. – Major section loss

South Abut. Seat – Major cracking w/ efflorescence

South Abut. Back Wall – Major cracking w/ efflorescence

South Abut. – Large cracks, spalling w/ exposed rebar

Sidewalk Bracket @ S. Pier – Section loss through bott. flange

S. Pier Cap – Major delamination, crushing. Loss of bearing

S. Pier, S. Wall – Delamination, spalling, cracking w/leaching

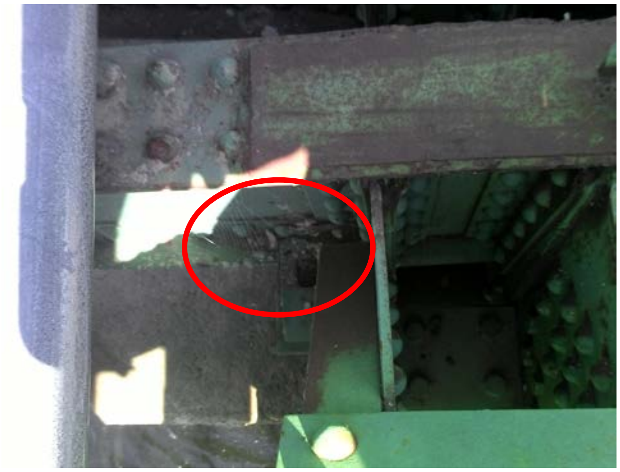
S. Pier, West End – Major cracking w/ efflorescence

S. Pier, East End – Major spalling near waterline

S. Pier, N. Wall – Major cracking w/ efflorescence

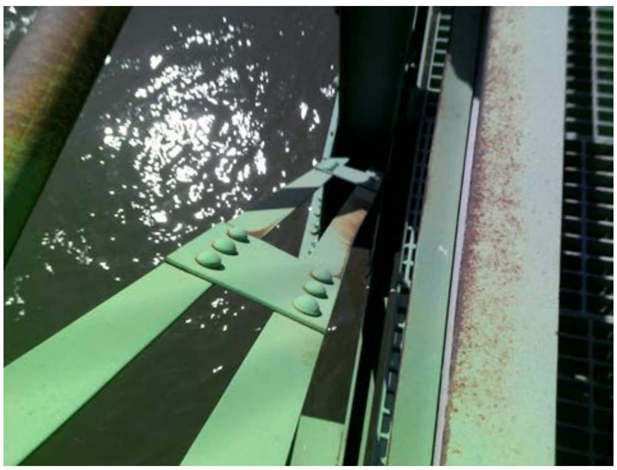
North Pier, South Wall – Cracking w/ efflorescence

North Abutment – Major cracking w/ efflorescence



Sidewalk Bracket @ N. Pier – Section loss through bott. Flange

West Truss, U2L3 – Distortion, out of plane bending



West Truss, U2L3 – Distortion, out of plane bending

S.W. Bracket @ N. Span, 4th Panel – Section loss through bott. Flange

Typical corrosion on deck grating

APPENDIX B

Load Rating Calculations

PEDESTRIAN ONLY USE

Span length: $L_{span\_1} := 120\mathbf{ft}$

Panel length: $L_{panel\_1} := 17.15\mathbf{ft}$

Deck width: $W_{deck} := 18\mathbf{ft}$

per IDOT HR-239: $F_y := 30\mathbf{ksi}$

Dead Load

Distributed DL taken from original plan sht. 4

$$w_{DL\_1} := 575\frac{\mathbf{lb}}{\mathbf{ft}} \qquad w_{DL\_2} := 1275\frac{\mathbf{lb}}{\mathbf{ft}}$$

DL Panel Point Load

$$DL_1 := L_{panel\_1} \cdot w_{DL\_1} = 9.86 \cdot \mathbf{k} \qquad DL_2 := L_{panel\_1} \cdot w_{DL\_2} = 21.87 \cdot \mathbf{k}$$

Pedestrian Live Load

AASHTO pedestrian LL per LRFD 3.6.1.6

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

LL Panel Point Load (east)

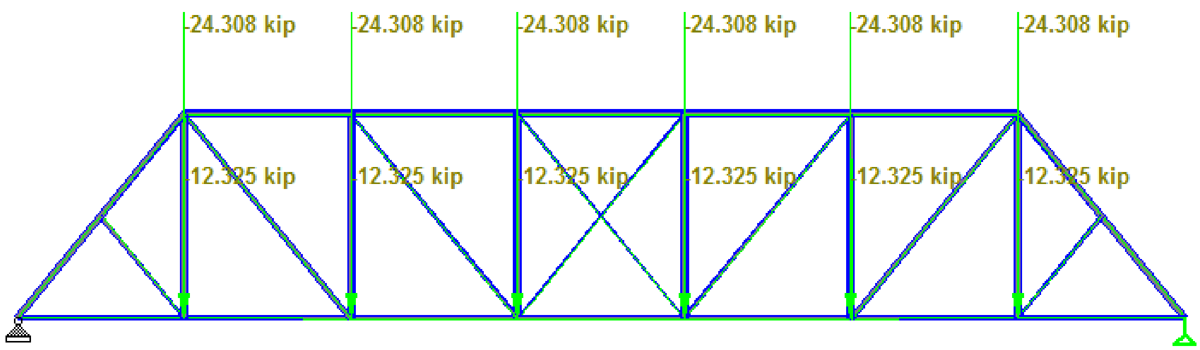
$$LL_{ped\_1} := \frac{W_{deck}}{2} \cdot L_{panel\_1} \cdot w_{ped\_LL} = 13.89 \cdot \mathbf{k}$$

LL Panel Point Load (west)

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

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 $\mathbf{C_{truss\_1}} := \mathbf{min}\left(0.85, \varphi_{c\_truss\_1} \cdot \varphi_{s\_truss\_1}\right) \cdot 228.3\mathbf{k}$

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

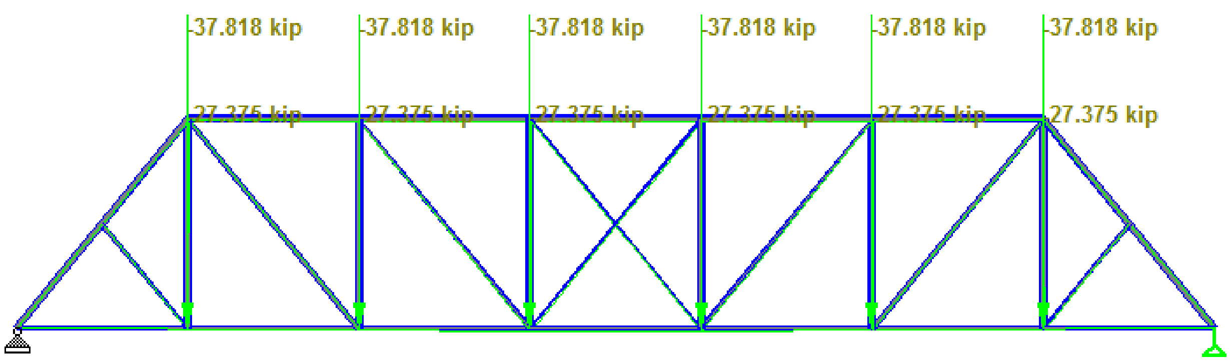
L2-L3 LL $\gamma\mathbf{LL_{truss\_1}} := 1.75 \cdot 56.7\mathbf{k}$

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

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

West Truss

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



Analysis Results per AASHTO LRFR:

			A_g (in^2)	L (in)	r (in)	KL/r	Allowable Comp. $ø_cP_n$ k	Allowable Tension $ø_yF_yA_g$ k	ACTUAL LOAD $?_iP_u$ k	S.R.
2C9x20 (w/ 3/8x15 PL)	L0-U1	TC	16.4	162.66	3.44	47.28	402.13	468.26	252.5	0.63
2C9x13.25 (w/ 3/8x15 PL)	U1-U2	TC	12.6	205.75	3.56	57.79	293.09	358.25	266.1	0.91
2C9x13.25 (w/ 3/8x15 PL)	U2-U3	TC	12.6	205.75	3.56	57.79	293.09	358.25	319.3	1.09
2C9x13.25 (w/ 3/8x15 PL)	U3-U4	TC	12.6	205.75	3.56	57.79	293.09	358.25	316.5	1.08
2L5x3.5x9/16	L0-L1	BC	11.6	205.75	0.97	212.11	58.19	330.60	-159.7	0.48
2L5x3.5x9/16	L1-L2	BC	11.6	205.75	0.97	212.11	58.19	330.60	-159.7	0.48
2L5x3.5x5/16 + 2L5x3.5x3/8	L2-L3	BC	11.2	205.75	1.02	201.72	62.23	319.77	-266.1	0.83
4L5x3.5x7/16	L3-L4	BC	10.2	205.75	1.02	201.72	56.80	291.84	-322.2	1.10
2C8x11.25 (w/ 2"x1/4" lacing)	L1-U1	vert	6.61	252.00	3.16	79.75	134.98	188.39	-65.2	0.35
2C8x11.25 (w/ 2"x1/4" lacing)	L2-U2	vert	6.61	252.00	3.16	79.75	134.98	188.39	65.2	0.48
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	-168.3	0.65
2L5x3x5/16	U2-L3	diag.	4.81	325.31	0.85	382.72	7.41	137.09	-84.2	0.61
2L2.5x2.5x1/4	U3-L4	diag.	2.37	162.66	0.76	214.02	11.68	67.55	4.5	0.39

Member L3-L4 controls

AASHTO LRFR Pedestrian Load Rating:

L3-L4 Capacity

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

L3-L4 DL

$$\gamma DL_{truss\_2} := 1.25 \cdot 108.2\mathbf{k}$$

L3-L4 LL

$$\gamma LL_{truss\_2} := 1.75 \cdot 106.8\mathbf{k}$$

Inventory Rating

$$INV_{truss\_2} := w_{ped\_LL} \cdot \frac{C_{truss\_2} - \gamma DL_{truss\_2}}{\gamma LL_{truss\_2}} = 42.36 \cdot \mathbf{psf}$$

Operating Rating

$$OPR_{truss\_2} := w_{ped\_LL} \cdot \frac{C_{truss\_2} - \gamma DL_{truss\_2}}{\gamma LL_{truss\_2} \cdot \left(\frac{1.35}{1.75}\right)} = 54.92 \cdot \mathbf{psf}$$

Floor System

Exterior Stringers:

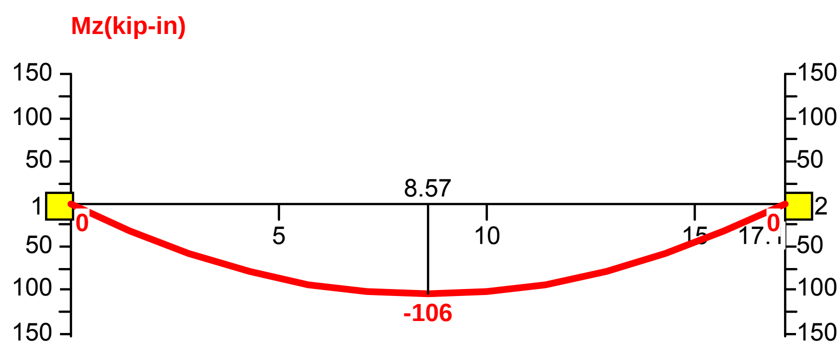
Distributed DL

$$w_{DL\_1\_ext\_strngr} := \frac{w_{DL\_1}}{W_{deck}} \cdot \frac{(2\mathbf{ft} + 5\mathbf{in})}{2} = 38.6 \frac{\mathbf{lb}}{\mathbf{ft}}$$

Distributed LL

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

Stringer forces analyzed in STAAD per AASHTO LRFR Pedestrian Strength I



East exterior stringer channels are C9x13.

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

Nominal flexural resistance

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

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:

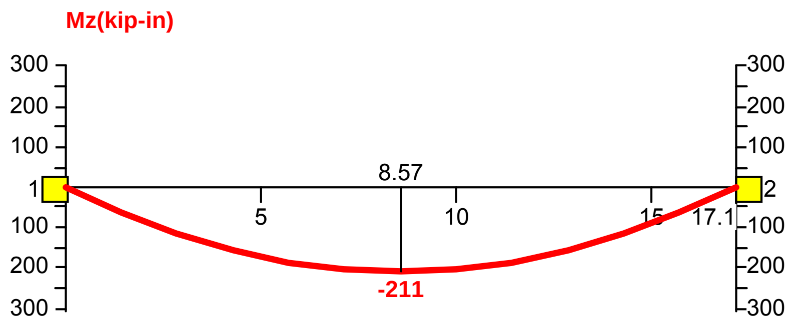
Distributed DL

$$w_{DL\_1\_strngr} := \frac{w_{DL\_1}}{W_{deck}} \cdot (2\mathbf{ft} + 5\mathbf{in}) = 77.2 \frac{\mathbf{lb}}{\mathbf{ft}}$$

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):

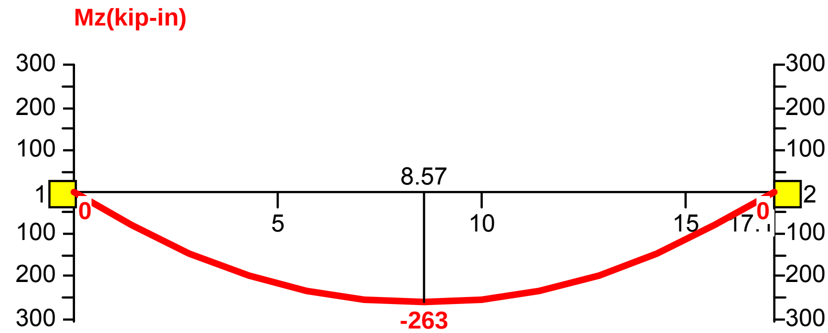
Distributed DL

$$w_{DL\_2\_ext\_strngr} := \frac{w_{DL\_2}}{W_{deck}} \cdot (2ft + 5in) = 171.18 \frac{lb}{ft}$$

Distributed LL

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

Stringer forces analyzed in STAAD per AASHTO LRFR Pedestrian Strength I



Stringers are C12x20

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

Nominal flexural resistance

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

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

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

Stringer LL moment

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

Inventory Rating

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

Operating Rating

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

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

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

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

Flexural resistance at truss end

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

AASHTO LRFR Load Rating:

Bracket Capacity

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

DL moment at truss end

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

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:

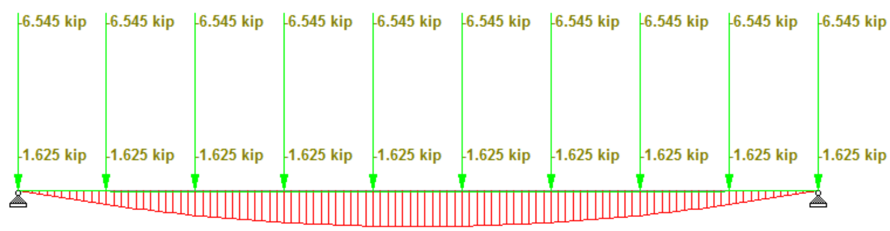
DL from stringers

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

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

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

Floor Beam LL moment

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

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

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

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

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 $\boxed{INV_{ped} = 42.4 \cdot psf}$

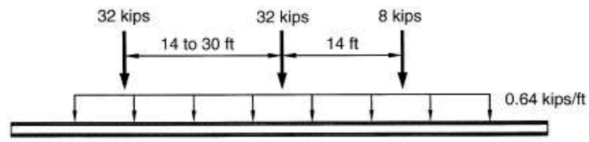
$OPR_{ped} := \min(OPR_{SW\_brg}, OPR_{FB\_1}, OPR_{strngr\_1}, OPR_{ext\_strngr\_1}, OPR_{ext\_strngr\_1}, OPR_{truss\_1}, OPR_{truss\_2}, OPR_{s}$

Operating Rating $\boxed{OPR_{ped} = 54.9 \cdot psf}$

This rating is 53% below AASHTO pedestrian design live load (90 psf). Restricting use to just the roadway (blocking off the sidewalk) would result in a rating of **69.3 psf** (Inventory level) which is still 23% below the design load.

VEHICULAR USE

HL-93



Design Live Load

Stringer spacing

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

Live load distribution factor per AASHTO 4.6.2.2.2b-1

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

Live load impact factor

$$IM := 0.33$$

Wheel loads

$$P_1 := 0.5 \cdot 8\mathbf{k} = 4000\,\mathbf{lb} \qquad P_2 := 0.5 \cdot 32\mathbf{k} = 16000\,\mathbf{lb}$$

Lane load

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

Floor System

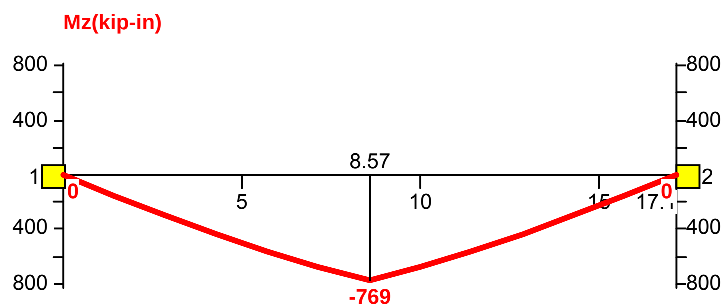
Interior

Point loads on stringers

$$LL_{stringer} := (1 + IM) \cdot DF_{interior} \cdot P_2 = 6.43 \cdot \mathbf{k}$$

(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

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

Inventory Rating Factor

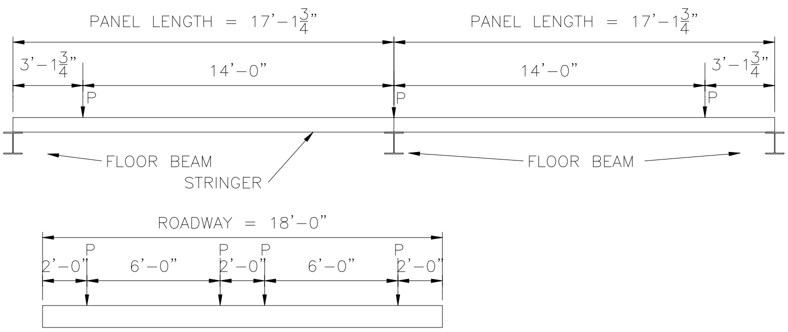
$$INV_{strngr\_2} := 1 \cdot \frac{C_{strngr\_1} - \gamma DL_{strngr\_1}}{\gamma LL_{strngr\_2}} = 0.7$$

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):



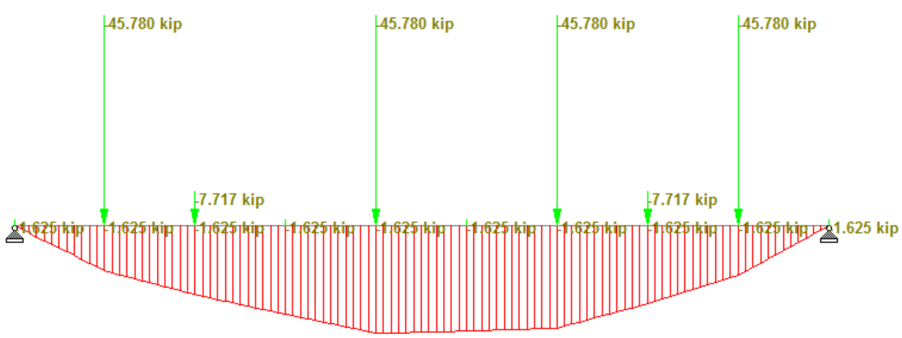
Point loads on floor beams

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

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

$$\gamma\mathbf{LL_{FB\_2}} := 1.75\cdot 274.9\mathbf{k}\cdot\mathbf{ft}$$

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

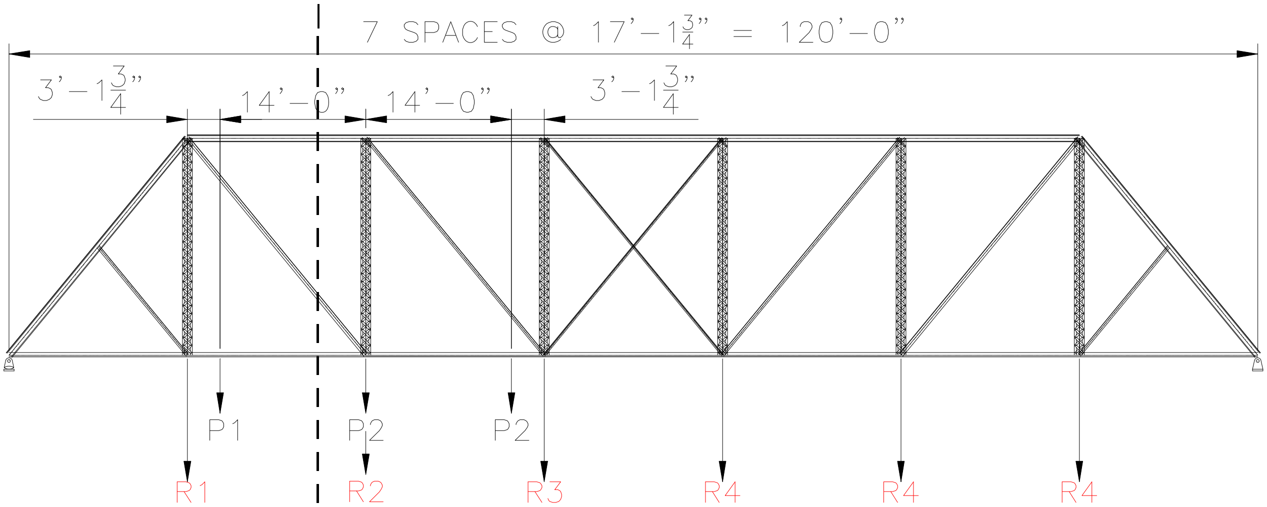
$$\mathbf{R_1} := (1 + \mathbf{IM}) \cdot 2\mathbf{P_1} \cdot \left(\frac{\mathbf{L_{panel\_1}} - 3\mathbf{ft} - 1.75\mathbf{in}}{\mathbf{L_{panel\_1}}} \right) + (1 + \mathbf{IM}) \cdot \mathbf{LL_{lane}} \cdot \mathbf{L_{panel\_1}} = 13.1 \cdot \mathbf{k}$$

