PORT OF HOOD RIVER COMMISSION
AGENDA
Sunday, June 30, 2024
Port Conference Room
1000 E. Port Marina Drive, Hood River

No packet materials are available at this time. Materials may be provided on the day of the meeting. The meeting will be live on YouTube. Here is the link: https://portofhoodriver.com/live-stream/

1. **Call to Order, Commissioner Kristi Chapman** – 10:00 AM

2. **Staff Report/Introductions, Kevin Greenwood, Executive Director, Kevin Greenwood**

3. **Bridge Analysis and Recommendations, Justin Doornink, HDR**

4. **Confirmation of HDR Process and Bridge Repair Approach, John Brestin, Kiewit**

5. **Possible Bridge Actions:**
   a. Motion related to bridge opening
   b. Motion related to permanent fix
   c. Motion delegating contract approval authorization

6. **Adjourn**

If you have a disability that requires any special materials, services, or assistance, please contact us at 541,386,1645 so we may arrange for appropriate accommodations.

The chair reserves the opportunity to change the order of the items if unforeseen circumstances arise. The Commission welcomes public comment on issues not on the agenda during the public comment period. With the exception of factual questions, the Commission does not immediately discuss issues raised during public comment. The Commission will either refer concerns raised during public comment to the Executive Director for a response or will request that the issue be placed on a future meeting agenda. People distributing copies of materials as part of their testimony should bring ***10 copies****. Written comment on issues of concern may be submitted to the Port Office at any time.*

Memo

Date:	Sunday, June 30, 2024
Project:	Hood River - White Salmon Interstate Bridge (ODOT Bridge Number 06645)
To:	Kevin Greenwood (Port)
From:	Justin Doornink, PhD, PE (HDR)
Subject:	**2024 Portal Damage Assessment**

Executive Summary

On June 27, 2024, the Port of Hood River (Port) contacted HDR Engineering, Inc. (HDR) regarding bridge members damaged by a vehicle impact on the existing Hood River - White Salmon Interstate Bridge. Based on the initial briefing and photographs provided by the Port, HDR recommended that the Port close the bridge until further notice until additional field investigations could take place to further assess the condition of the bridge.

Working with Port staff on site, the HDR engineering and inspection team observed the damage at six (6) overhead brace locations. HDR then performed an assessment of the bridge response in a damaged condition, which was informed by review of existing plans, the existing calculations, and damage observed on site. The assessment did not identify a structural deficiency that would preclude vehicular traffic from using the bridge in the lowered, damaged condition.

Work performed by HDR has been accomplished utilizing reasonable efforts, assumptions and standard of care commensurate with the limited timeframe, access and information available for this effort. At the discretion of the Port, reopening of the bridge to vehicular traffic may proceed provided that vehicles are compliant with the posted load rating of the bridge.

HDR

Memo

Date:	Sunday, June 30, 2024
Project:	Hood River - White Salmon Interstate Bridge (ODOT Bridge Number 06645)
To:	Kevin Greenwood (Port)
From:	Justin Doornink, PhD, PE (HDR)
Subject:	**2024 Portal Damage Assessment**

1.0 Incident Background and Existing Conditions Summary

At approximately 11:00am on June 27, 2024, the Port of Hood River (Port) contacted Mikal Mitchell and Justin Doornink with HDR Engineering, Inc. (HDR) regarding damaged members on the Hood River - White Salmon Interstate Bridge (ODOT Bridge Number 06645). During this initial briefing, it was communicated to HDR that several overhead bracing members were damaged after being struck by an oversized vehicle, and photographs of the damaged members were provided to HDR by the Port.

Based on the initial briefing and provided photographs, at approximately 11:30am Justin Doornink recommended that the Port close the bridge until further notice so that additional field investigations could take place to further assess the condition of the bridge. The Port agreed and subsequently closed the bridge to vehicular traffic around 11:45am. The bridge currently remains closed to vehicular traffic.

2.0 Field Observations Summary

As directed by the Port, HDR mobilized Justin Doornink (Project Manager), Eric Rau (Senior Bridge Engineer), and Mark Schneider (Bridge Inspector) to the site on the afternoon of June 27, 2024. They arrived to the site at approximately 2:30pm. Upon arrival, the HDR Team observed the damage with Ryan Klapprich (Facilities Manager) of the Port. From the bridge deck, damage could be seen at six (6) portal brace locations:

- South Support Tower (Pier 11), Back Face
- South Support Tower (Pier 11), Front Face
- South End Portal of Truss Lift Span 11
- North Support Tower (Pier 12), Back Face
- North Support Tower (Pier 12), Front Face
- North End Portal of Truss Lift Span 11

After this initial site reconnaissance, Mark Schneider utilized a 40-ft articulated boom lift provided by the Port to inspect the damage more closely. The locations and details of the impacted portal bracings and photographs taken by Mark Schneider are included in Appendix A. After an initial inspection of the portal bracings, Mark Schneider observed the bearing seats of the lift span and determined, with assistance and input from Ryan Klapprich, that the impact event to the bridge did not alter the pre-existing bearing seat conditions provided by the Port.

3.0 Structural Assessment and Findings

Once initial field reconnaissance efforts concluded, an expedited high-level assessment of the bridge in its observed damaged condition was initiated. The HDR team was in continuous communication with the Port during the assessment. The assessment considered both vertical and lateral response of the bridge, which was informed by review of existing plans, the current 2020 load rating, and damage observed on site. The assessment generally compared loads, forces, and capacities of the bridge in the damaged condition to baseline values documented in the existing information provided by the Port. The bridge component would be deemed sufficient if the comparison resulted in similar values as the baseline. Refined structural analysis or detailed determination of member capacities was not performed as part of this expedited work. Outside of the damage associated with the events of June 27, 2024, the in-service condition of the 1937 structure was not considered, and due to the emergency nature of the services, a full and detailed investigation was not performed.