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

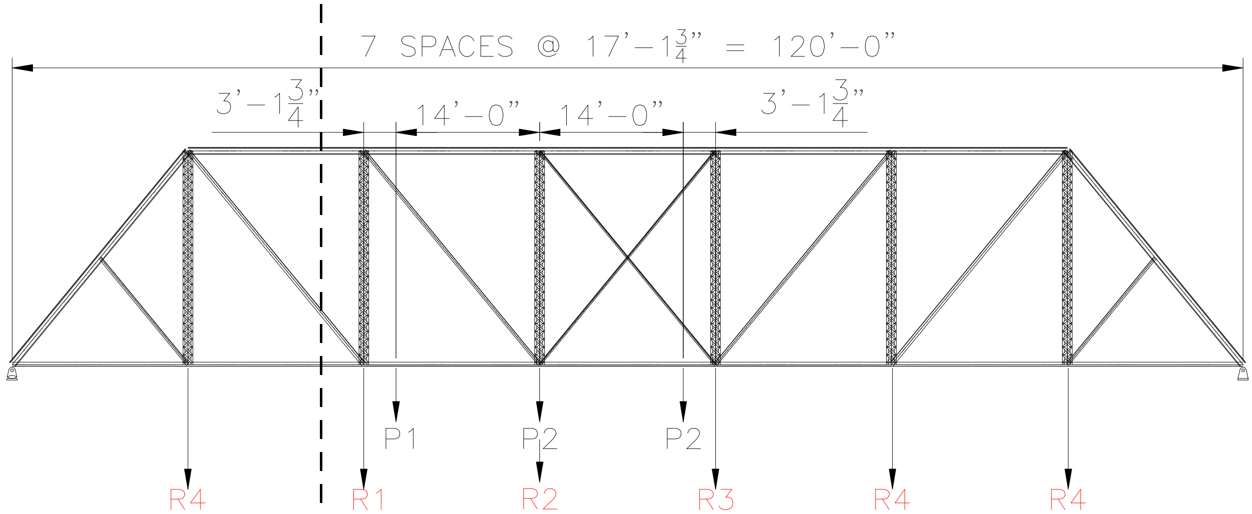
$$\mathbf{R_3} := (1 + \mathbf{IM}) \cdot 2\mathbf{P_2} \cdot \left(\frac{\mathbf{L_{panel\_1}} - 3\mathbf{ft} - 1.75\mathbf{in}}{\mathbf{L_{panel\_1}}} \right) + (1 + \mathbf{IM}) \cdot \mathbf{LL_{lane}} \cdot \mathbf{L_{panel\_1}} = 39.16 \cdot \mathbf{k}$$

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

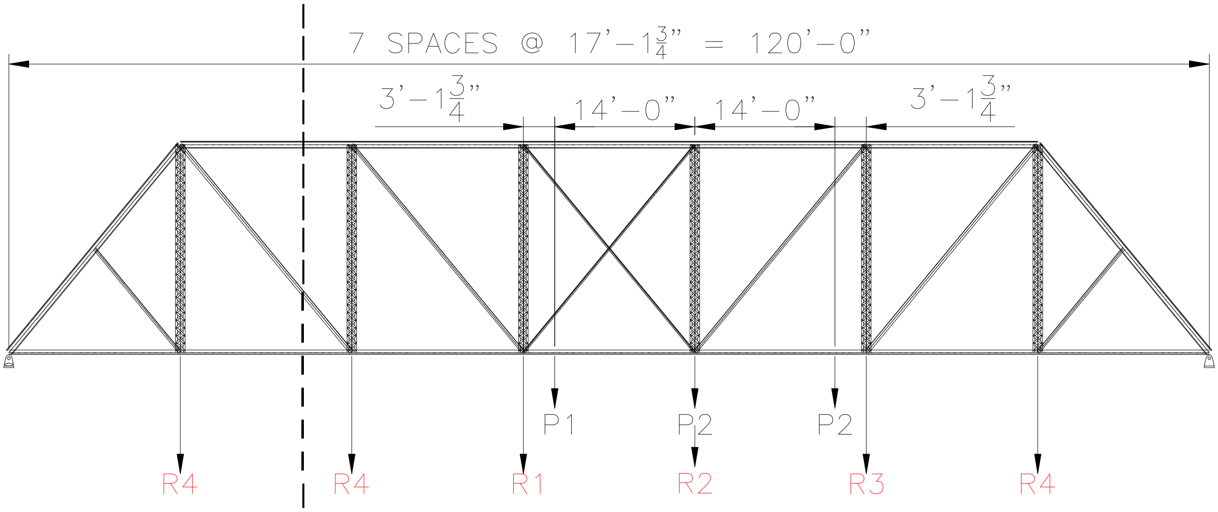
Case 1:



Case 2:



Case 3:



Truss forces analyzed in STAAD per AASHTO LRFR Strength I

							Allowable Comp.	Allowable Tension	ACTUAL LOAD	
			A_g (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.
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 $$C_{truss\_3} := \min(0.85, \varphi_{c\_truss\_1} \cdot \varphi_{s\_truss\_1}) \cdot 82.65\mathbf{k}$$

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

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

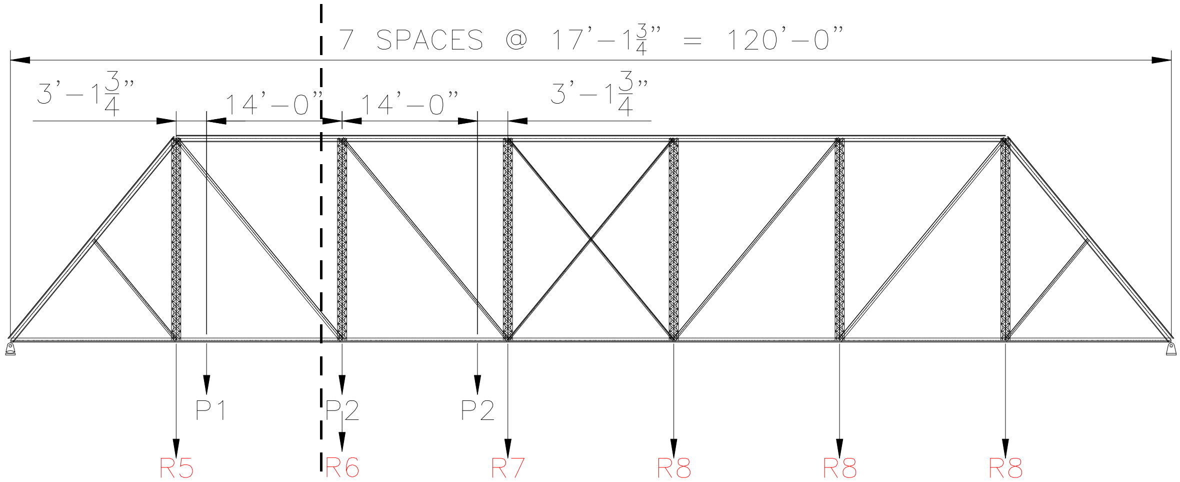
$$R_6 := (1+IM)\cdot 2P_2\cdot\left(\frac{3ft + 1.75in}{L_{panel\_1}} + 1\right) + (1+IM)\cdot 2P_1\cdot\left(\frac{3ft + 1.75in}{L_{panel\_1}}\right) + (1+IM)\cdot LL_{lane}\cdot L_{panel\_1} + w_{ped\_LL}\cdot L_{pan}$$

$$R_6 = 64.45\cdot k$$

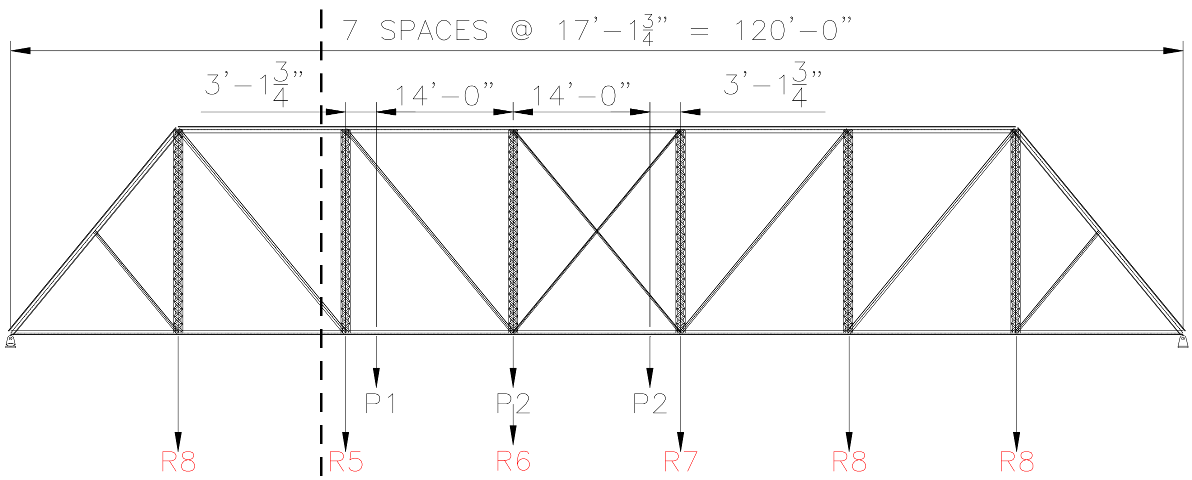
$$R_7 := (1+IM)\cdot 2P_2\cdot\left(\frac{L_{panel\_1} - 3ft - 1.75in}{L_{panel\_1}}\right) + (1+IM)\cdot LL_{lane}\cdot L_{panel\_1} + w_{ped\_LL}\cdot L_{panel\_1}\cdot 5ft = 46.$$

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

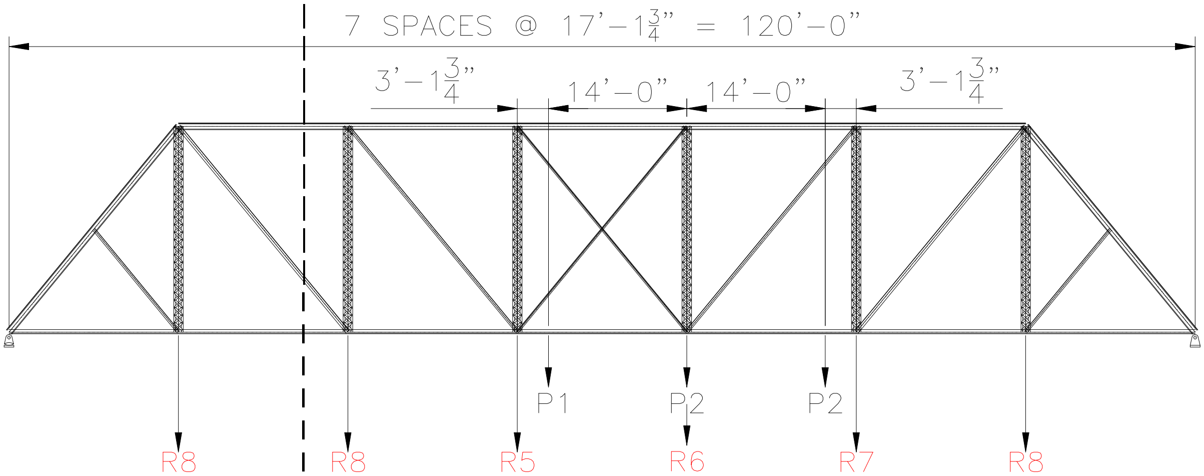
Case 1:



Case 2:



Case 3:



Truss forces analyzed in STAAD per AASHTO LRFR Strength I

			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 ?ᵢP_u k	S.R.
2C9x20 (w/ 3/8x15 PL)	L0-U1	TC	16.4	162.66	3.44	47.28	402.13	468.26	320.4	0.80
2C9x13.25 (w/ 3/8x15 PL)	U1-U2	TC	12.6	205.75	3.56	57.79	293.09	358.25	370.9	1.27
2C9x13.25 (w/ 3/8x15 PL)	U2-U3	TC	12.6	205.75	3.56	57.79	293.09	358.25	444	1.51
2C9x13.25 (w/ 3/8x15 PL)	U3-U4	TC	12.6	205.75	3.56	57.79	293.09	358.25	433.3	1.48
2L5x3.5x9/16	L0-L1	BC	11.6	205.75	0.97	212.11	58.19	330.60	-211.5	0.64
2L5x3.5x9/16	L1-L2	BC	11.6	205.75	0.97	212.11	58.19	330.60	-211.5	0.64
2L5x3.5x5/16 + 2L5x3.5x3/8	L2-L3	BC	11.2	205.75	1.02	201.72	62.23	319.77	-370.9	1.16
4L5x3.5x7/16	L3-L4	BC	10.2	205.75	1.02	201.72	56.80	291.84	-441.1	1.51
2C8x11.25 (w/ 2"x1/4" lacing)	L1-U1	vert	6.61	252.00	3.16	79.75	134.98	188.39	-63.8	0.34
2C8x11.25 (w/ 2"x1/4" lacing)	L2-U2	vert	6.61	252.00	3.16	79.75	134.98	188.39	127	0.94
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	-257.6	1.00
2L5x3x5/16	U2-L3	diag.	4.81	325.31	0.85	382.72	7.41	137.09	-163.9	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

$$INV_{truss\_4} := 1 \cdot \frac{C_{truss\_2} - \gamma DL_{truss\_2}}{\gamma LL_{truss\_4}} = 0.29$$

Operating Rating Factor

$$OPR_{truss\_4} := 1 \cdot \frac{C_{truss\_2} - \gamma DL_{truss\_2}}{\gamma LL_{truss\_4} \cdot \left(\frac{1.35}{1.75}\right)} = 0.37$$

BRIDGE RATINGFOR VEHICULAR USE:

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

Inventory Rating Factor

$$\boxed{INV_{vehicle} = 0.29}$$

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

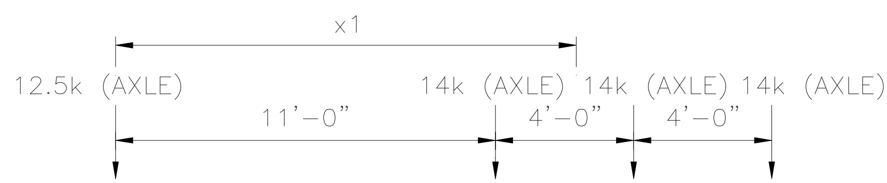
Operating Rating Factor

$$\boxed{OPR_{vehicle} = 0.37}$$

This rating is 71% below AASHTO LRFD design live load at the Inventory level. Legal loads should be evaluated.

Legal Loads

Type 4 Truck

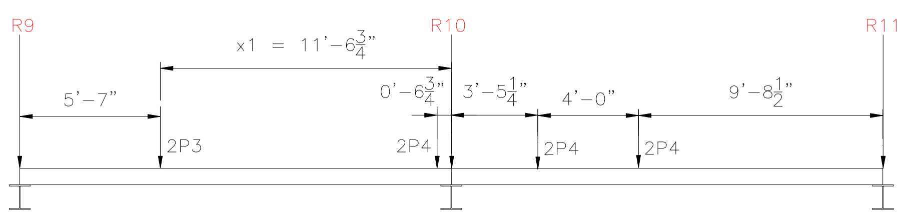


Wheel loads

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

Resultant distance

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



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

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

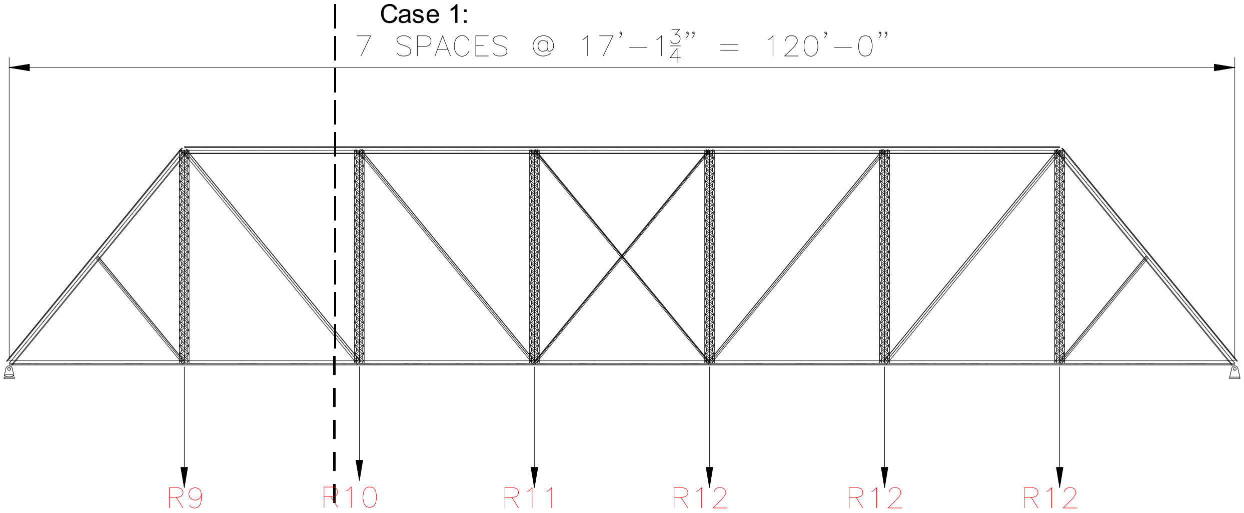
$$R_{10} = 62.36 \cdot \mathbf{k}$$

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

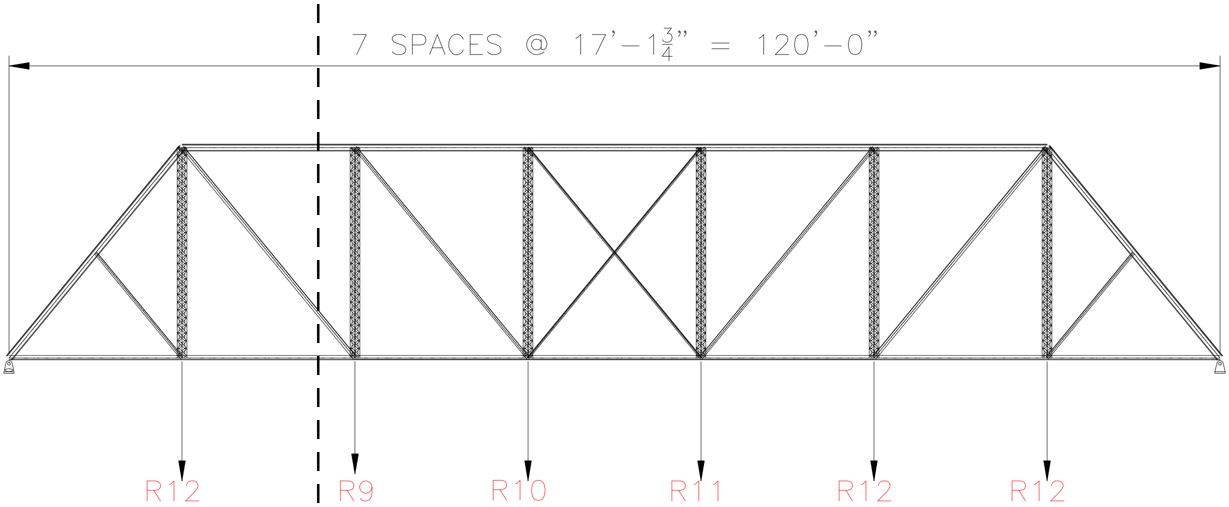
$$R_{12} := w_{ped\_LL} \cdot L_{panel\_1} \cdot 5\mathbf{ft} = 7.72 \cdot \mathbf{k}$$

West Truss

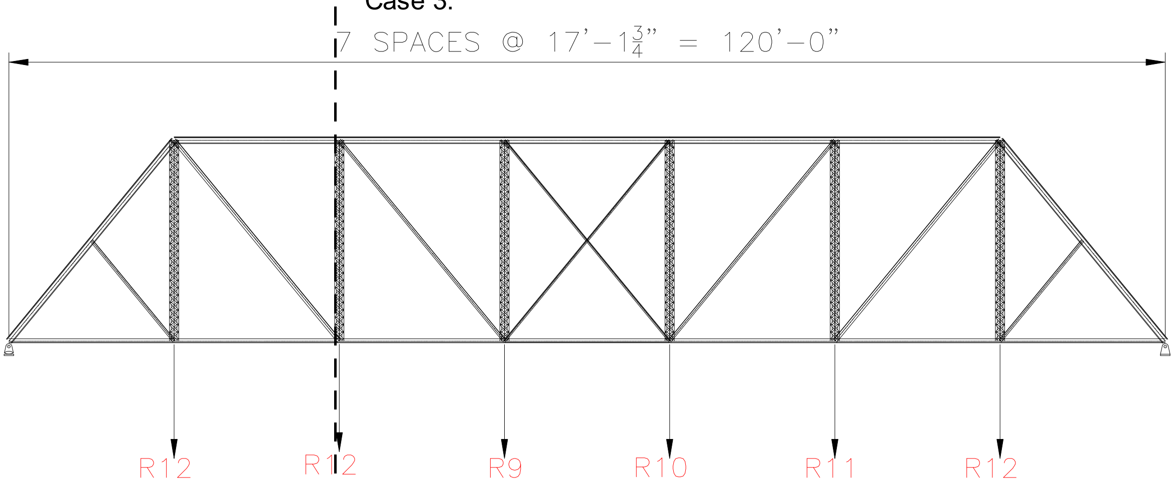
Case 1:



Case 2:



Case 3:



Truss forces analyzed in STAAD per AASHTO LRFR Strength I

							Allowable Comp.	Allowable Tension	ACTUAL LOAD	
			A_g (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	284.6	0.71
2C9x13.25 (w/ 3/8x15 PL)	U1-U2	TC	12.6	205.75	3.56	57.79	293.09	358.25	309.7	1.06
2C9x13.25 (w/ 3/8x15 PL)	U2-U3	TC	12.6	205.75	3.56	57.79	293.09	358.25	375.1	1.28
2C9x13.25 (w/ 3/8x15 PL)	U3-U4	TC	12.6	205.75	3.56	57.79	293.09	358.25	356.3	1.22
2L5x3.5x9/16	L0-L1	BC	11.6	205.75	0.97	212.11	58.19	330.60	-180	0.54
2L5x3.5x9/16	L1-L2	BC	11.6	205.75	0.97	212.11	58.19	330.60	-180	0.54
2L5x3.5x5/16 + 2L5x3.5x3/8	L2-L3	BC	11.2	205.75	1.02	201.72	62.23	319.77	-309.7	0.97
4L5x3.5x7/16	L3-L4	BC	10.2	205.75	1.02	201.72	56.80	291.84	-362.7	1.24
2C8x11.25 (w/ 2"x1/4" lacing)	L1-U1	vert	6.61	252.00	3.16	79.75	134.98	188.39	-61.6	0.33
2C8x11.25 (w/ 2"x1/4" lacing)	L2-U2	vert	6.61	252.00	3.16	79.75	134.98	188.39	99.6	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	-206.6	0.80
2L5x3x5/16	U2-L3	diag.	4.81	325.31	0.85	382.72	7.41	137.09	-128.6	0.94
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:

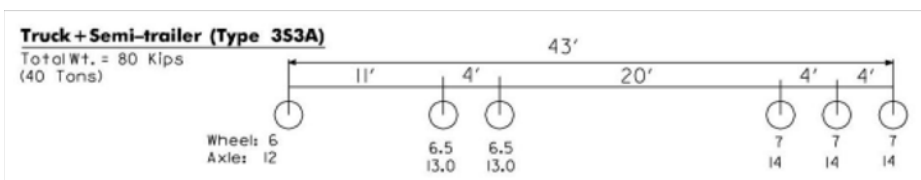
U2-U3 Capacity $C_{truss\_5} := 0.85 \cdot 293.09\mathbf{k}$

U2-U3 DL $\gamma DL_{truss\_5} := 1.25 \cdot 107.3\mathbf{k}$

U2-U3 LL $\gamma LL_{truss\_5} := 1.75 \cdot 137.7\mathbf{k}$

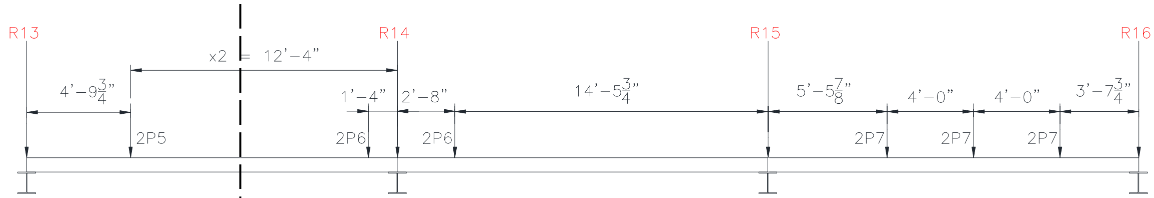
Type 4 Truck Rating
$$\boxed{\mathbf{Type\_4} := 27.25\mathbf{ton} \cdot \frac{C_{truss\_5} - \gamma DL_{truss\_5}}{\gamma LL_{truss\_5}} = 13 \cdot \mathbf{ton}}$$

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



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

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

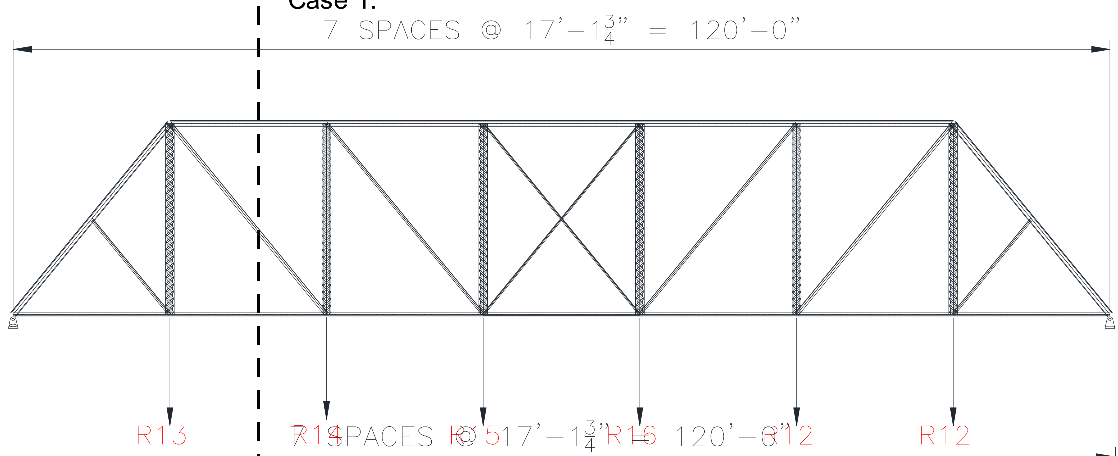
$$R_{15} := (1 + IM)\cdot\left[2P_6\cdot\left(\frac{2ft + 8in}{L_{panel\_1}}\right) + 2P_7\cdot\left[\left(\frac{L_{panel\_1} - 13ft - 5.875in}{L_{panel\_1}}\right) + \left(\frac{L_{panel\_1} - 5ft - 5.875in}{L_{panel\_1}}\right) + \left(\frac{L_{panel\_1} -}{L_p}\right.\right.\right.$$

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

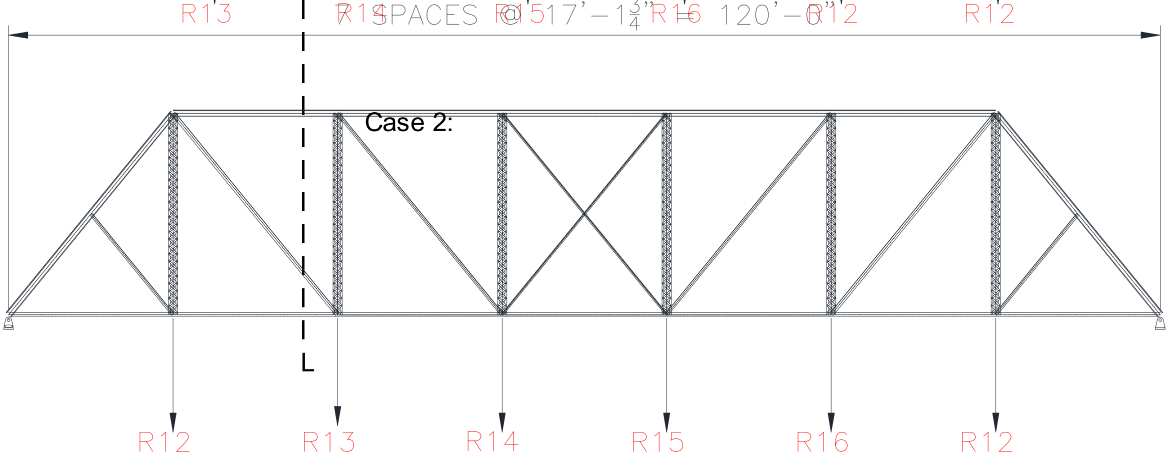
$$R_{16} := (1 + IM)\cdot\left[2P_7\cdot\left[\left(\frac{13ft + 5.875in}{L_{panel\_1}}\right) + \left(\frac{5ft + 5.875in}{L_{panel\_1}}\right) + \left(\frac{9ft + 5.875in}{L_{panel\_1}}\right)\right]\right] + w_{ped\_LL}\cdot L_{panel\_1}\cdot 5ft = 38.63\cdot k$$

West Truss

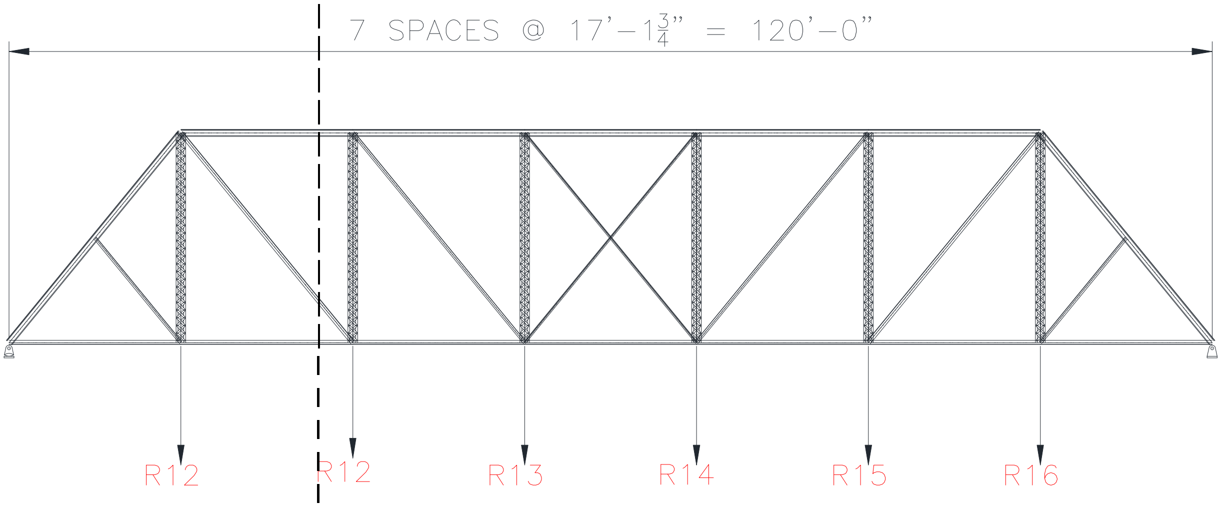
Case 1:



Case 2:



Case 3:



Truss forces analyzed in STAAD per AASHTO LRFR Strength I

Member			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	329	0.82
2C9x13.25 (w/ 3/8x15 PL)	U1-U2	TC	12.6	205.75	3.56	57.79	293.09	358.25	348	1.19
2C9x13.25 (w/ 3/8x15 PL)	U2-U3	TC	12.6	205.75	3.56	57.79	293.09	358.25	411	1.40
2C9x13.25 (w/ 3/8x15 PL)	U3-U4	TC	12.6	205.75	3.56	57.79	293.09	358.25	407.3	1.39
2L5x3.5x9/16	L0-L1	BC	11.6	205.75	0.97	212.11	58.19	330.60	-207.9	0.63
2L5x3.5x9/16	L1-L2	BC	11.6	205.75	0.97	212.11	58.19	330.60	-207.9	0.63
2L5x3.5x5/16 + 2L5x3.5x3/8	L2-L3	BC	11.2	205.75	1.02	201.72	62.23	319.77	-348	1.09
4L5x3.5x7/16	L3-L4	BC	10.2	205.75	1.02	201.72	56.80	291.84	-414.6	1.42
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

$$\gamma DL_{truss\_6} := 1.25 \cdot 108.2\mathbf{k}$$

L3-L4 LL

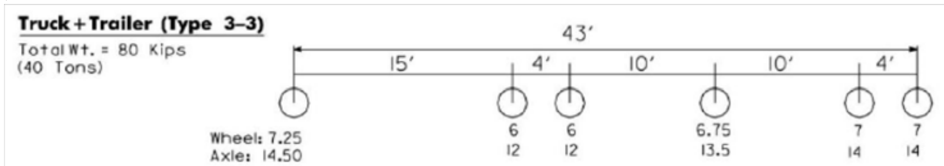
$$\gamma LL_{truss\_6} := 1.75 \cdot \mathbf{159.6k}$$

Type 3S3 Truck Rating

$$\boxed{Type\_3S3 := 40\mathbf{ton} \cdot \frac{C_{truss\_6} - \gamma DL_{truss\_6}}{\gamma LL_{truss\_6}} = 16.2 \cdot \mathbf{ton}}$$

L

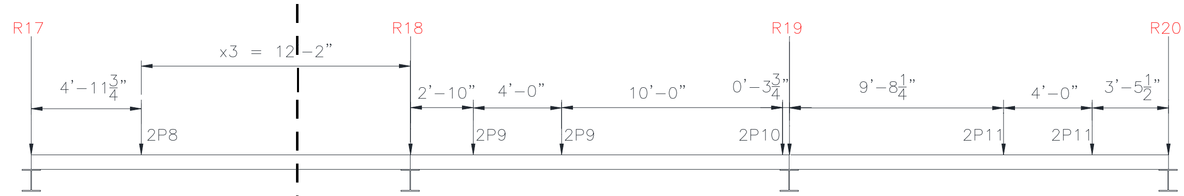
Type 3-3 Truck



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

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



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

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

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

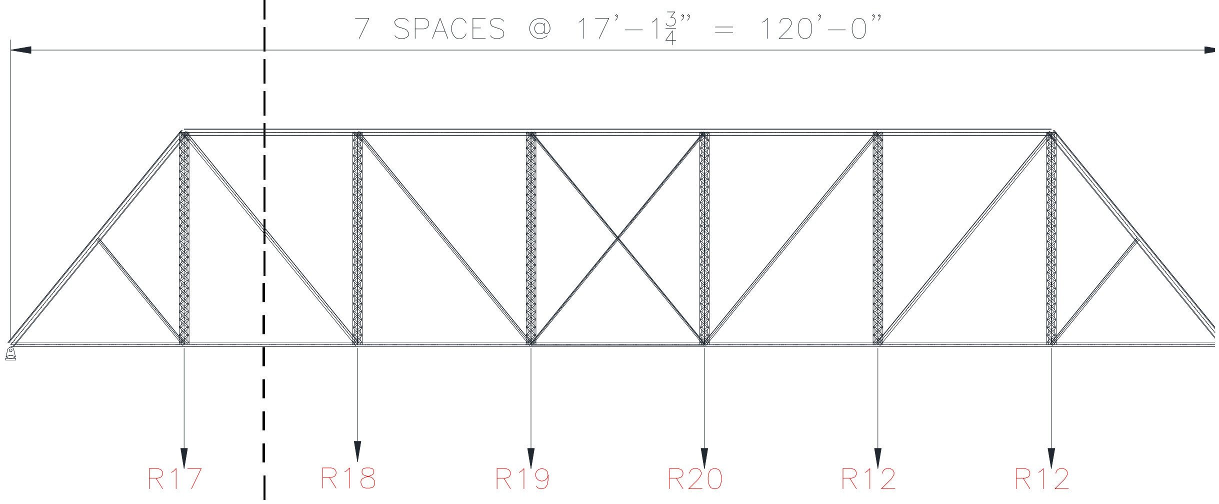
$$\mathbf{R_{19}} := (1 + \mathbf{IM}) \cdot \left[2\mathbf{P_{11}} \cdot \left[\left(\frac{\mathbf{L_{panel\_1}} - 9\mathbf{ft} - 8.25\mathbf{in}}{\mathbf{L_{panel\_1}}} \right) + \left(\frac{\mathbf{L_{panel\_1}} - 13\mathbf{ft} - 8.25\mathbf{in}}{\mathbf{L_{panel\_1}}} \right) \right] + 2\mathbf{P_9}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

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

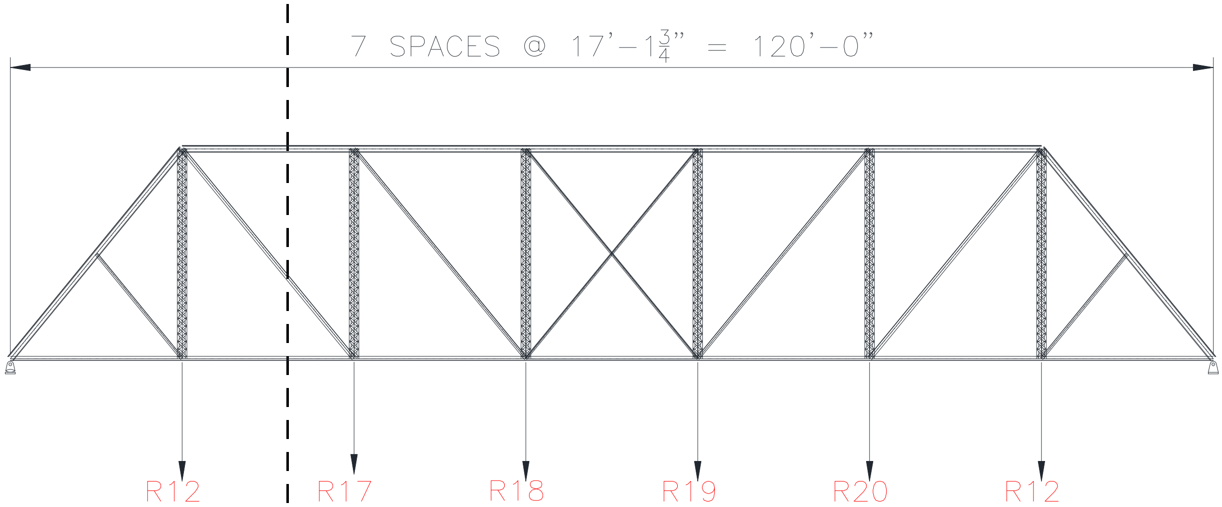
$$\mathbf{R_{20}} := (1 + \mathbf{IM}) \cdot \left[2\mathbf{P_{11}} \cdot \left[\left(\frac{9\mathbf{ft} + 8.25\mathbf{in}}{\mathbf{L_{panel\_1}}} \right) + \left(\frac{13\mathbf{ft} + 8.25\mathbf{in}}{\mathbf{L_{panel\_1}}} \right) \right] \right] + \mathbf{w_{ped\_LL}} \cdot \mathbf{L_{panel\_1}} \cdot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