Given the urgency of this work and consistent with the observed damage of the bridge, conservative assumptions were used to simplify the work. These included:

- Damage is consistent at the three (3) north and (3) south locations.
- Damage is isolated to the portal frame members and interior connections (gusset plate, bolts, and rivets) and damage did not propagate to the primary members of the tower supports (front and back legs) and lift span truss chords.
- The vertical load assessment was specific to dead load and the vehicular live load.
- The horizontal load assessment was specific to wind on structure loads.
- The lift span truss is adequately seated in a lowered position thereby limiting the assessment of the tower support (back and front legs) to dead and wind on structure loads.
- The lift span truss will remain in a lowered position until corrective work is performed.

With the lift span truss in a lowered position, the structural response of the lift truss span and tower support (back and front legs) act independently of each other. Therefore, a different assessment was performed to review the adequacy of the tower support (back and front leg) and lift truss span in the damaged condition. The assessment of each of these components was also reviewed independently for vertical and horizontal response.

The bridge is a truss structure that spans north to south across the Columbia River. The truss is composed of east and west chords with transverse bracing between. This truss description is applicable to both the lift span truss (bridge span 11) and the lift tower supports (bridge piers 11 and 12) composed of both front and back legs. The portal braces are oriented perpendicular to the long axis (north-south) of the bridge. They are secondary members that provide transverse support to the east and west primary members.

Lift Support Tower:

Vertical Load Assessment:

- Each leg of the tower support is a vertically oriented truss that is divided into several braced bays. The top of the tower support is approximately 100-feet above the bridge deck.
- The axial capacity of the vertical members is based on the length between brace points and the section properties within that brace length.
- The axial capacity of the vertical members varies over its height with variable brace length and section properties.
- The assessment reviewed the axial capacity of the tower support with an emphasis on considering the damaged portal brace being fully removed from the lowest brace bay.
- While removal of the portal brace reduced the axial capacity of the tower within the lowest braced bay, the adjacent braced bay (which is not altered by removal of the portal brace) has a lower axial capacity.
- Therefore, it was determined that the Lift Tower Supports are adequate for vertical loads in its current damaged condition.

Horizontal Load Assessment:

- A simplified 2-D truss model of the back tower leg was developed using wind loads defined in the original 1937 plans.
- The findings of the back tower leg are applicable to the front tower leg as the portal brace configuration is similar.
- The analysis model demonstrated that the force demands for tower components are insignificantly different when the damaged components of the portal are removed from the analysis.
- Therefore, it was determined that the Lift Tower Supports are adequate for horizontal loads in its current damaged condition.
- The original design basis considered wind acting on the structure with the lift truss span in both a raised and lowered position. Drawing number 44714 of the original 1937 plan set

provides a summary of the force demands from wind loads acting on the tower supports. The plans document that the force demands are approximately 50% lower for front tower leg with the lift truss span in a lowered position. It is therefore recommended that the lift span truss remain in a lowered position until corrective work is performed.

Lift Truss Span:

Vertical Load Assessment:

- Based on a review of the current 2020 load rating, it appears that the axial capacity of the compression chord is not impacted by the portal brace.
- The unbraced length of this chord element is based on bracing exclusively at panel points and conservatively ignores support provided at the portal brace location.
- As the load rating does not consider the portal brace, its damaged or undamaged condition does not alter the results of the load rating.
- Therefore, it appears that the Lift Truss Span is adequate for vertical loads in its current damaged condition.

Horizontal Load Assessment:

- The assessment considered the capacity of the portal frame in both the damaged and undamaged conditions.
- A simplified 2-D truss model of the portal was developed with force demands based on winds acting on the structure and an assumed minimum force required to brace the compression chord of the lift span truss. Consideration of the minimum bracing force was conservative as the load rating evaluation of the compression chord did not consider bracing provided at the portal location.
- The analysis model was used to if determine force demands for portal components are functional after the damaged component was removed.
- While force demands were shown to increase, they remained below the capacity of the members.
- Therefore, it appears that the Lift Truss Span is adequate for horizontal loads in its current damaged condition.

4.0 Stipulations and Recommendations

Given the emergency nature of the services requested of HDR by the Port, and in conjunction with limited access to the site, data, information and third parties that HDR would have access to when performing these types of services under normal circumstances, HDR has necessarily relied upon, in whole or in part, the data and information provided by the Port. As such, the information provided has not been independently verified by HDR and is assumed to be accurate, complete, reliable, and current.

Work performed by HDR has been accomplished utilizing reasonable efforts, assumptions and standard of care commensurate with the limited timeframe, access and information available for this effort. As such, HDR does not warrant nor guarantee the conclusions set forth below.

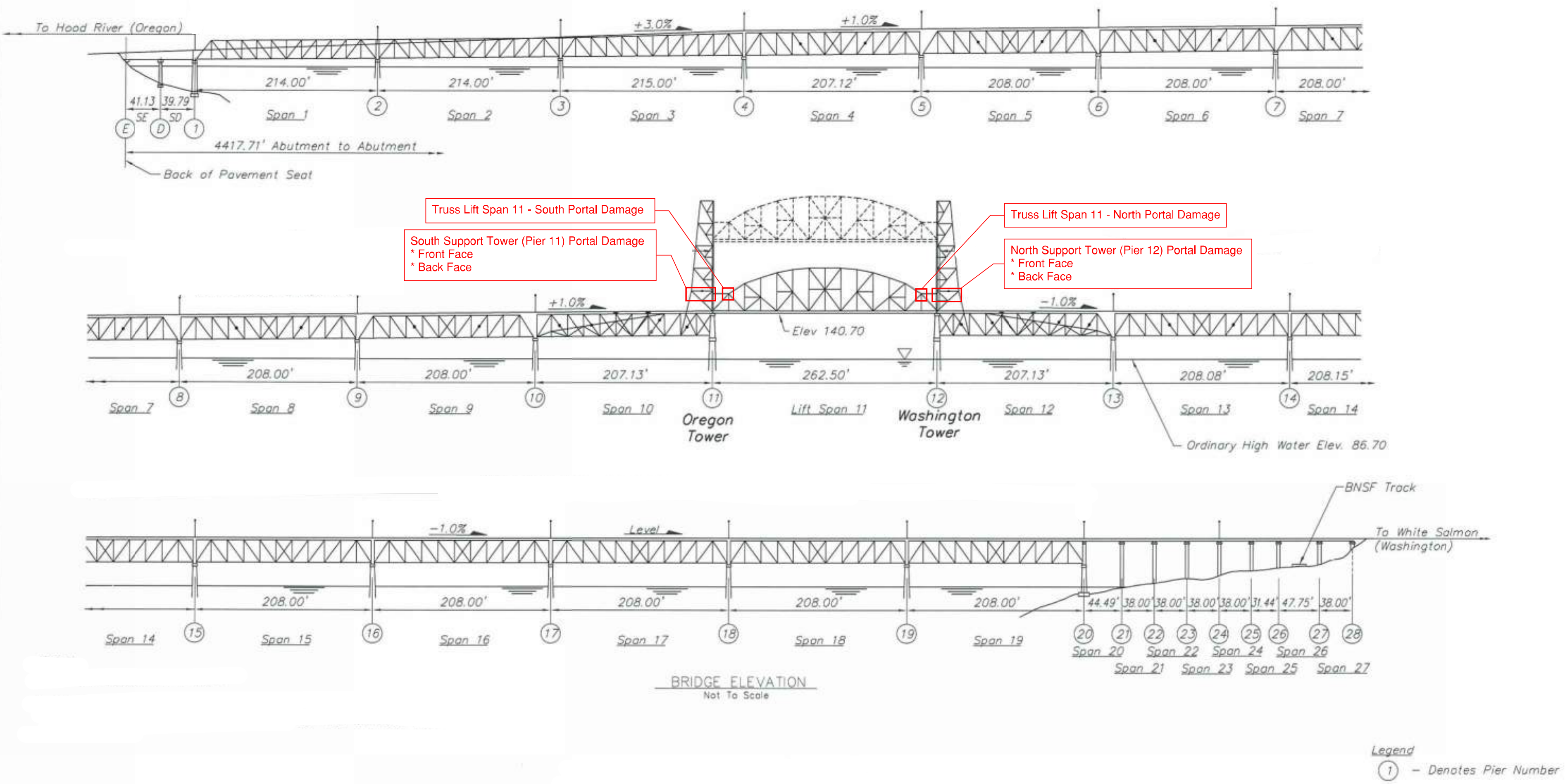
With that said, HDR's initial analysis of the bridge in its current damaged condition supports the following conclusions:

- The existing bridge lift span should remain in place and not be raised until all damaged portal bracing is repaired.
- At the discretion of the Port, resumption of marine traffic under the bridge may proceed provided the Port can determine that there is sufficient vertical clearance to do so without raising the lift span.
- At the discretion of the Port, reopening of the bridge to vehicular traffic may proceed provided that vehicles are compliant with the posted load rating of the bridge.
- Design of final bridge repair plans, specifications, and estimate should be expedited with the resulting repairs to follow immediately thereafter.