West Truss

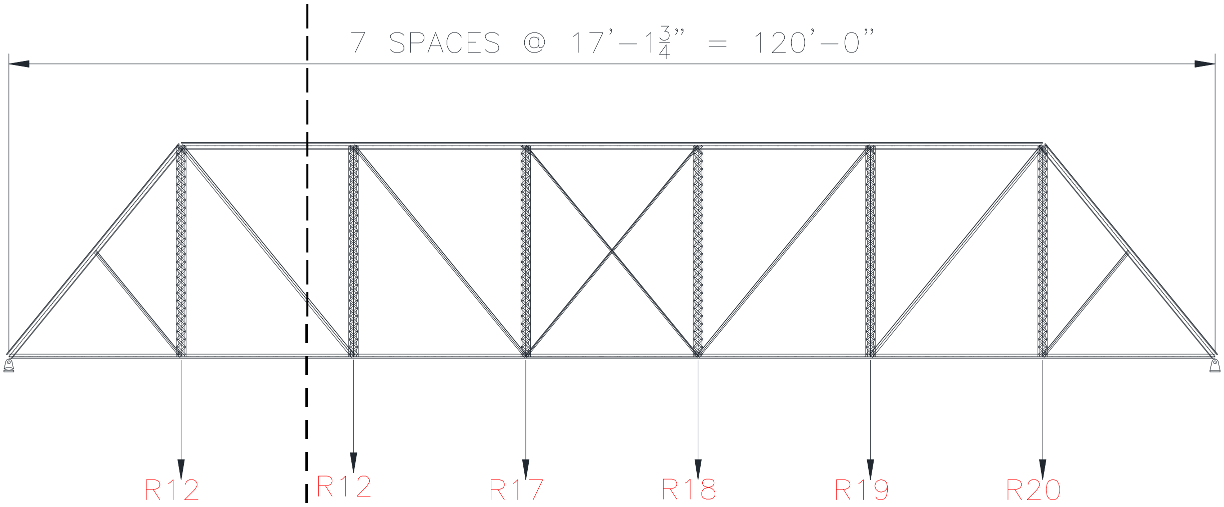
Case 1:



Case 2:



Case 3:



Truss forces analyzed in STAAD per AASHTO LRFR Strength I

							Allowable Comp.	Allowable Tension	ACTUAL LOAD	
			A_g	L	r		$ø_cP_n$	$ø_yF_yA_g$	$?_iP_u$	
			(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	-404	1.38
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

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

L3-L4 DL

$$\mathbf{\gamma DL_{truss\_7}} := 1.25 \cdot 108.2\mathbf{k}$$

L3-L4 LL

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

Type 3-3 Truck Rating

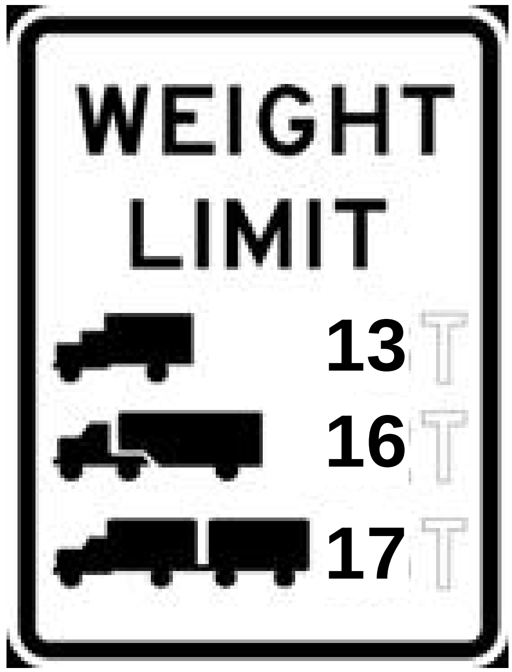
$$\boxed{\mathbf{Type\_3\_3} := 40\mathbf{ton} \cdot \frac{\mathbf{C_{truss\_7}} - \mathbf{\gamma DL_{truss\_7}}}{\mathbf{\gamma LL_{truss\_7}}} = 16.8 \cdot \mathbf{ton}}$$

Legal Load Rating Summary

$\mathbf{Type\_4} = 13 \cdot \mathbf{ton}$

$\mathbf{Type\_3S3} = 16 \cdot \mathbf{ton}$

$\mathbf{Type\_3\_3} = 17 \cdot \mathbf{ton}$



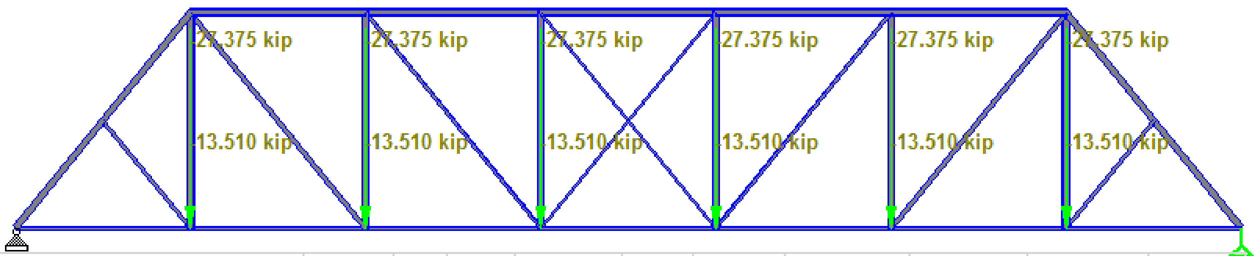
SIDEWALK ONLY

Pedestrian Live Load

LL Panel Point Load (west) $\mathbf{LL}_{\mathbf{ped\_3}} := 5\mathbf{ft} \cdot \mathbf{L}_{\mathbf{panel\_1}} \cdot \mathbf{w}_{\mathbf{ped\_LL}} = 7.72 \cdot \mathbf{k}$

West Truss

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



							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	158.3	0.39
2C9x13.25 (w/ 3/8x15 PL)	U1-U2	TC	12.6	205.75	3.56	57.79	293.09	358.25	166.9	0.57
2C9x13.25 (w/ 3/8x15 PL)	U2-U3	TC	12.6	205.75	3.56	57.79	293.09	358.25	200.3	0.68
2C9x13.25 (w/ 3/8x15 PL)	U3-U4	TC	12.6	205.75	3.56	57.79	293.09	358.25	198.5	0.68
2L5x3.5x9/16	L0-L1	BC	11.6	205.75	0.97	212.11	58.19	330.60	-100.1	0.30
2L5x3.5x9/16	L1-L2	BC	11.6	205.75	0.97	212.11	58.19	330.60	-100.1	0.30
2L5x3.5x5/16 + 2L5x3.5x3/8	L2-L3	BC	11.2	205.75	1.02	201.72	62.23	319.77	-166.9	0.52
4L5x3.5x7/16	L3-L4	BC	10.2	205.75	1.02	201.72	56.80	291.84	-202	0.69
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 $\mathbf{C}_{\mathbf{truss\_8}} := \mathbf{min}\left(0.85, \varphi_{c\_truss\_1} \cdot \varphi_{s\_truss\_1}\right) \cdot 291.8\mathbf{k}$

L3-L4 DL $\gamma \mathbf{DL}_{\mathbf{truss\_8}} := 1.25 \cdot 108.2\mathbf{k}$

L3-L4 LL $\gamma \mathbf{LL}_{\mathbf{truss\_8}} := 1.75 \cdot 38.1\mathbf{k}$

Inventory Rating $\boxed{\mathbf{INV}_{\mathbf{truss\_8}} := \mathbf{w}_{\mathbf{ped\_LL}} \cdot \frac{\mathbf{C}_{\mathbf{truss\_8}} - \gamma \mathbf{DL}_{\mathbf{truss\_8}}}{\gamma \mathbf{LL}_{\mathbf{truss\_8}}} = 118.75 \cdot \mathbf{psf}}$

Operating Rating $\boxed{\mathbf{OPR}_{\mathbf{truss\_8}} := \mathbf{w}_{\mathbf{ped\_LL}} \cdot \frac{\mathbf{C}_{\mathbf{truss\_8}} - \gamma \mathbf{DL}_{\mathbf{truss\_8}}}{\gamma \mathbf{LL}_{\mathbf{truss\_8}} \cdot \left(\frac{1.35}{1.75}\right)} = 153.94 \cdot \mathbf{psf}}$

APPENDIX C

Rehabilitation Cost Opinions

3rd Street Bridge
Option 2 - Rehabilitate for Pedestrian Use

COST ESTIMATE SHEET

Description: Rehabilitate bridge to accommodate pedestrian use.

Date: 7/20/2015
Est. By: TJM
Check By:

ITEM #	ITEM CODE	BID ITEM DESCRIPTION	QUANTITY	UNIT	RATE	TOTAL
1	2212-5070310	PATCH, FULL-DEPTH REPAIR	200	SY	101.18	20236.00
2	2301-1033100	STD/S-F PCC PAV'T, CL C CL 3, 10"	500	SY	45.47	22735.00
3	2401-6750001	REMVL (DECK & S.W.)	1	LS	40000	40000.00
4	2403-0100010	STRUCT CONC (BRIDGE)	50	CY	476.26	23813.00
5	2404-7775000	REINFORC STEEL	8000	LB	0.91	7280.00
6	2408-6772011	REPAIR BEAM, HEAT STRAIGHTEN	3	EA	29740.47992	89221.44
7	2408-7800000	STRUCTURAL STEEL	3100	LB	6	18600.00
8	2409-4575001	TREATED TIMBER+LUMBER	18.2	MFBM	9073.901905	165145.01
9	2501-8400172	TEMP SHORING	1	LS	200000	200000.00
10	2507-3250005	ENGINEER FABRIC	5000	SY	2.8	14000.00
11	2507-6800061	REVETMENT, CLASS E	4000	TON	39.04	156160.00
12	2533-4980005	MOBILIZATION	1	LS	113578.5682	113578.57
13		ENGINEERING SERVICES	1	LS	174153.8045	174153.80
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					Total Cost $	1044922.83

3rd Street Bridge

Option 3 - Rehabilitate for Vehicular Use

COST ESTIMATE SHEET

Description: Rehabilitate bridge to accommodate vehicular and pedestrian use.

Date: 7/21/2015
Est. By: TJM
Check By:

ITEM #	ITEM CODE	BID ITEM DESCRIPTION	QUANTITY	UNIT	RATE	TOTAL
1	2212-5070310	PATCH, FULL-DEPTH REPAIR	200	SY	101.18	20236.00
2	2301-1033100	STD/S-F PCC PAV'T, CL C CL 3, 10"	500	SY	45.47	22735.00
3	2403-0100010	STRUCT CONC (BRIDGE)	50	CY	476.26	23813.00
4	2404-7775000	REINFORC STEEL	8000	LB	0.91	7280.00
5	2408-6772011	REPAIR BEAM, HEAT STRAIGHTEN	3	EA	29740.47992	89221.44
6	2408-7800000	STRUCTURAL STEEL	120000	LB	6	720000.00
7	2501-8400172	TEMP SHORING	1	LS	200000	200000.00
8	2507-3250005	ENGINEER FABRIC	5000	SY	2.8	14000.00
9	2507-6800061	REVETMENT, CLASS E	4000	TON	39.04	156160.00
10	2533-4980005	MOBILIZATION	1	LS	188016.816	188016.82
11		ENGINEERING SERVICES	1	LS	288292.4511	288292.45
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					Total Cost $	1729754.71

3rd Street Bridge

Option 4 - Replace w/ New Pedestrian Bridge

COST ESTIMATE SHEET

Description: Replace with 3 span Steel Truss bridge, 360' x14' for pedestrian use.

Date: 7/21/2015
Est. By: TJM
Check By:

ITEM #	ITEM CODE	BID ITEM DESCRIPTION	QUANTITY	UNIT	RATE	TOTAL
1	2401-6745650	RMVL OF EXIST STRUCT	1	LS	65000	65000.00
2	2402-2720000	EXCAVATION, CL 20	500	CY	20.45	10225.00
3	2403-0100010	STRUCT CONC (BRIDGE)	840	CY	476.26	400058.40
4	2404-7775000	REINFORC STEEL	143000	LB	0.91	130130.00
5	2429-0000100	PRE-ENGINEERED STEEL TRUSS TRAIL BRDG,__	3	EACH	160000	480000.00
6	2501-0201057	PILE, STEEL, HP 10X57	1200	LF	40.66	48792.00
7	2507-3250005	ENGINEER FABRIC	3000	SY	2.8	8400.00
8	2507-6800061	REVETMENT, CLASS E	2500	TON	39.04	97600.00
9	2533-4980005	MOBILIZATION	1	LS	186030.81	186030.81
10		ENGINEERING SERVICES	1	LS	285247.242	285247.24
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					Total Cost $	1711483.45

3rd Street Bridge

Option 5 - Replace In-Kind

COST ESTIMATE SHEET

Description: Replace with 3 span Steel Truss bridge, 360' x40' for vehicular and pedestrian use.

Date: 7/20/2015
Est. By: TJM
Check By:

ITEM #	ITEM CODE	BID ITEM DESCRIPTION	QUANTITY	UNIT	RATE	TOTAL
1	2301-1033100	STD/S-F PCC PAV'T, CL C CL 3, 10"	500	SY	45.47	22735.00
2	2401-6745650	RMVL OF EXIST STRUCT	1	LS	65000	65000.00
3	2402-2720000	EXCAVATION, CL 20	500	CY	20.45	10225.00
4	2403-0100010	STRUCT CONC (BRIDGE)	1200	CY	476.26	571512.00
5	2404-7775000	REINFORC STEEL	204000	LB	0.91	185640.00
6	2408-7800000	STRUCTURAL STEEL	500000	LB	1.8	900000.00
7	2414-6424110	CONC BARRIER RAIL	720	LF	51.23	36885.60
8	2414-6445100	STRUCTURAL STEEL PEDESTRIAN HAND RAIL	360	LF	110.94	39938.40
9	2414-6625502	STRUCT STEEL RAIL, TRAFFIC	360	LF	60	21600.00
10	2501-0201057	PILE, STEEL, HP 10X57	3000	LF	40.66	121980.00
11	2507-3250005	ENGINEER FABRIC	5000	SY	2.8	14000.00
12	2507-6800061	REVETMENT, CLASS E	4000	TON	39.04	156160.00
13	2533-4980005	MOBILIZATION	1	LS	321851.4	321851.40
15		ENGINEERING SERVICES	1	LS	493505.48	493505.48
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					Total Cost $	2961032.88

3rd Street Bridge

Option 6 - Replace w/ PPCB Bridge

COST ESTIMATE SHEET

Description: Replace with 3 span PPCB bridge, 360'x40' for vehicular and pedestrian use.

Date: 7/20/2015
Est. By: TJM
Check By:

ITEM #	ITEM CODE	BID ITEM DESCRIPTION	QUANTITY	UNIT	RATE	TOTAL
1	2301-1033100	STD/S-F PCC PAV'T, CL C CL 3, 10"	500	SY	45.47	22735.00
2	2401-6745650	RMVL OF EXIST STRUCT	1	LS	65000	65000.00
3	2402-2720000	EXCAVATION, CL 20	500	CY	20.45	10225.00
4	2403-0100010	STRUCT CONC (BRIDGE)	1300	CY	476.26	619138.00
5	2404-7775000	REINFORC STEEL	207000	LB	0.91	188370.00
6	2407-0563120	BEAM, PPC, BTC120	18	EACH	26125	470250.00
7	2408-7800000	STRUCTURAL STEEL	4800	LB	1.31	6288.00
8	2414-6424110	CONC BARRIER RAIL	720	LF	51.23	36885.60
9	2414-6445100	STRUCTURAL STEEL PEDESTRIAN HAND RAIL	360	LF	110.94	39938.40
10	2414-6625502	STRUCT STEEL RAIL, TRAFFIC	360	LF	60	21600.00
11	2501-0201057	PILE, STEEL, HP 10X57	3000	LF	40.66	121980.00
12	2507-3250005	ENGINEER FABRIC	5000	SY	2.8	14000.00
13	2507-6800061	REVETMENT, CLASS E	4000	TON	39.04	156160.00
14	2533-4980005	MOBILIZATION	1	LS	265885.5	265885.50
15		ENGINEERING SERVICES	1	LS	407691.1	407691.10
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					Total Cost $	2446146.60

APPENDIX D

2008 FEMA FIS

FLOODING SOURCE		FLOODWAY			1-PERCENT-ANNUAL-CHANCE FLOOD WATER SURFACE ELEVATION			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQ.FEET)	MEAN VELOCITY (FEET/SEC.)	REGULATORY (FEET NAVD)	WITHOUT FLOODWAY (FEET NAVD)	WITH FLOODWAY (FEET NAVD)	INCREASE (FEET)
CEDAR RIVER								
A	219.24	480	7,365	5.6	883.9	883.9	884.8	0.9
B	219.33	325	4,668	8.8	883.9	883.9	884.8	0.9
C	219.42	328	4,872	8.4	884.8	884.8	885.5	0.7
D	219.52	430	6,771	6.1	886.2	886.2	886.7	0.5
E	219.74	508	7,201	5.7	887.0	887.0	887.4	0.4
F	220.03	670	9,481	4.3	888.3	888.3	888.7	0.4
G	220.62	420	7,566	5.4	889.3	889.3	889.8	0.5
H	221.89	1,100	14,834	2.8	891.5	891.5	892.2	0.7
I	222.76	1,650	17,935	2.3	892.8	892.8	893.6	0.8
J	223.37	930	10,559	3.9	893.8	893.8	894.6	0.8
K	223.90	1,749	17,227	2.4	895.2	895.2	896.1	0.9
L	224.35	2,180	19,210	2.1	895.9	895.9	896.8	0.9
M	225.06	1,200	13,746	3.0	897.0	897.0	897.9	0.9
N	225.65	440	6,980	5.6	898.6	898.6	899.5	0.9
O	226.12	1,340	15,682	2.5	900.0	900.0	901.0	1.0
P	226.47	560	9,028	4.3	900.4	900.4	901.4	1.0
Q	227.14	780	10,374	3.0	901.8	901.8	902.8	1.0
R	227.73	361	7,054	5.5	902.9	902.9	903.9	1.0
S	228.10	900	5,936	6.5	903.8	903.8	904.7	0.9
T	228.41	900	7,384	5.2	905.0	905.0	906.0	1.0
U	228.59	710	7,066	5.5	905.6	905.6	906.5	0.9
V	228.93	1,000	9,861	3.9	906.5	906.5	907.4	0.9
W	229.16	700	8,663	4.5	906.7	906.7	907.7	1.0
X	229.39	700	10,492	3.7	907.3	907.3	908.2	0.9
Y	229.82	950	12,866	3.0	908.0	908.0	908.9	0.9
Z	230.08	470	8,336	4.6	908.6	908.6	909.5	0.9

[1]MILES ABOVE MOUTH

TABLE 3

FEDERAL EMERGENCY MANAGEMENT AGENCY

BREMER COUNTY, IA

AND INCORPORATED AREAS

FLOODWAY DATA

CEDAR RIVER

FLOODING SOURCE		FLOODWAY			1-PERCENT-ANNUAL-CHANCE FLOOD WATER SURFACE ELEVATION			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQ.FEET)	MEAN VELOCITY (FEET/SEC.)	REGULATORY (FEET NAVD)	WITHOUT FLOODWAY (FEET NAVD)	WITH FLOODWAY (FEET NAVD)	INCREASE (FEET)
CEDAR RIVER								
AA	230.26	550	9,216	4.2	909.0	909.0	909.8	0.8
AB	230.52	346	5,709	6.8	909.5	909.5	910.3	0.8
AC	230.67	510	8,259	4.7	915.2	915.2	915.3	0.1
AD	230.90	960	13,972	2.8	915.9	915.9	916.2	0.3
AE	231.00	1,028	15,652	2.5	916.0	916.0	916.3	0.3
AF	231.18	1,130	16,295	2.4	916.1	916.1	916.4	0.3
AG	231.56	1,050	11,610	3.3	916.4	916.4	916.6	0.2
AH	232.09	605	9,253	4.2	917.3	917.3	917.8	0.5
AI	232.41	1,250	14,655	2.6	917.7	917.7	918.4	0.7
AJ	232.70	1,150	13,400	2.9	917.9	917.9	918.6	0.7
AK	233.02	1,300	18,936	2.0	918.5	918.5	919.1	0.6
AL	233.87	1,710	20,874	1.8	919.1	919.1	919.8	0.7
AM	234.26	1,650	18,437	2.1	919.7	919.7	920.4	0.7
AN	235.11	1,790	19,746	2.0	920.9	920.9	921.7	0.8
AO	236.33	1,190	13,638	2.8	922.8	922.8	923.7	0.9
AP	236.56	1,170	13,179	2.9	923.4	923.4	924.3	0.9
AQ	236.91	369	5,691	6.7	924.1	924.1	925.0	0.9
AR	237.05	990	13,693	2.8	925.3	925.3	925.9	0.6