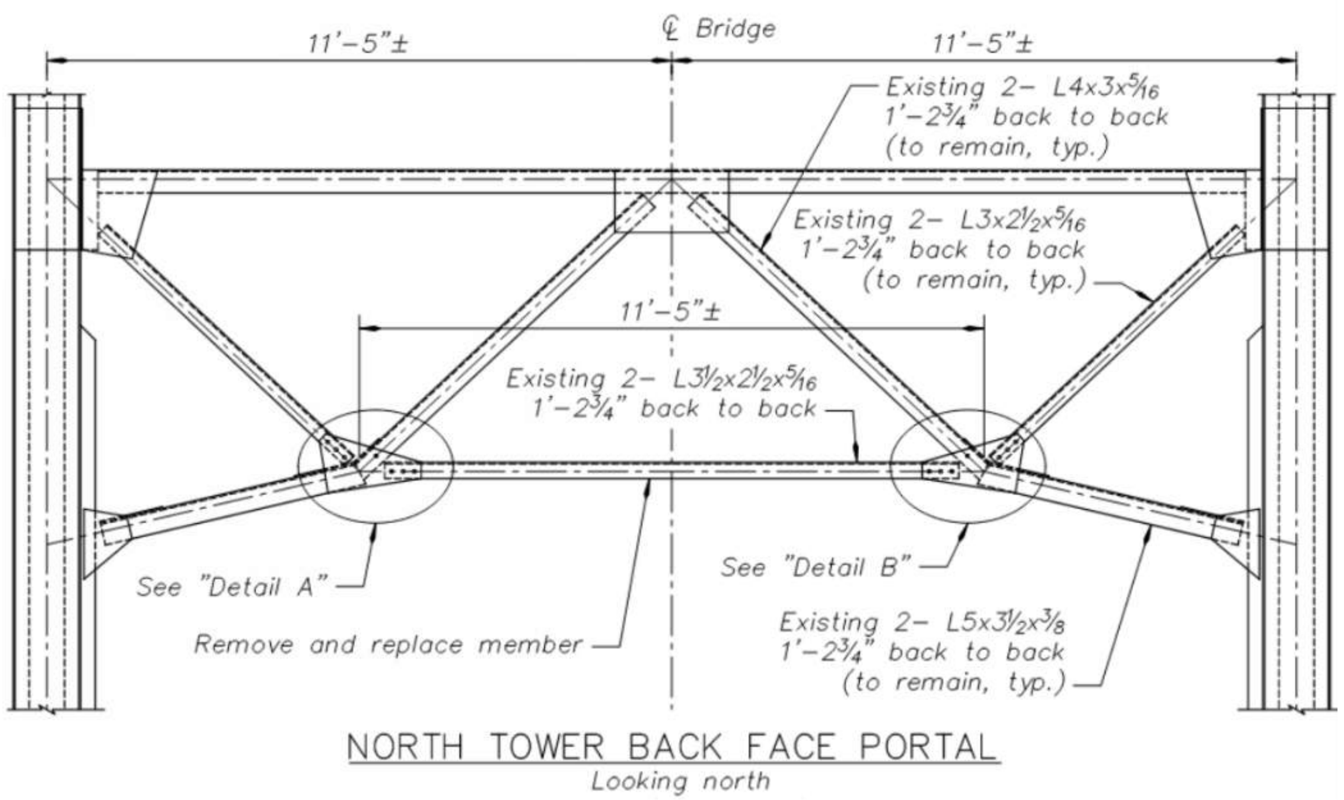
Appendix A

Portal Details and Photographs

Observed Damage Locations

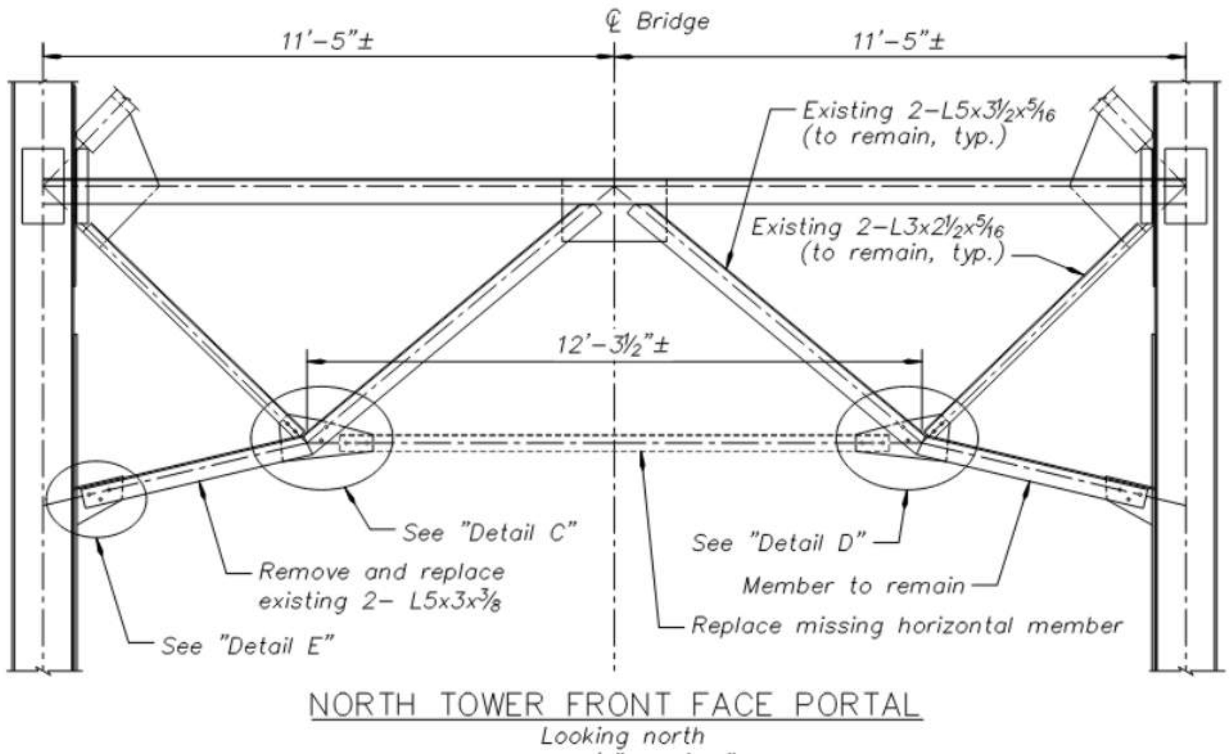


North Support Tower (Pier 12): Back Face Portal Details
Source: Dwg. 100741



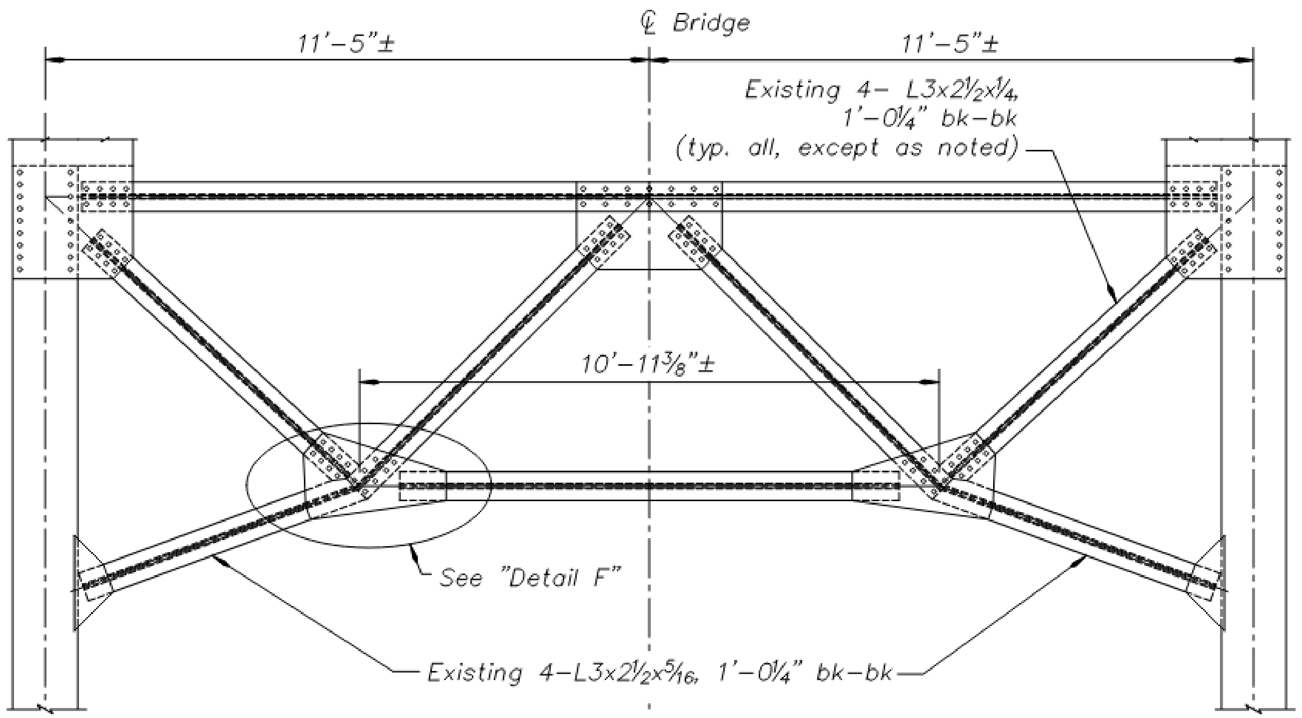
Select Pictures of Observed Damage

North Support Tower (Pier 12): Front Face Portal Details
Source: Dwg. 100742



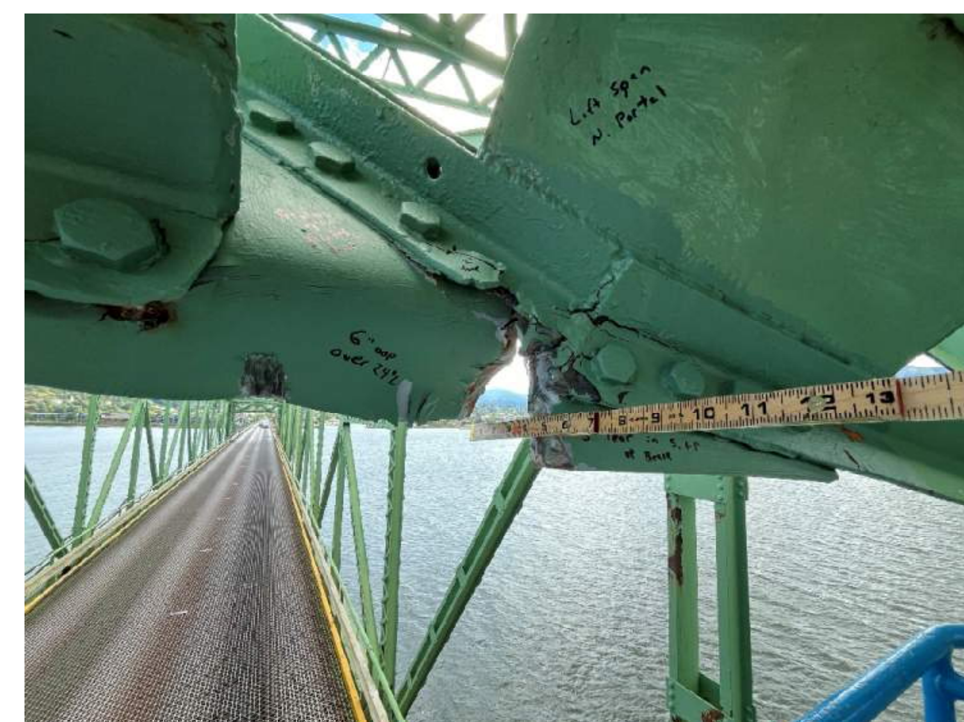
Select Pictures of Observed Damage

Truss Lift Span 11: North Portal Details
Source: Dwg. 102770