[1]MILES ABOVE MOUTH

TABLE 3

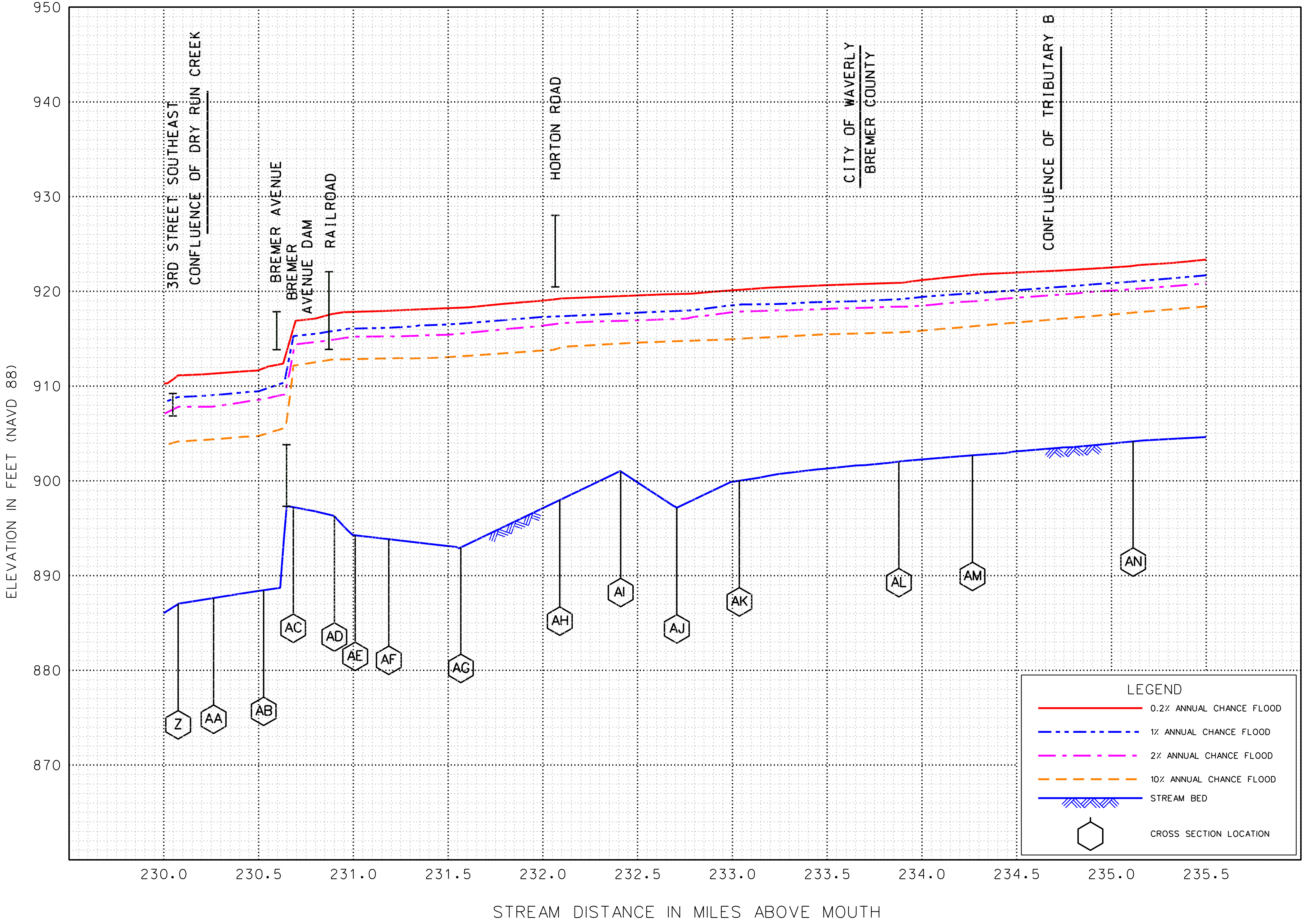
FEDERAL EMERGENCY MANAGEMENT AGENCY

BREMER COUNTY, IA

AND INCORPORATED AREAS

FLOODWAY DATA

CEDAR RIVER



FLOOD PROFILES

CEDAR RIVER

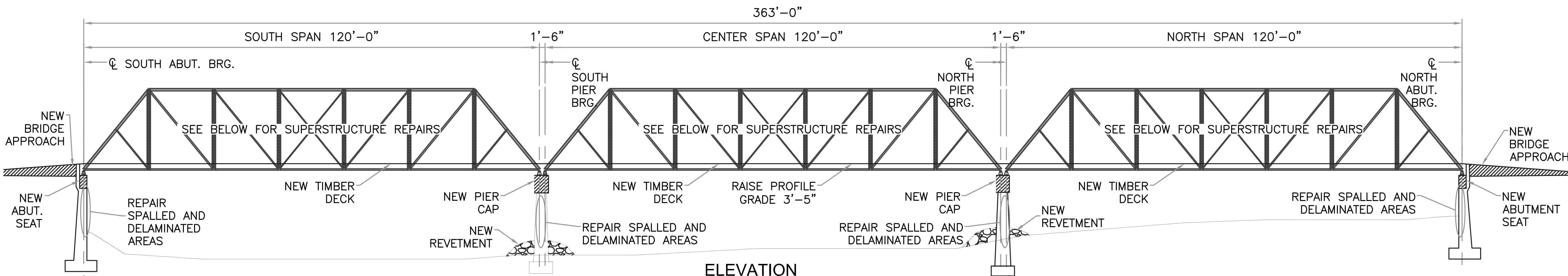
FEDERAL EMERGENCY MANAGEMENT AGENCY

BREMER COUNTY, IOWA AND INCORPORATED AREAS

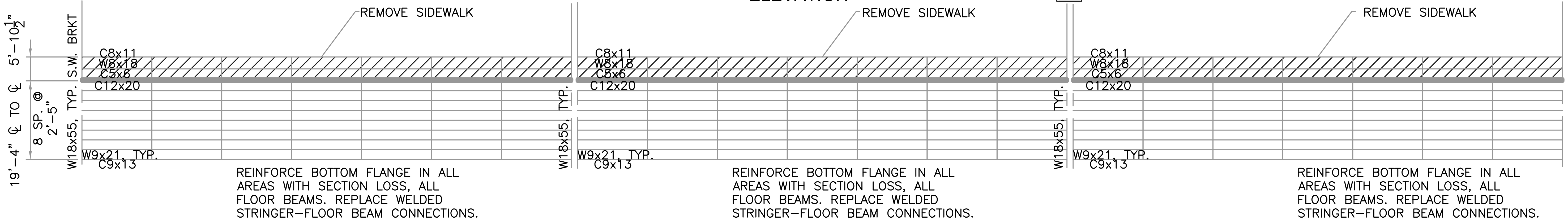
03P

APPENDIX E

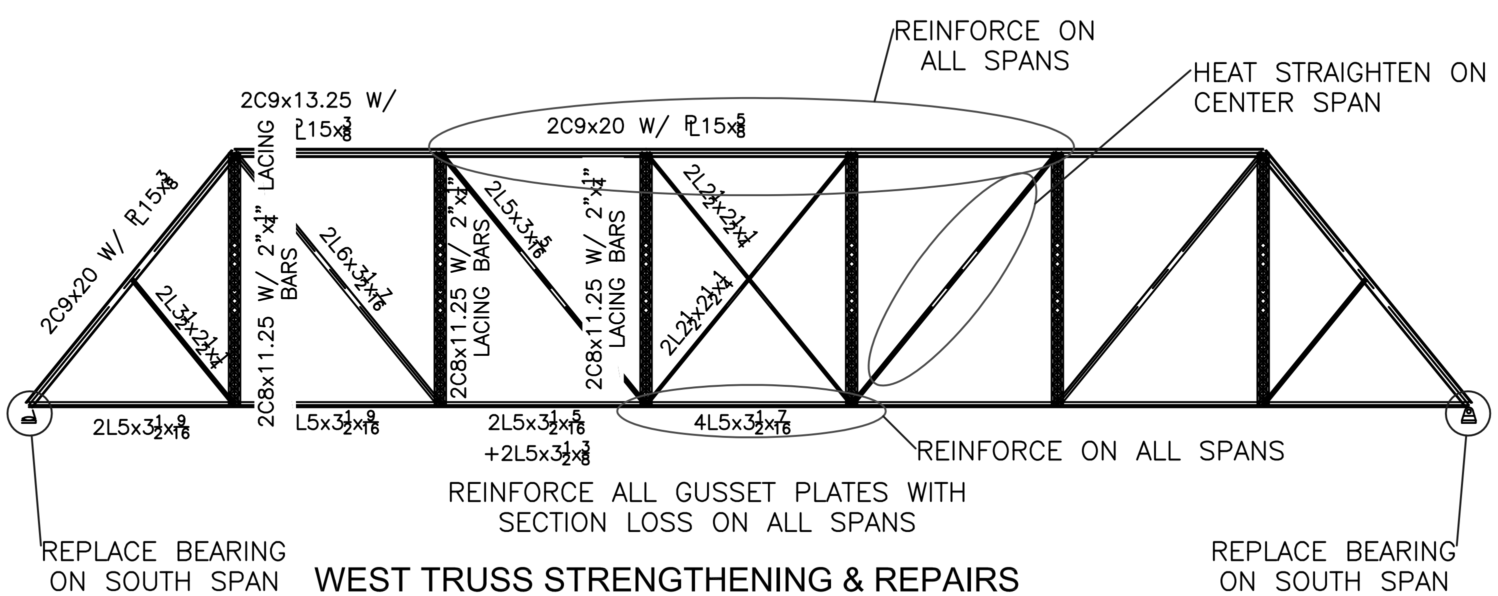
DRAWINGS



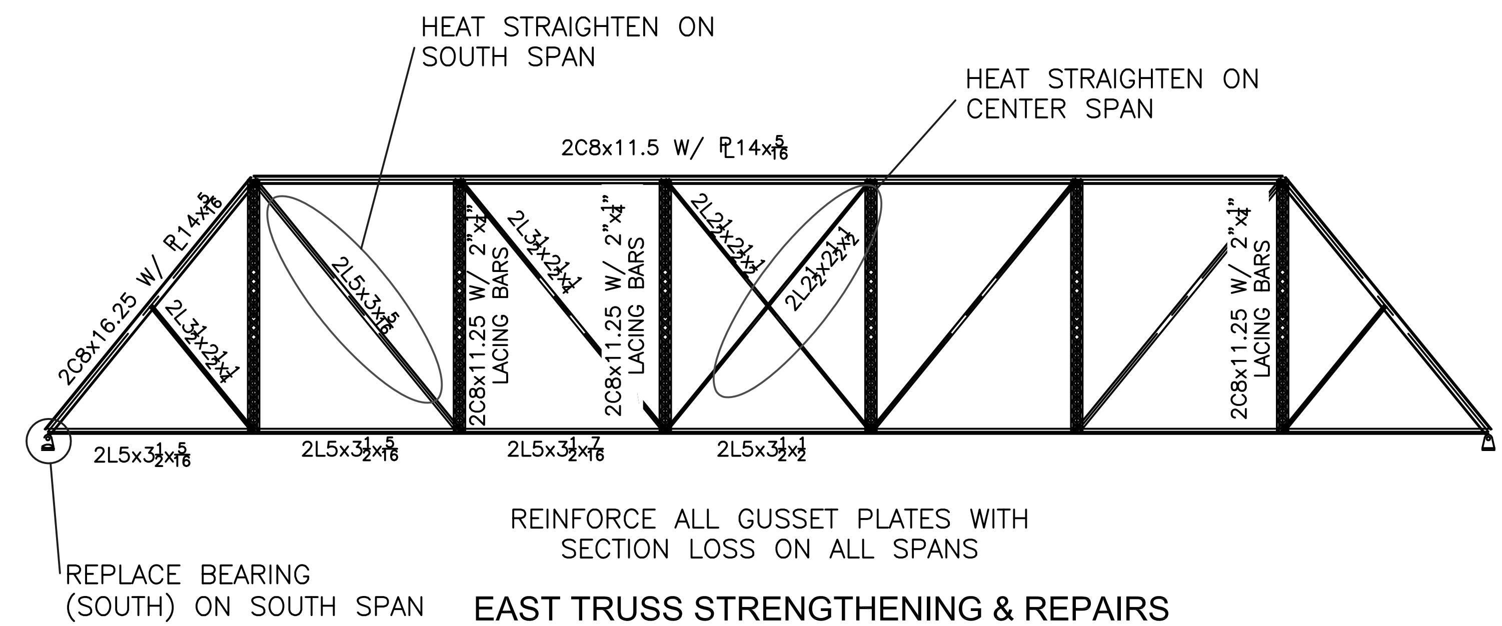
ELEVATION



FLOOR SYSTEM REPAIRS



WEST TRUSS STRENGTHENING & REPAIRS



EAST TRUSS STRENGTHENING & REPAIRS

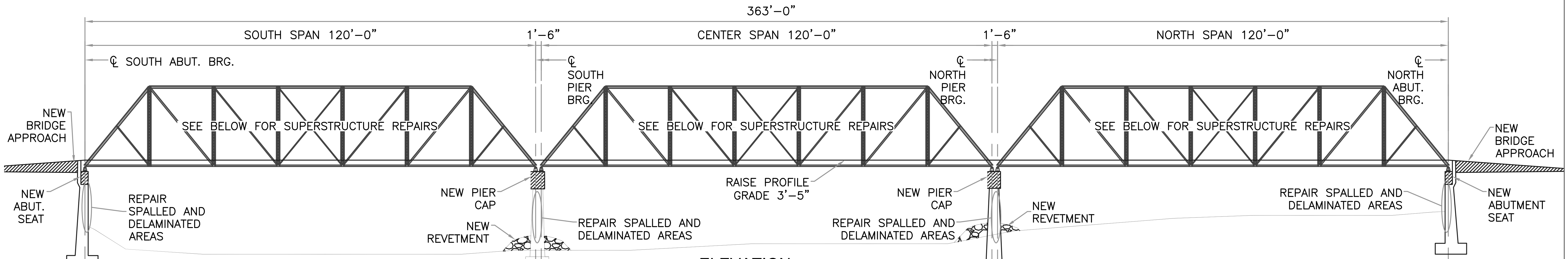
363'-0 x 19'-4 STEEL TRUSS BRIDGE
3RD STREET SE OVER CEDAR RIVER
CITY OF WAVERLY

OPTION 2
REHABILITATE FOR PEDESTRIAN USE

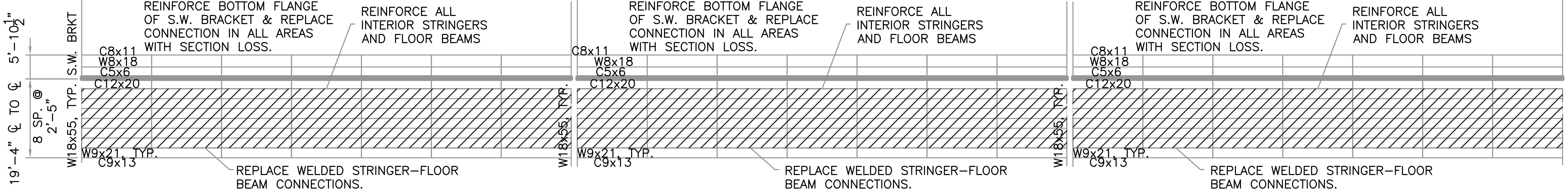
VJ Engineering
2570 Holiday Road, Suite 10
Coralville, Iowa – 319-338-4939

JOB NO.	15-3055
DESIGNED BY	TJM
DRAWN BY	TJM

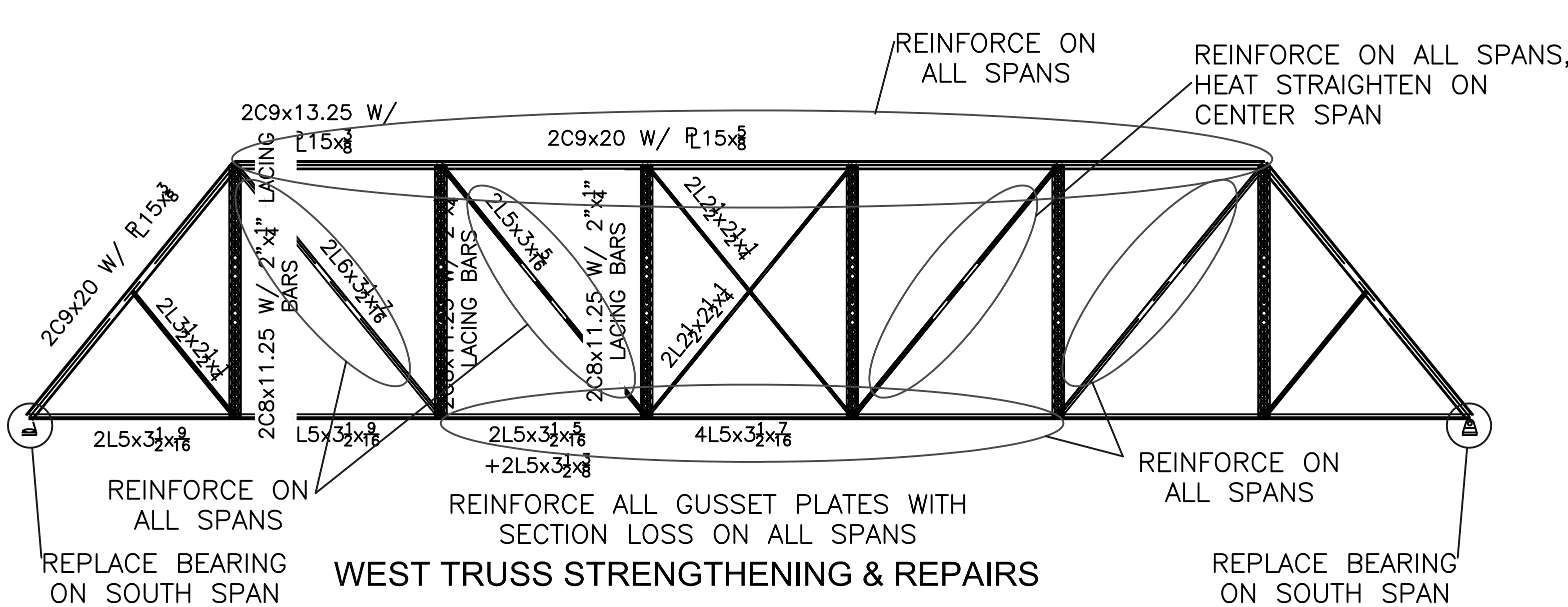
COUNTY	PROJECT NUMBER	SHEET NUMBER
BREMER	15-3055	1