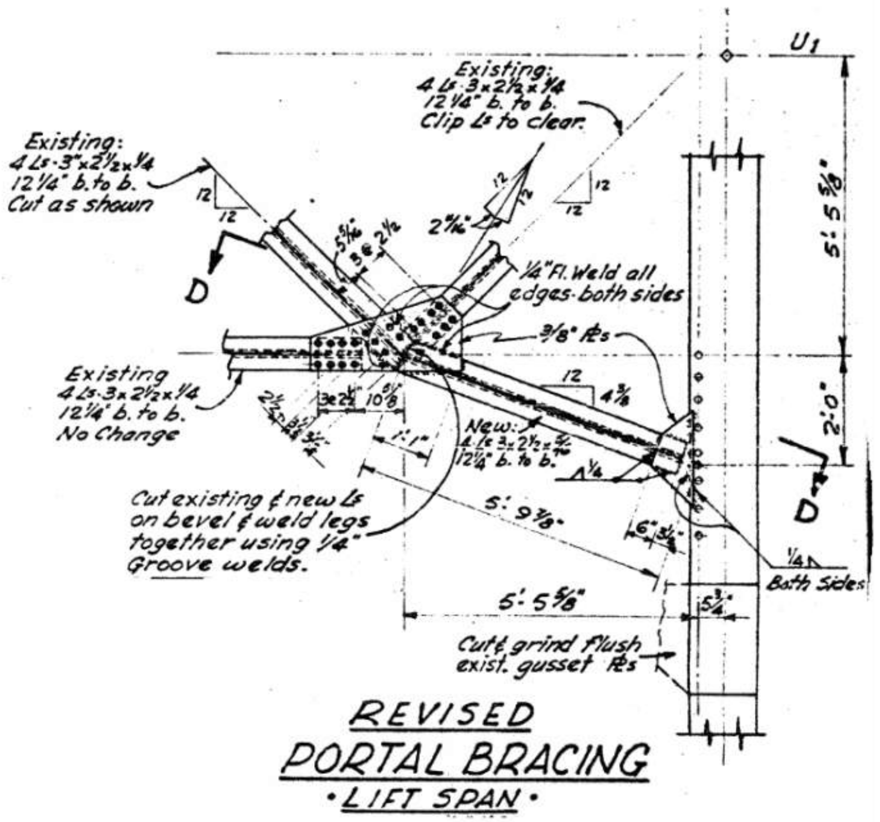


LIFT SPAN PORTAL

Select Pictures of Observed Damage

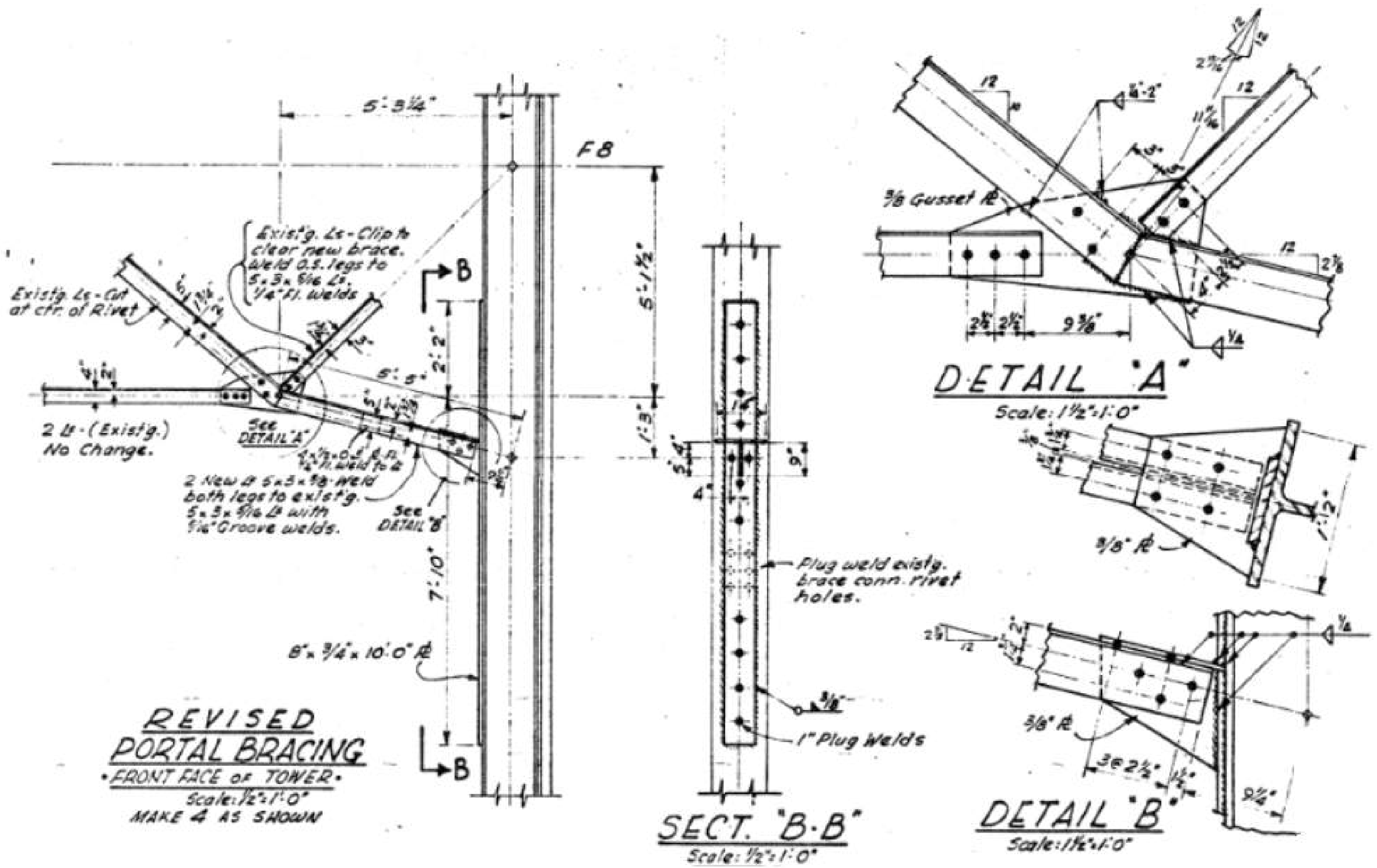


Truss Lift Span 11: South Portal Details
Source: Drawing File 362A



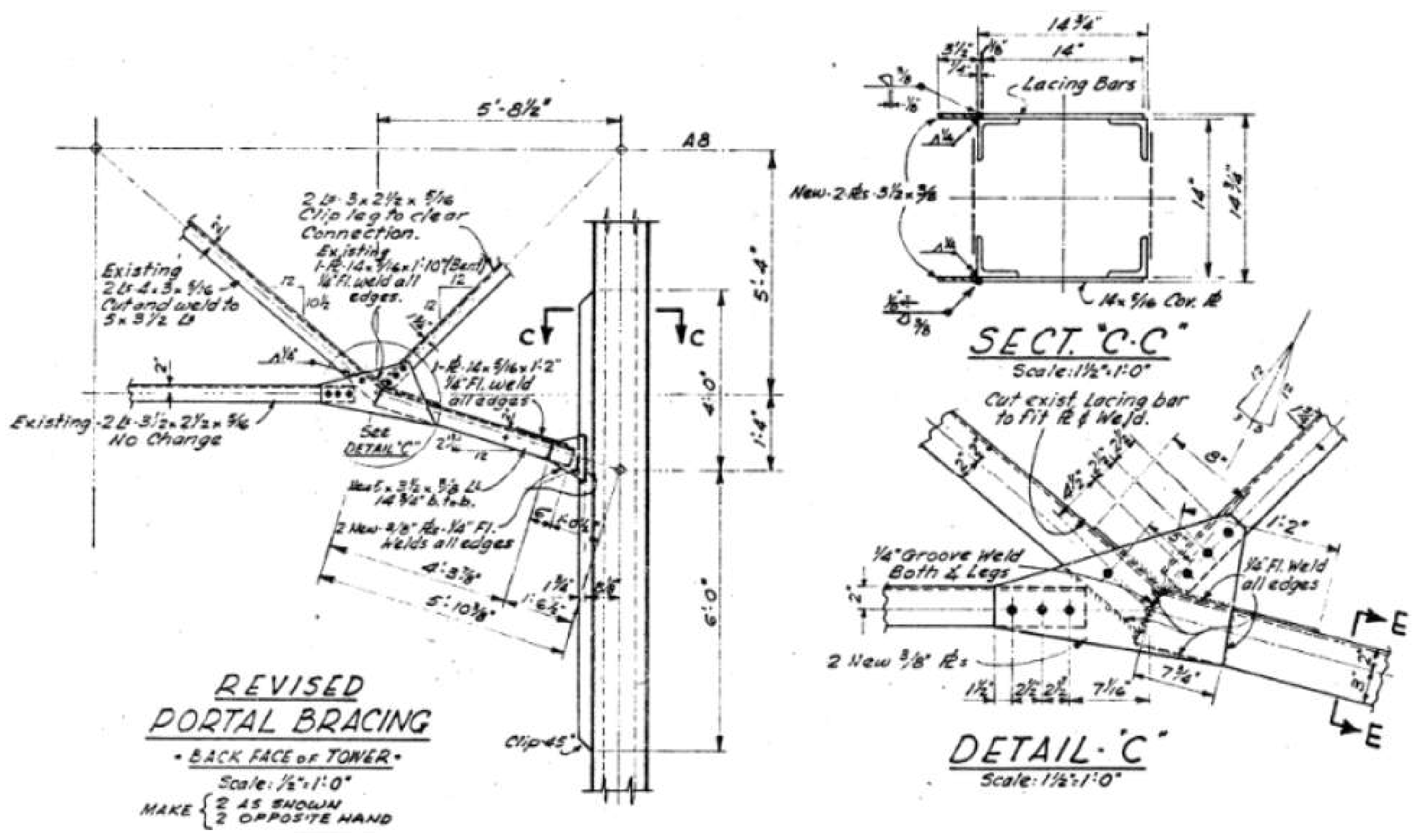