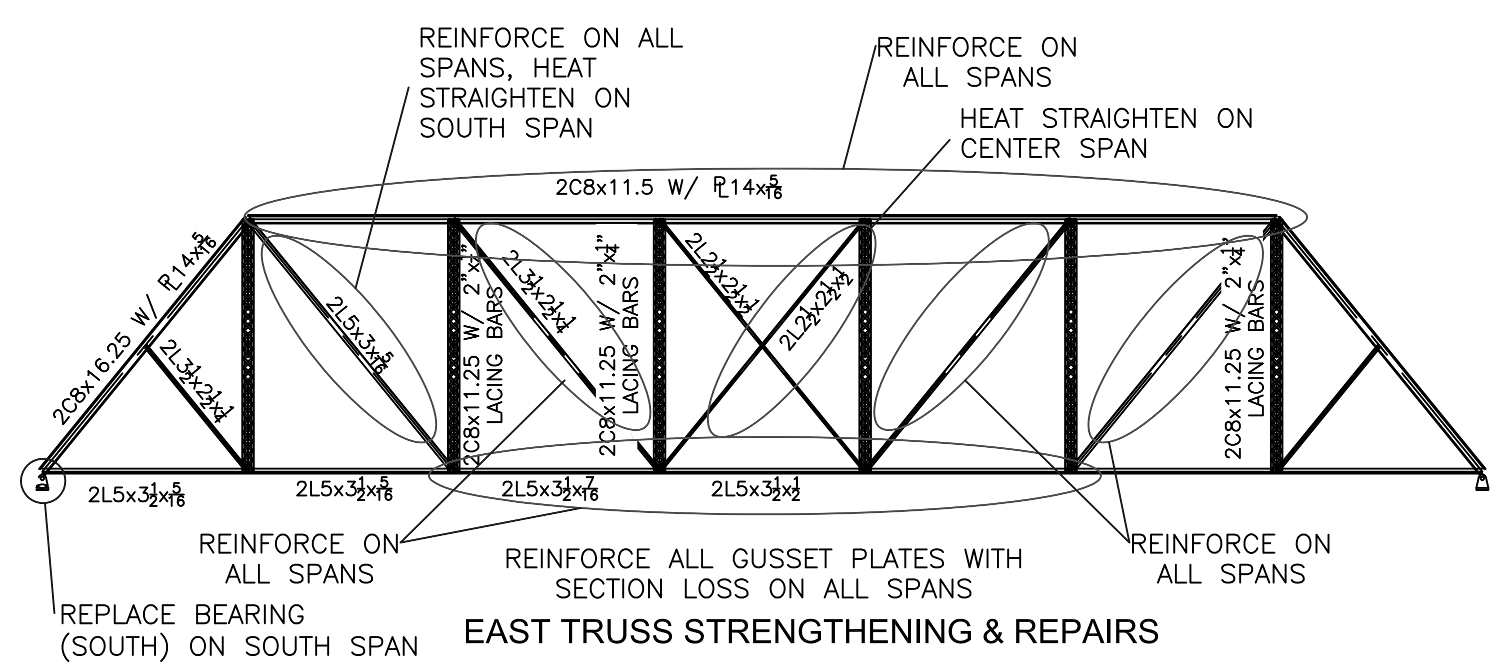
ELEVATION



FLOOR SYSTEM REPAIRS



WEST TRUSS STRENGTHENING & REPAIRS



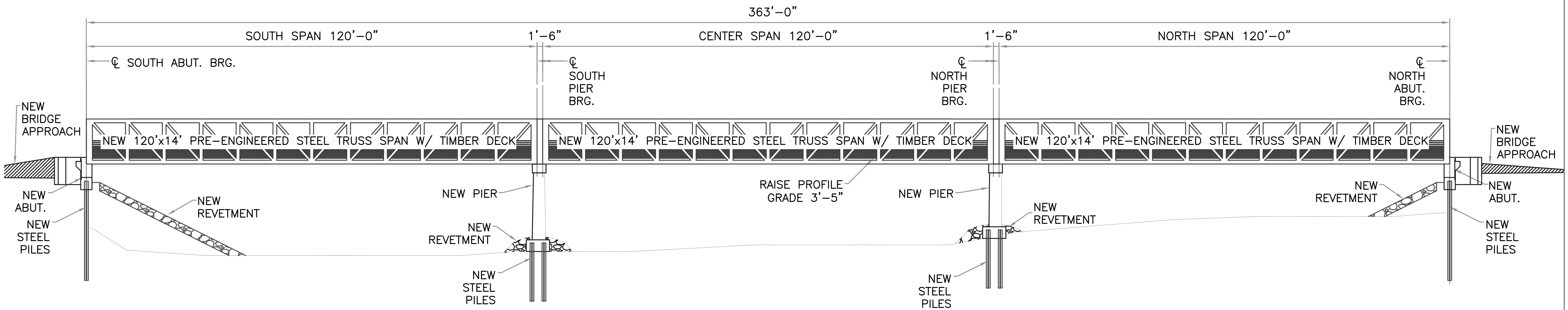
EAST TRUSS STRENGTHENING & REPAIRS

363'-0 x 19'-4 STEEL TRUSS BRIDGE
3RD STREET SE OVER CEDAR RIVER
CITY OF WAVERLY

OPTION 3
REHABILITATE FOR VEHICULAR & PEDESTRIAN USE

VJ Engineering 2570 Holiday Road, Suite 10 Coralville, Iowa - 319-338-4939	JOB NO. 15-3055	DESIGNED BY TJM	DRAWN BY TJM

COUNTY	PROJECT NUMBER	SHEET NUMBER
BREMER	15-3055	2



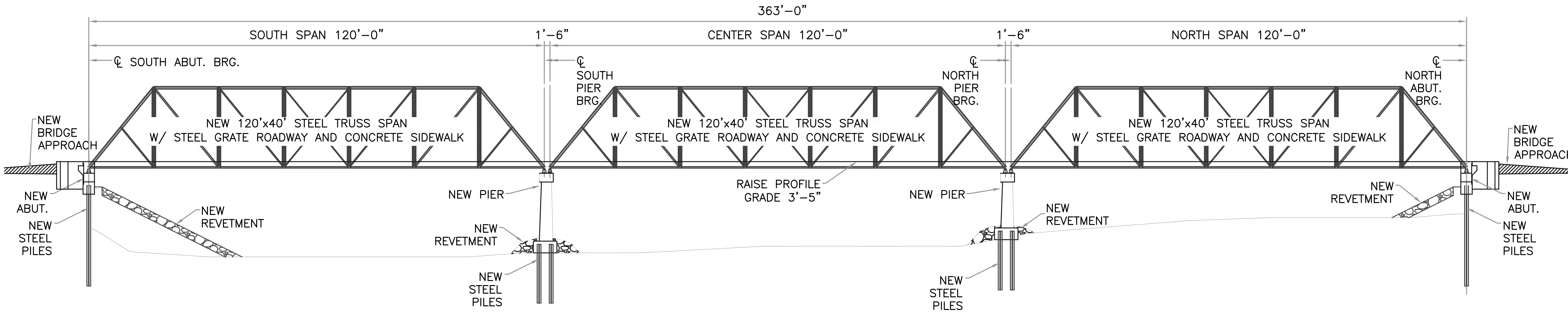
ELEVATION

363'-0 x 19'-4 STEEL TRUSS BRIDGE
3RD STREET SE OVER CEDAR RIVER
CITY OF WAVERLY
OPTION 4
REPLACE WITH NEW PRE-ENGINEERED
TRUSS BRIDGE FOR PEDESTRIAN USE

VJ Engineering
2570 Holiday Road, Suite 10
Coralville, Iowa – 319-338-4939

JOB NO.	15-3055
DESIGNED BY	TJM
DRAWN BY	TJM

COUNTY: BREMER | PROJECT NUMBER: 15-3055 | SHEET NUMBER: 3



ELEVATION

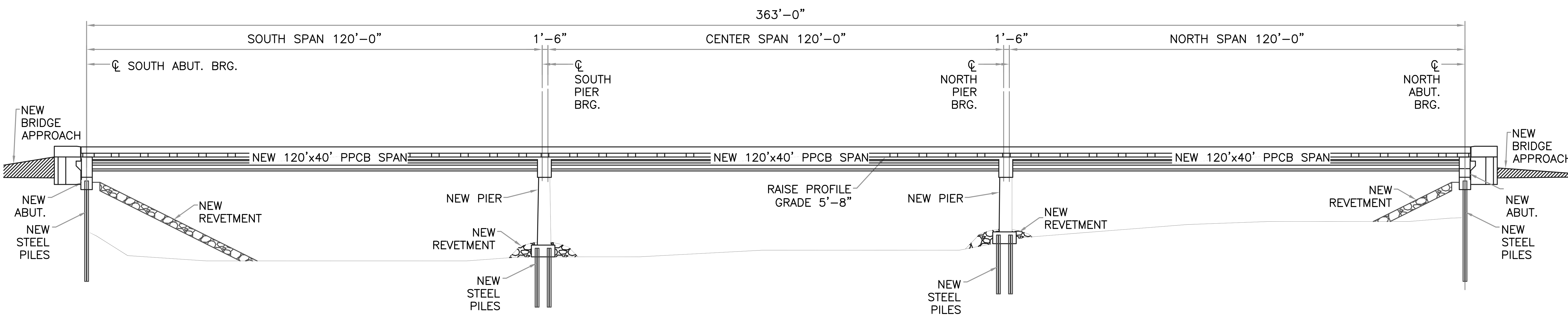
363'-0 x 19'-4 STEEL TRUSS BRIDGE
3RD STREET SE OVER CEDAR RIVER
CITY OF WAVERLY
OPTION 5

REPLACE WITH NEW STEEL TRUSS BRIDGE
FOR VEHICULAR & PEDESTRIAN USE

VJ Engineering
2570 Holiday Road, Suite 10
Coralville, Iowa – 319-338-4939

JOB NO.	15-3055
DESIGNED BY	TJM
DRAWN BY	TJM

COUNTY	PROJECT NUMBER	SHEET NUMBER
BREMER	15-3055	4



ELEVATION

363'-0 x 19'-4 STEEL TRUSS BRIDGE
3RD STREET SE OVER CEDAR RIVER
CITY OF WAVERLY
OPTION 6
REPLACE WITH NEW PPCB BRIDGE
FOR VEHICULAR & PEDESTRIAN USE

VJ Engineering
2570 Holiday Road, Suite 10
Coralville, Iowa – 319-338-4939

JOB NO.	15-3055
DESIGNED BY	TJM
DRAWN BY	TJM

COUNTY: BREMER | PROJECT NUMBER: 15-3055 | SHEET NUMBER: 5

RED HIGHLIGHT= REVISIONS MADE ON 10-23-15

PEDESTRIAN ONLY USE

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

Panel length: $L_{panel\_1} := 17.15ft$ $\quad F_y := 30ksi$ 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)

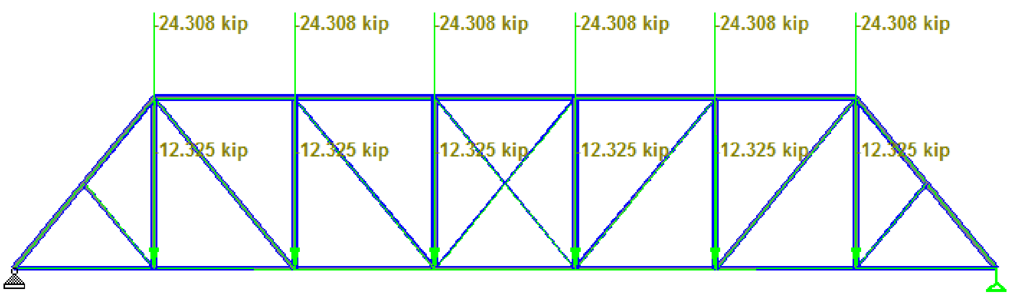
$$LL_{ped\_1} := \frac{W_{deck}}{2} \cdot L_{panel\_1} \cdot w_{ped\_LL} = 13.89 \cdot k$$

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 $\mathbf{C_{truss\_1}} := \mathbf{min}\left(0.85, \varphi_{c\_truss\_1} \cdot \varphi_{s\_truss\_1}\right) \cdot 228.3\mathbf{k}$

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

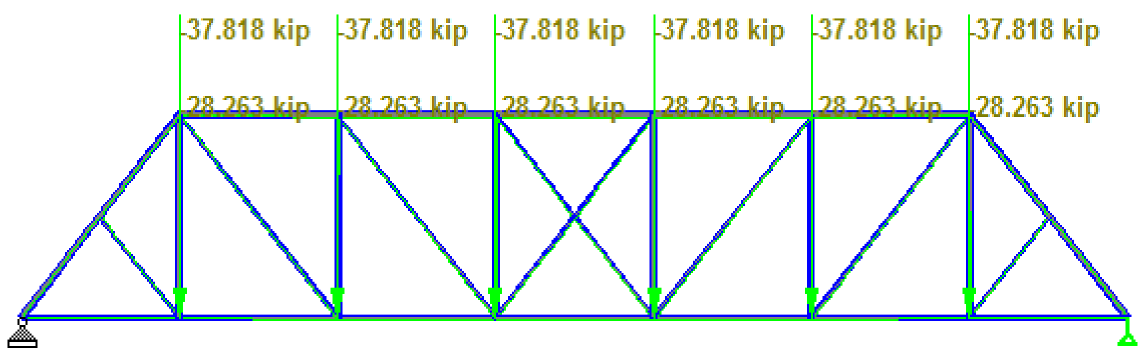
L2-L3 LL $\gamma\mathbf{LL_{truss\_1}} := 1.75 \cdot 56.7\mathbf{k}$

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

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

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
$$C_{truss\_2} := \min(0.85, \varphi_{c\_truss\_1} \cdot \varphi_{s\_truss\_1}) \cdot 291.8\mathbf{k}$$

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

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

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

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

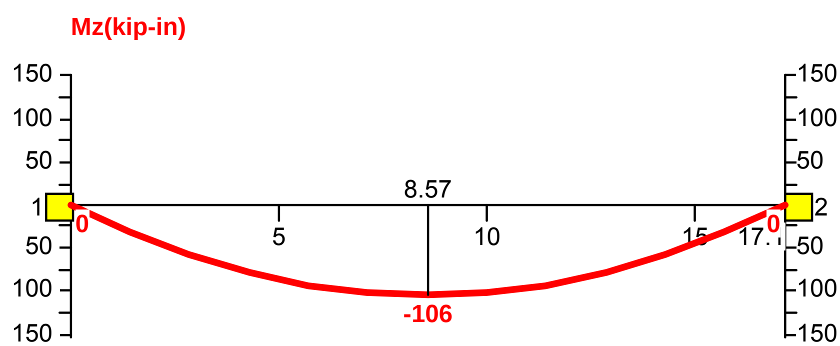
Floor System

Exterior Stringers:

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

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

Stringer forces analyzed in STAAD per AASHTO LRFR Pedestrian Strength I



East exterior stringer channels are C9x13.

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

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

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

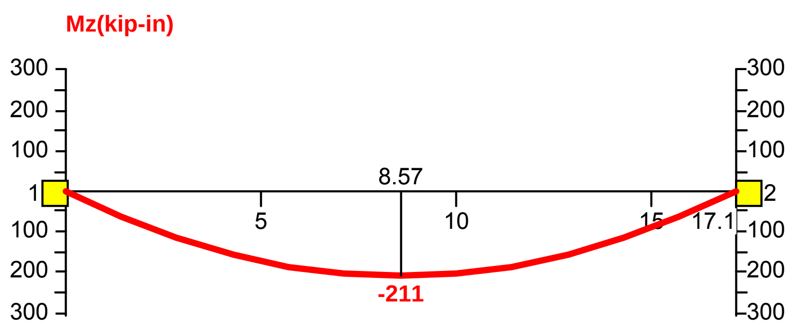
Operating Rating: $OPR_{ext\_strngr\_1} := w_{ped\_LL} \cdot \dfrac{C_{ext\_strngr\_1} - \gamma DL_{ext\_strngr\_1}}{\gamma LL_{ext\_strngr\_1} \cdot \left(\dfrac{1.35}{1.75}\right)} = 415.42 \cdot \mathbf{psf}$

Interior Stringers:

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

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

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

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

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

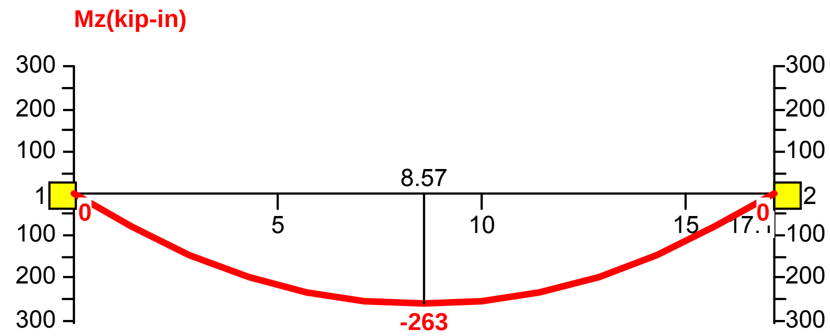
Distributed DL

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

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

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

Inventory Rating

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

Operating Rating

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

Floor Beams:

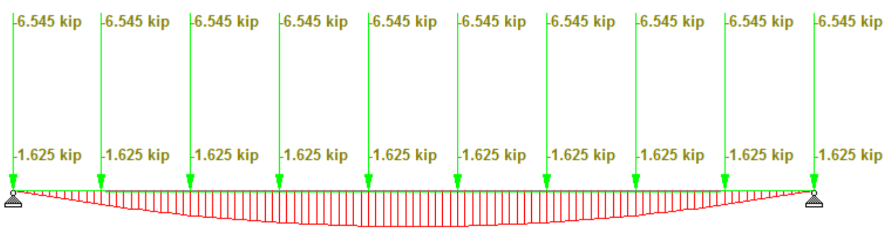
DL from stringers

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

LL from stringers

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

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



Floor beams are W18x55

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

Nominal flexural resistance

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

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

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

Floor Beam LL moment

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

Inventory Rating

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

Operating Rating

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

Truss Bearings:

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

Pin & Bearing plates

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

Nominal bearing resistance

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

Strength I truss reactions (from STAAD analysis)

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

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

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:

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

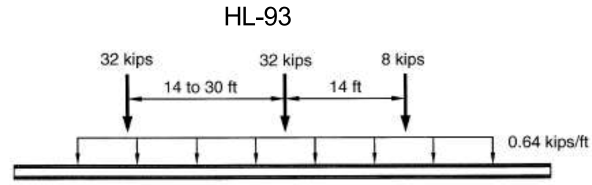
Inventory Rating $INV_{ped} = 40.3 \cdot psf$

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

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

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

Lane load

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

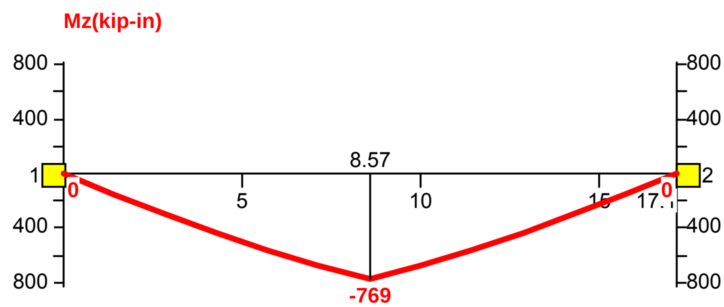
Floor System

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

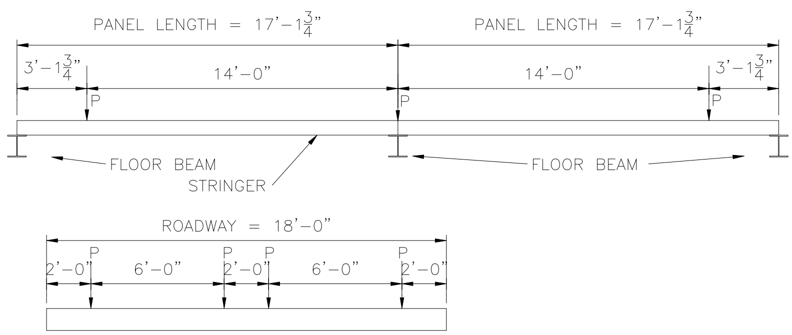
$$INV_{strngr\_2} := 1 \cdot \frac{C_{strngr\_1} - \gamma DL_{strngr\_1}}{\gamma LL_{strngr\_2}} = 0.7$$

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):



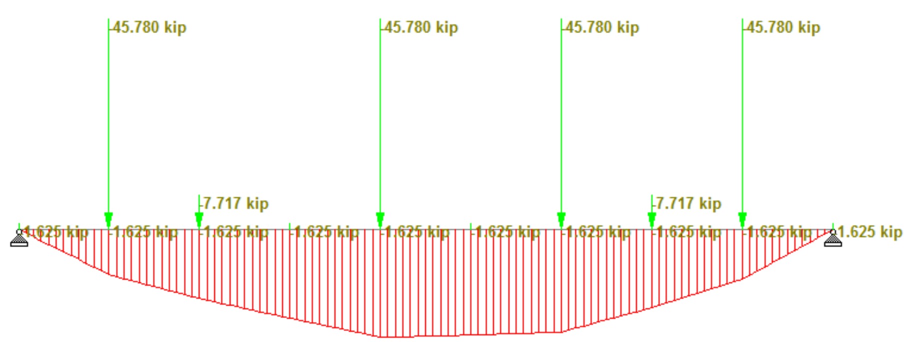
Point loads on floor beams

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

Lane loads on floor beams

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

$$\gamma LL_{FB\_2} := 1.75\cdot 274.9k\cdot ft$$

Inventory Rating Factor

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

Operating Rating Factor

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

East Truss

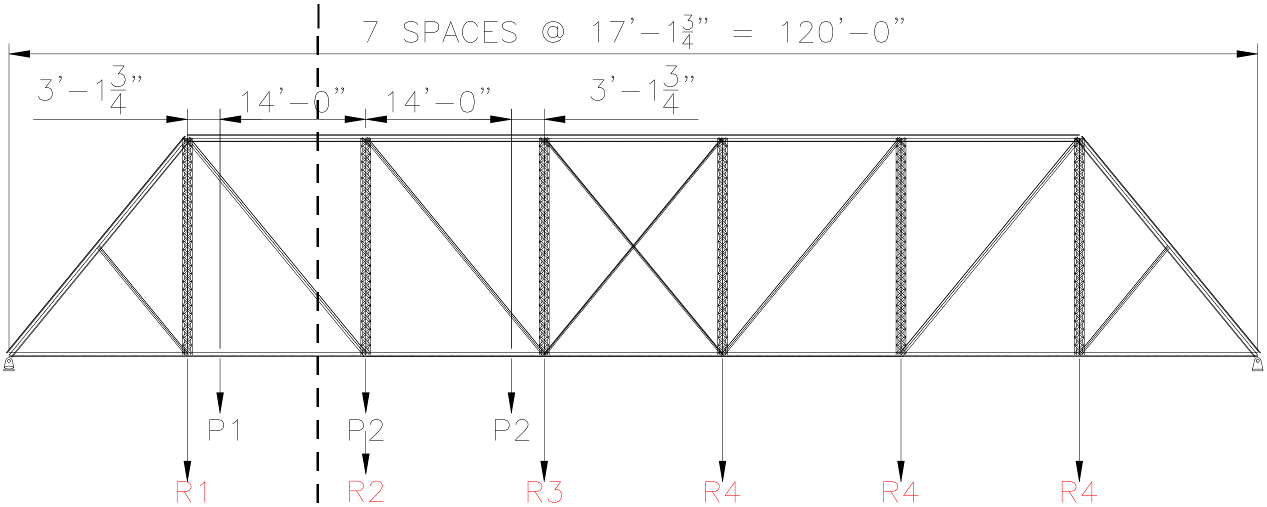
$$R_1 := (1 + IM) \cdot 2P_1 \cdot \left(\frac{L_{panel\_1} - 3ft - 1.75in}{L_{panel\_1}} \right) + (1 + IM) \cdot LL_{lane} \cdot L_{panel\_1} = 13.1 \cdot k$$