Select Pictures of Observed Damage

South Support Tower (Pier 11): Front Face Portal Details
Source: Drawing File 362A

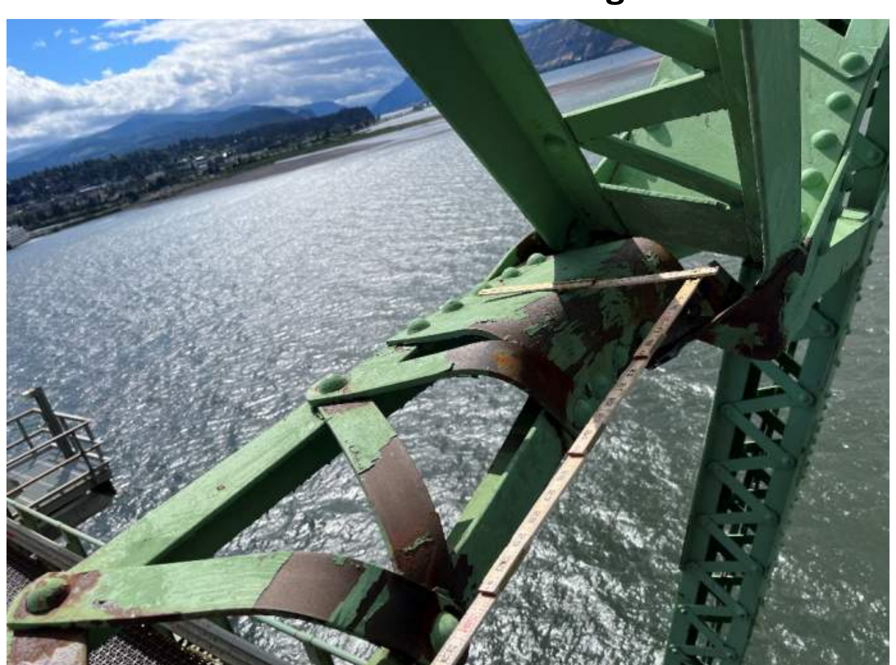


Pictures of Observed Damage

South Support Tower (Pier 11): Back Face Portal Details
Source: Drawing File 362A