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

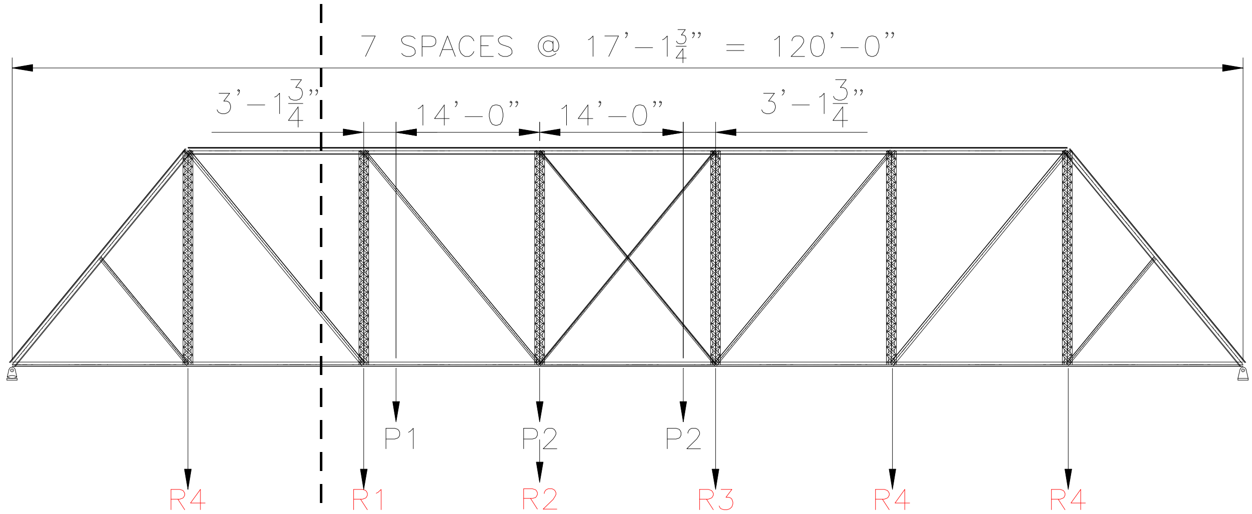
$$R_3 := (1 + IM) \cdot 2P_2 \cdot \left(\frac{L_{panel\_1} - 3ft - 1.75in}{L_{panel\_1}} \right) + (1 + IM) \cdot LL_{lane} \cdot L_{panel\_1} = 39.16 \cdot k$$

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

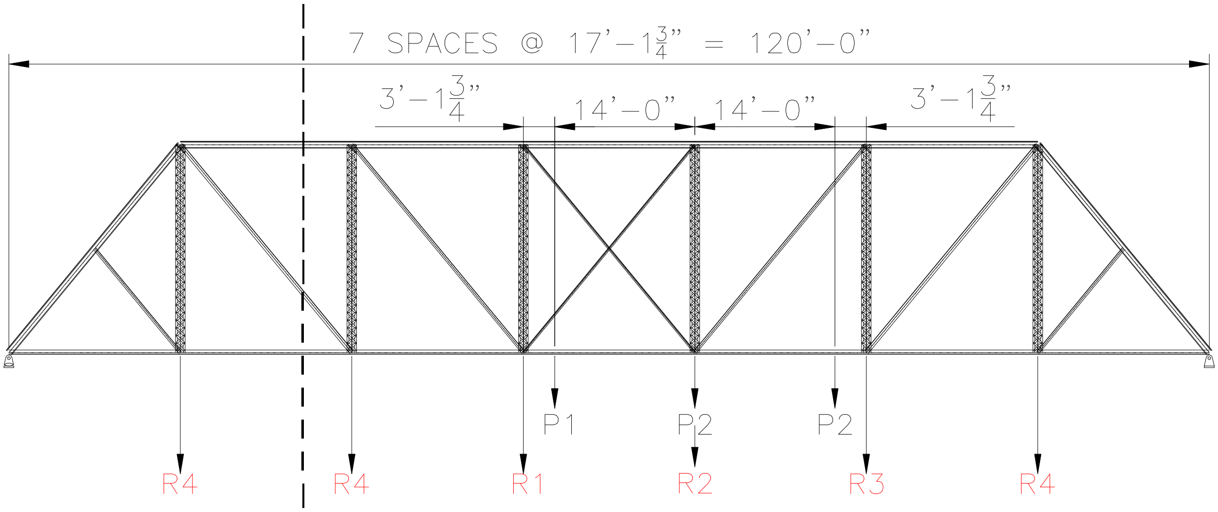
Case 1:



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

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

U2-L3 DL

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

U2-L3 LL

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

Inventory Rating Factor

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

Operating Rating Factor

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

West Truss

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

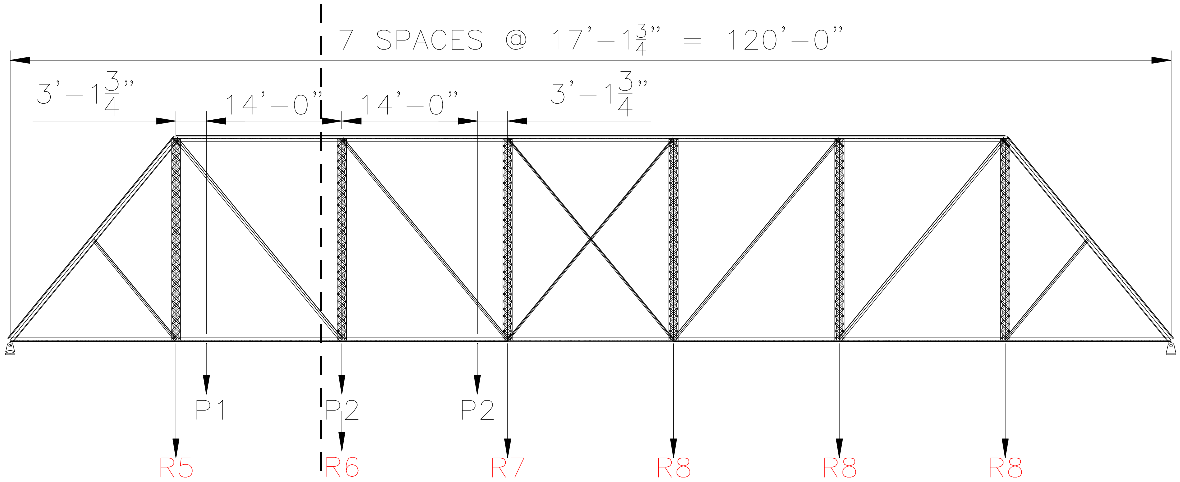
$$\mathbf{R_6} := (1 + \mathbf{IM}) \cdot \mathbf{2P_2} \cdot \left(\frac{3\mathbf{ft} + 1.75\mathbf{in}}{\mathbf{L_{panel\_1}}} + 1 \right) + (1 + \mathbf{IM}) \cdot \mathbf{2P_1} \cdot \left(\frac{3\mathbf{ft} + 1.75\mathbf{in}}{\mathbf{L_{panel\_1}}} \right) + (1 + \mathbf{IM}) \cdot \mathbf{LL_{lane}}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

$$\mathbf{R_6} = 64.45 \cdot \mathbf{k}$$

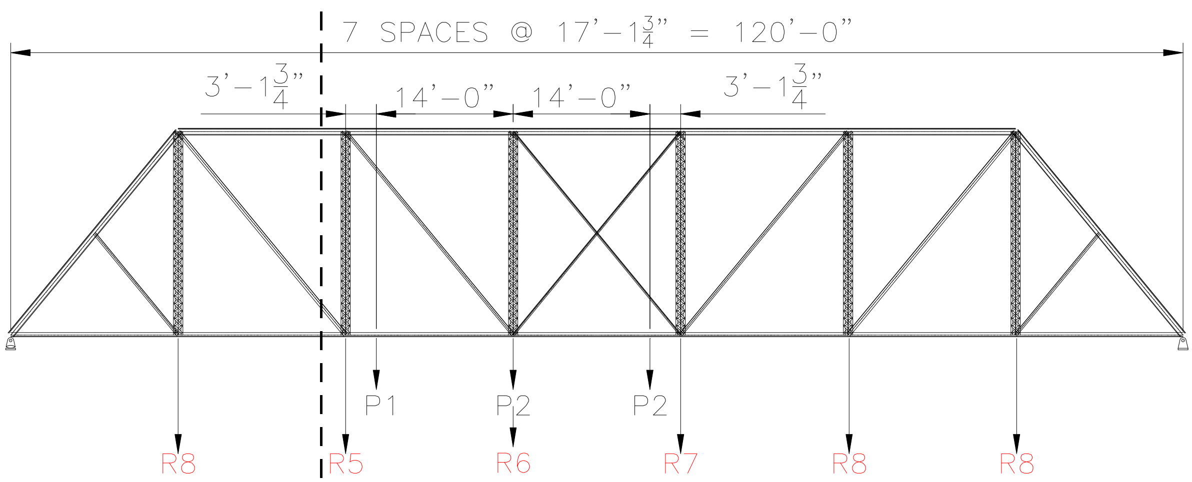
$$\mathbf{R_7} := (1 + \mathbf{IM}) \cdot \mathbf{2P_2} \cdot \left(\frac{\mathbf{L_{panel\_1}} - 3\mathbf{ft} - 1.75\mathbf{in}}{\mathbf{L_{panel\_1}}} \right) + (1 + \mathbf{IM}) \cdot \mathbf{LL_{lane}} \cdot \mathbf{L_{panel\_1}} + \mathbf{w_{ped\_LL}} \cdot \mathbf{L_{panel\_1}} \cdot 5\mathbf{ft}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

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

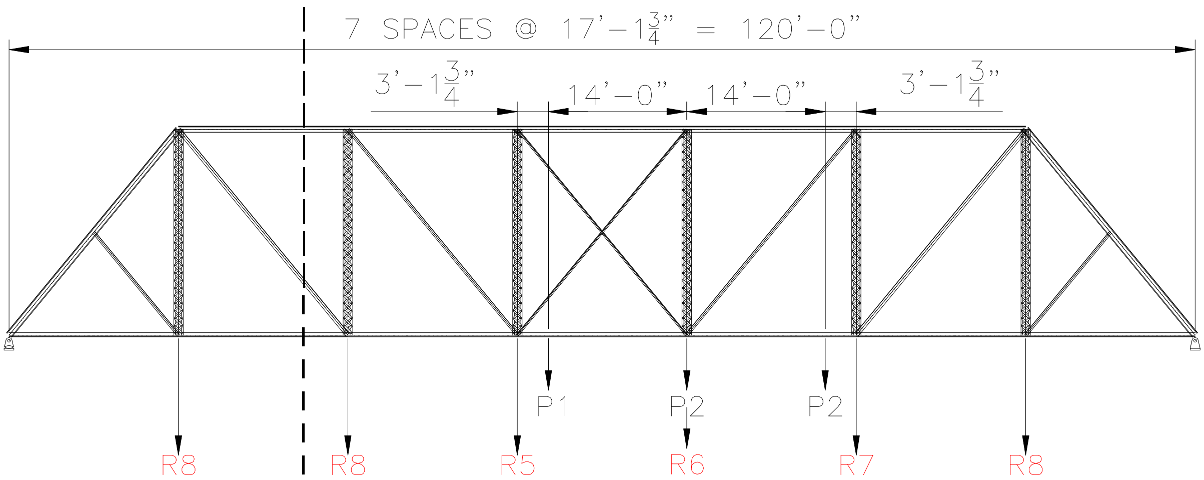
Case 1:



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	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

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

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

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

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

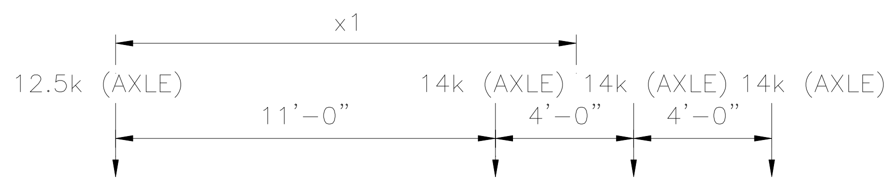
Operating Rating Factor

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

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

Legal Loads

Type 4 Truck

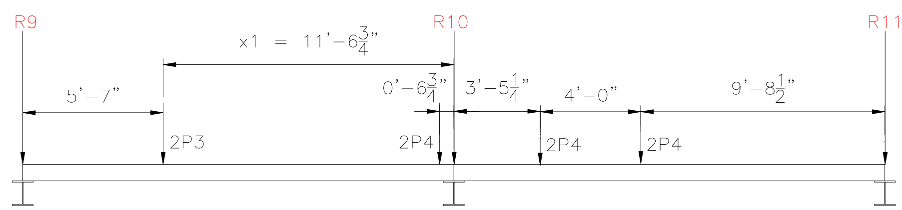


Wheel loads

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

Resultant distance

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



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

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

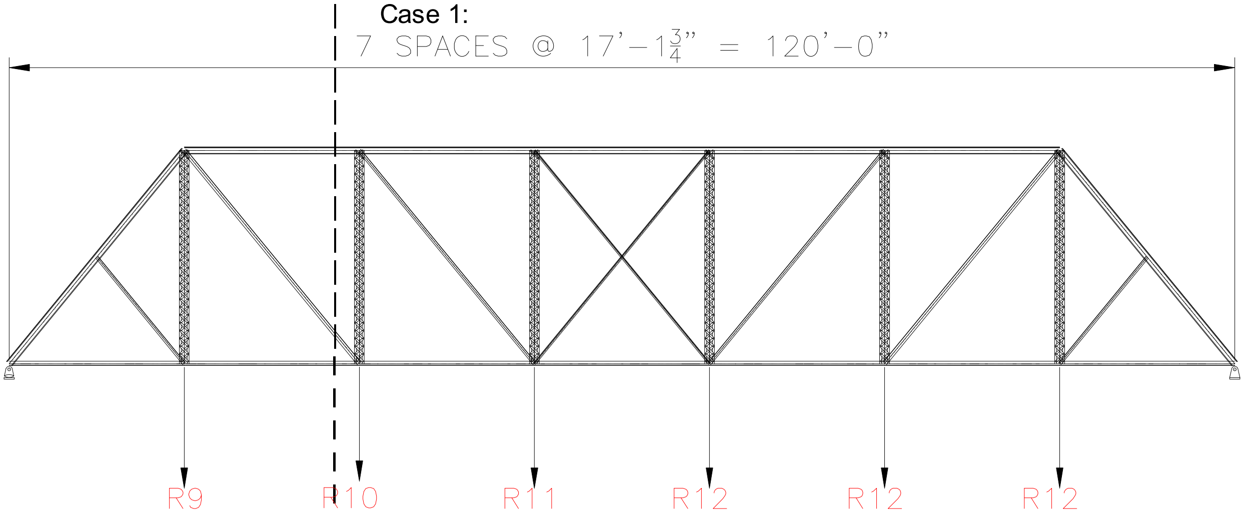
$$\mathbf{R_{10}} = 62.36 \cdot \mathbf{k}$$

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

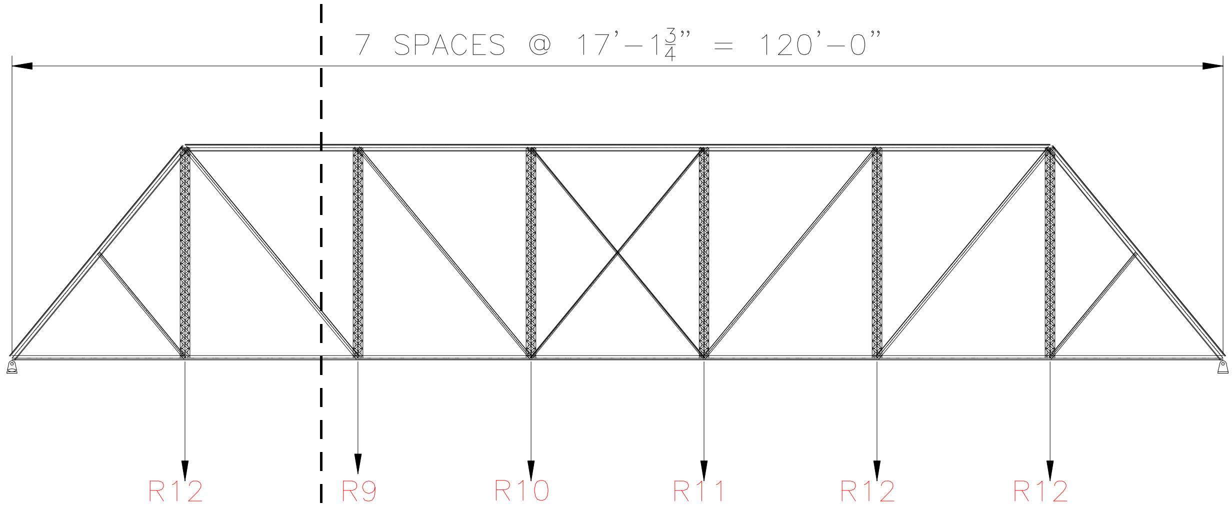
$$\mathbf{R_{12}} := \mathbf{w_{ped\_LL}} \cdot \mathbf{L_{panel\_1}} \cdot 5\mathbf{ft} = 7.72 \cdot \mathbf{k}$$

West Truss

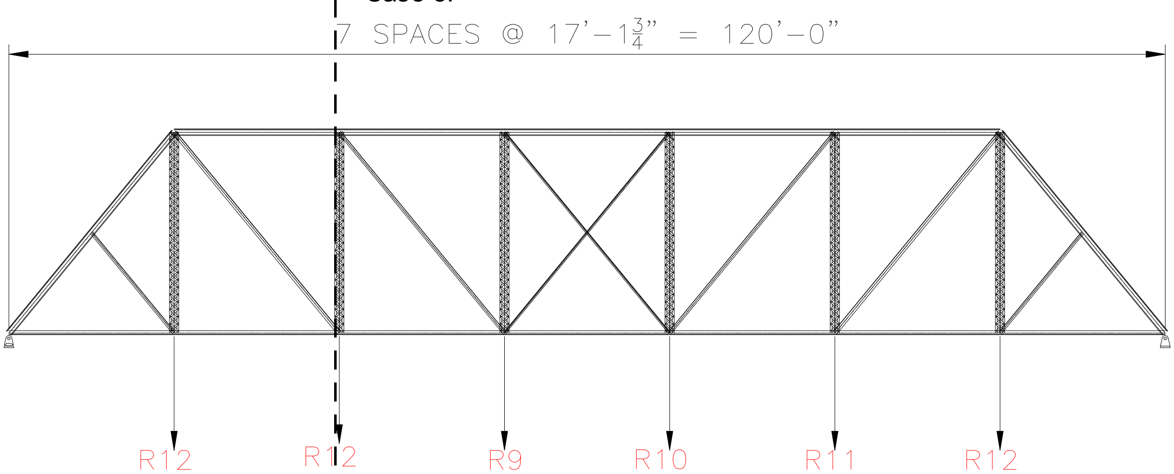
Case 1:



Case 2:



Case 3:



Truss forces analyzed in STAAD per AASHTO LRFR Strength I

			A_g	L	r		Allowable Comp. $\phi_c P_n$	Allowable Tension $\phi_y F_y A_g$	ACTUAL LOAD $?_i P_u$	
			(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

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

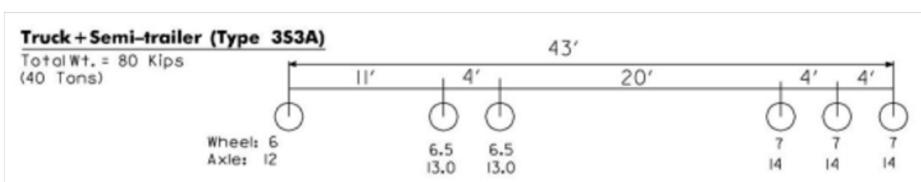
U2-U3 LL

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

Type 4 Truck Rating

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

Type 3S3 Truck

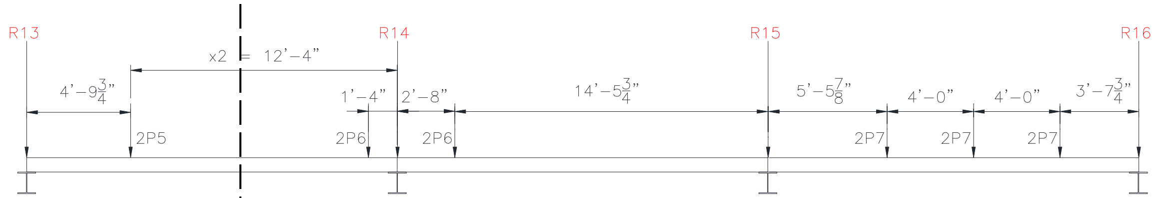


Wheel loads

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

Resultant distance

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



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

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

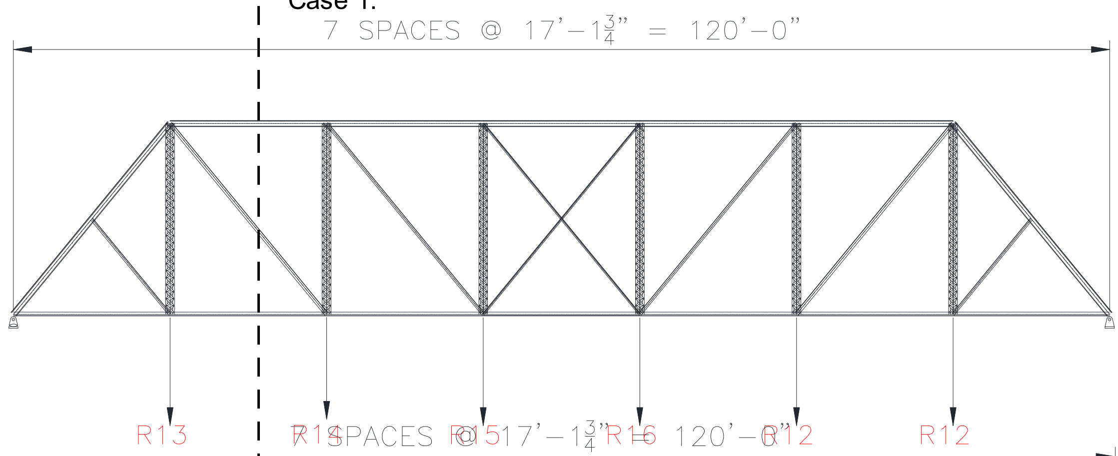
$$R_{15} := (1 + IM)\cdot\left[2P_6\cdot\left(\frac{2ft + 8in}{L_{panel\_1}}\right) + 2P_7\cdot\left[\left(\frac{L_{panel\_1} - 13ft - 5.875in}{L_{panel\_1}}\right) + \left(\frac{L_{panel\_1} - 5ft - 5.875in}{L_{panel\_1}}\right) + \left(\frac{L_{panel\_1} -}{L_p}\right.\right.\right.$$

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

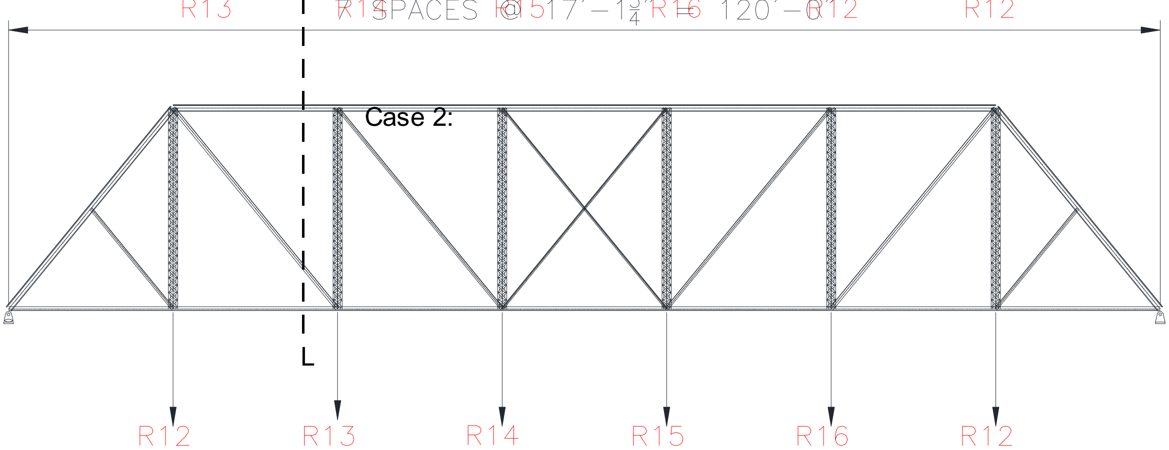
$$R_{16} := (1 + IM)\cdot\left[2P_7\cdot\left[\left(\frac{13ft + 5.875in}{L_{panel\_1}}\right) + \left(\frac{5ft + 5.875in}{L_{panel\_1}}\right) + \left(\frac{9ft + 5.875in}{L_{panel\_1}}\right)\right]\right] + w_{ped\_LL}\cdot L_{panel\_1}\cdot 5ft = 38.63\cdot k$$

<u>West Truss</u>

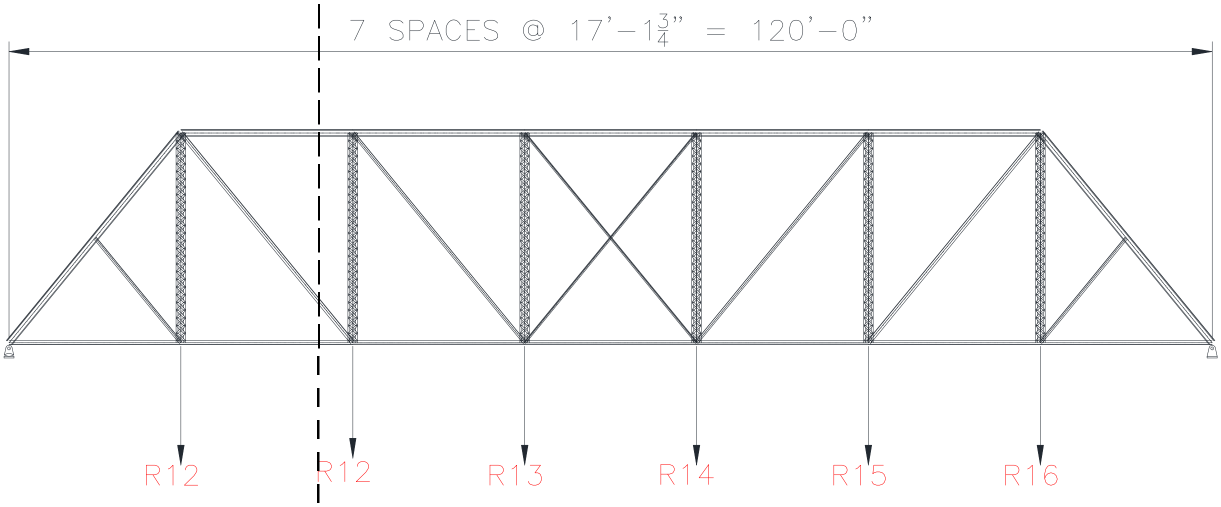
Case 1:



Case 2:



Case 3:



Truss forces analyzed in STAAD per AASHTO LRFR Strength I

							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	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 $C_{truss\_6} := 0.85 \cdot 291.84\mathbf{k}$

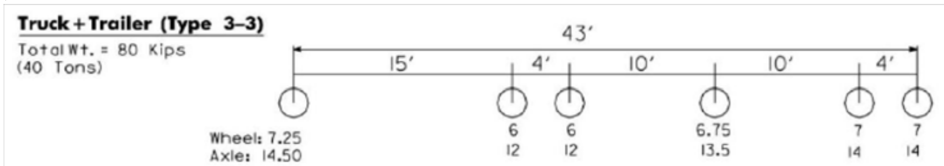
L3-L4 DL $\gamma DL_{truss\_6} := 1.25 \cdot 111.7\mathbf{k}$

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

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

℄

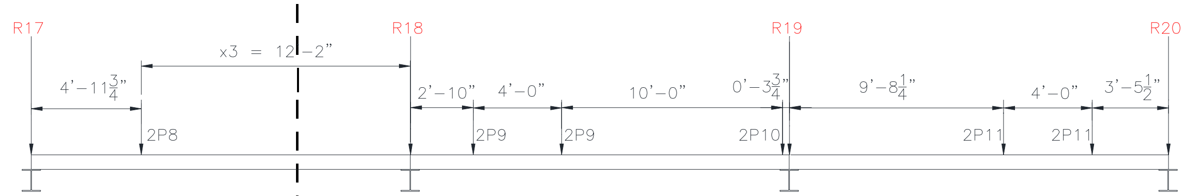
Type 3-3 Truck



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

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



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

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

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

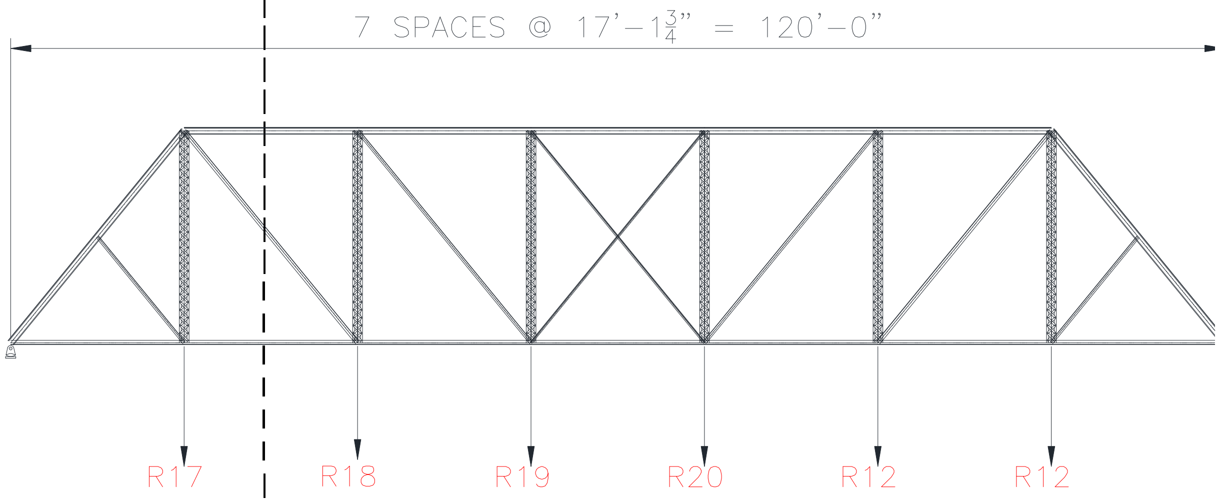
$$\mathbf{R_{19}} := (1 + \mathbf{IM})\cdot\left[\mathbf{2P_{11}}\cdot\left[\left(\frac{\mathbf{L_{panel\_1}} - 9\mathbf{ft} - 8.25\mathbf{in}}{\mathbf{L_{panel\_1}}}\right) + \left(\frac{\mathbf{L_{panel\_1}} - 13\mathbf{ft} - 8.25\mathbf{in}}{\mathbf{L_{panel\_1}}}\right)\right] + \mathbf{2P_9}\cdot\left[\left(\frac{34\mathbf{in}}{\mathbf{L_{panel\_1}}}\right)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

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

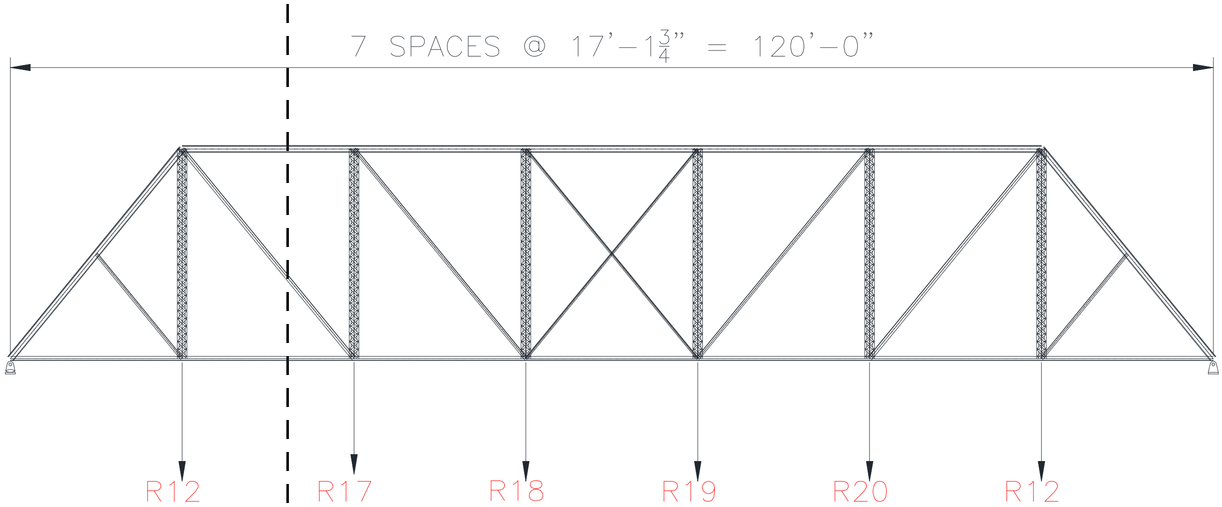
$$\mathbf{R_{20}} := (1 + \mathbf{IM})\cdot\left[\mathbf{2P_{11}}\cdot\left[\left(\frac{9\mathbf{ft} + 8.25\mathbf{in}}{\mathbf{L_{panel\_1}}}\right) + \left(\frac{13\mathbf{ft} + 8.25\mathbf{in}}{\mathbf{L_{panel\_1}}}\right)\right]\right] + \mathbf{w_{ped\_LL}}\cdot\mathbf{L_{panel\_1}}\cdot 5\mathbf{ft}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

West Truss

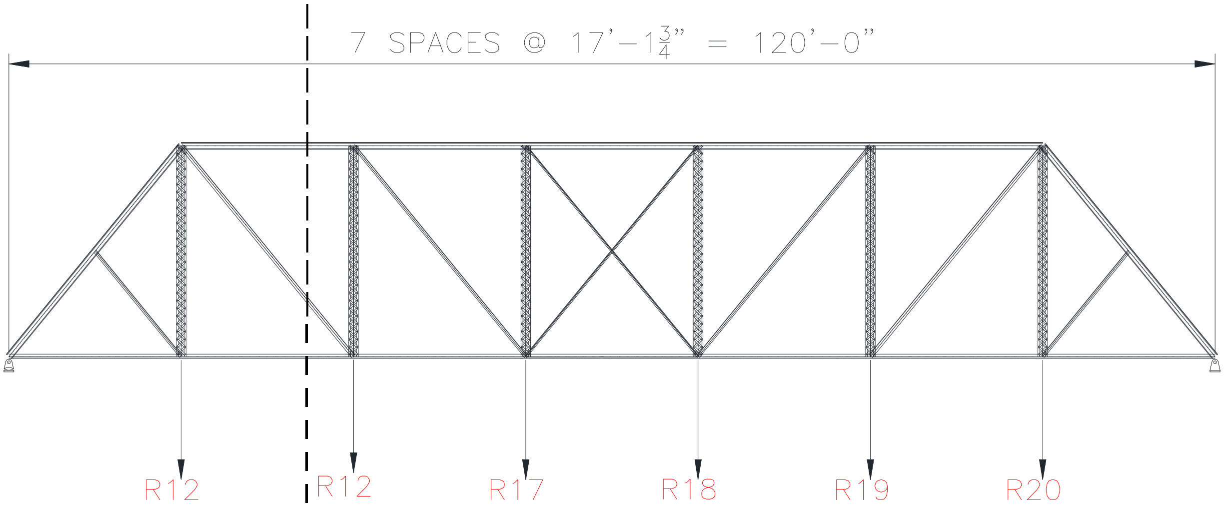
Case 1:



Case 2:



Case 3:



Truss forces analyzed in STAAD per AASHTO LRFR Strength I

			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	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

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

L3-L4 DL

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

L3-L4 LL

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

Type 3-3 Truck Rating

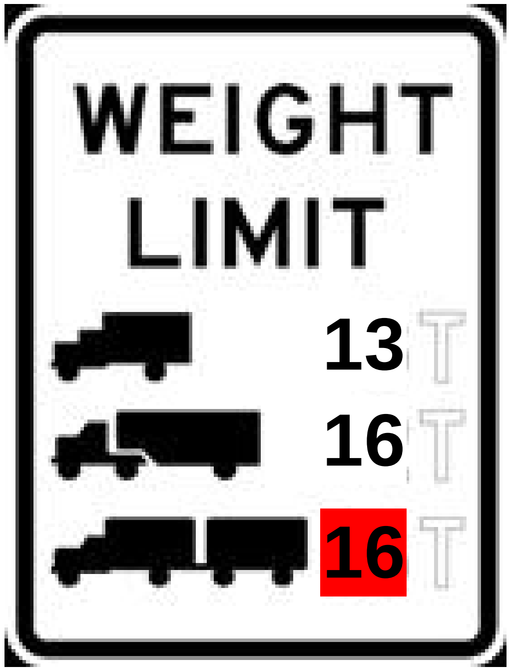
$$\mathbf{Type\_3\_3} := 40\mathbf{ton} \cdot \frac{\mathbf{C_{truss\_7}} - \boldsymbol{\gamma}\mathbf{DL_{truss\_7}}}{\boldsymbol{\gamma}\mathbf{LL_{truss\_7}}} = 16.1 \cdot \mathbf{ton}$$

Legal Load Rating Summary

Type_4 = 13·**ton**

Type_3S3 = 16·**ton**

Type_3_3 = 16·**ton**



L

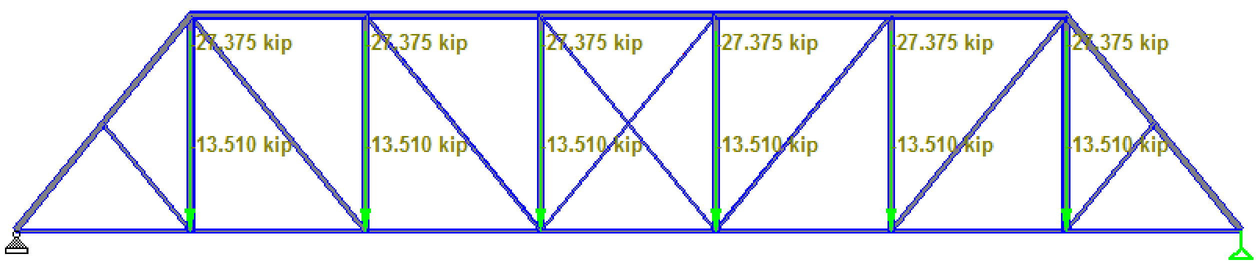
SIDEWALK ONLY

Pedestrian Live Load

LL Panel Point Load (west) $\mathbf{LL_{ped\_3}} := 5\mathbf{ft}\cdot\mathbf{L_{panel\_1}}\cdot\mathbf{w_{ped\_LL}} = 7.72\cdot\mathbf{k}$

West Truss

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



							Allowable	Allowable	ACTUAL	
							Comp.	Tension	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	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 $\mathbf{C_{truss\_8}} := \mathbf{min}(0.85, \varphi_{c\_truss\_1}\cdot\varphi_{s\_truss\_1})\cdot 291.8\mathbf{k}$

L3-L4 DL $\gamma\mathbf{DL_{truss\_8}} := 1.25\cdot 111.7\mathbf{k}$

L3-L4 LL $\gamma\mathbf{LL_{truss\_8}} := 1.75\cdot 38.1\mathbf{k}$

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

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