Select Pictures of Observed Damage

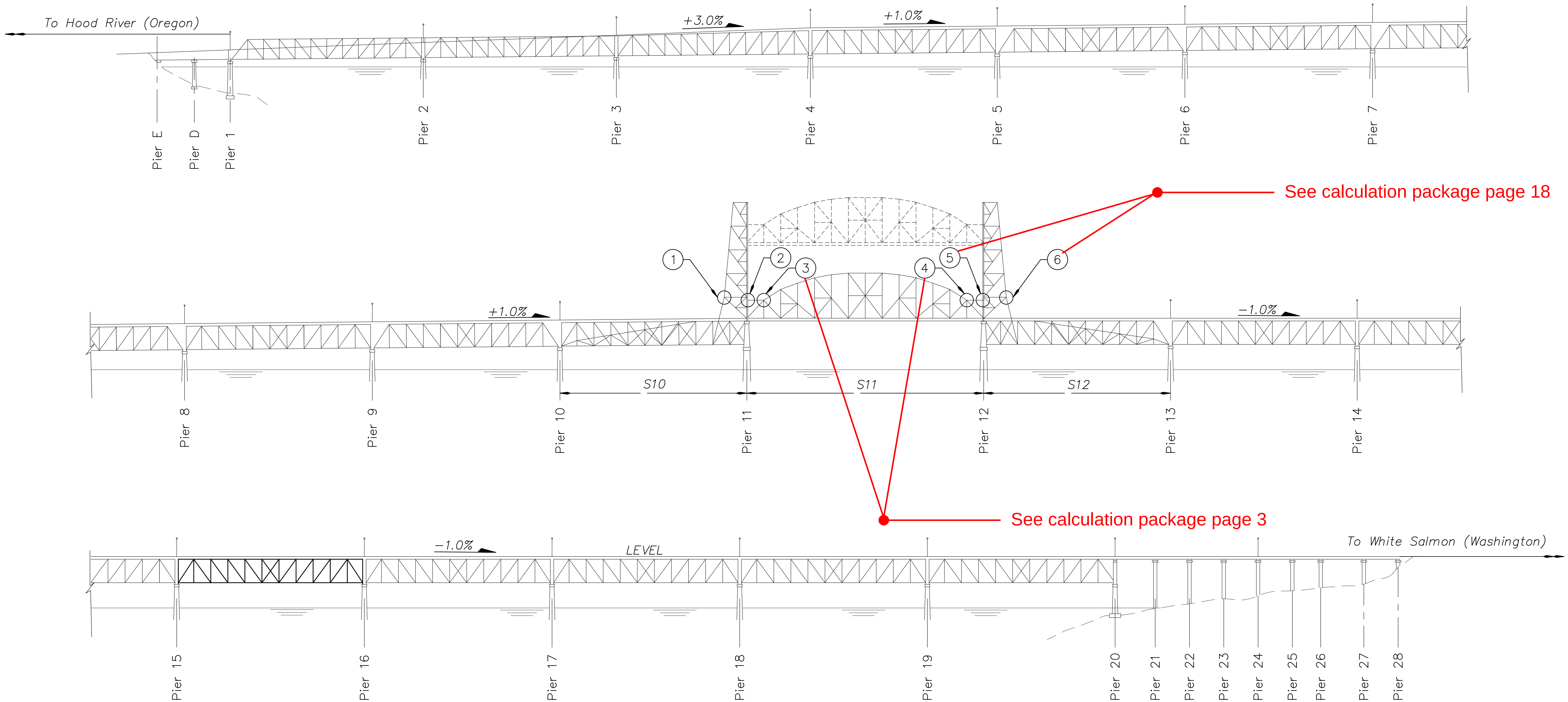


Appendix B

Calculations Package

Appendix B Calculation Package:

Please note that the conclusions stated in this Appendix are subject to the qualifications and assumptions set forth in the Assessment Memo dated June 30, 2024. Specifically, HDR performed these calculations based on a preliminary investigation with limited time and access, and reliant upon information provided by the client.



BRIDGE ELEVATION

Scale: NTS

(East side of bridge shown, looking west)
(West side similar)

Notes:

1. All dimensions are in feet.
2. The dimensions shown on the plans are based on drawings of the existing structure provided by the Port Of Hood River Commission. Actual dimensions and conditions shall be field verified by the Contractor prior to fabrication.

Legend

S1 – Denotes Span Number

NTS – Denotes "Not to Scale"

1 South Support Tower (Pier 11), Back Face

2 South Support Tower (Pier 11), Front Face

3 South End Portal of Truss Lift Span 11

4 North End Portal of Truss Lift Span 11

5 North Support Tower (Pier 12), Front Face

6 North Support Tower (Pier 12), Back Face

DATE	REVISION	BY

ACCOMPANIED BY DWGS.

DRAFTER:

DESIGNER:

CHECKER:

REVIEWER:

PORT OF HOOD RIVER COMMISSION

HDR

STRUCTURE NO. 06645

DATE June – 2024

CALC. BOOK

HOOD RIVER INTERSTATE BRIDGE
MISCELLANEOUS TRUSS AND STEEL REPAIRS
HOOD RIVER – WHITE SALMON HIGHWAY
HOOD RIVER, OR & KLICKITAT, WA COUNTIES

GENERAL ELEVATION

SHEET OF

DRAWING NO.

Xrefs: POHR_T01_Bridge_Border.dwg X_POHR_Bridge.dwg

Overview

1) A 2D LARSA truss model was created for the lift span portal frame. The geometry of the model was based on 2019 plans. The portal frame was loaded with STR-III wind load on structure (WS) load and bracing loads. The wind load was calculated for the effective tributary area of the truss. The bracing loads was conservatively considered as 5% of the total axial capacity of the main truss member. Though the load rating is not based on the portal frame acting as a brace, for purposes of this assessment it was assumed to be a brace point with ab assumed load of 5%.

2) The analysis model was used to if determine force demands for portal components are functional after the damaged component was removed.

3) While force demands were shown to increase on portal members in the damaged configuration, they remained well below the capacity of the members.

References:

1) ODOT BDM
2) AASHTO LRFD BDS
3) AASHTO Guide Spec for Wind Loads during Construction
4) 1923 Bridge Plans (9 Pages)
5) 1967 Hood River Bridge Portal Bracing "Revisions" (1 Page)
6) 2019 Plans (DWG No: 100739-100742)

PHOTOS

Figure: Damaged portal frame in the lift span (lower third of the picture)

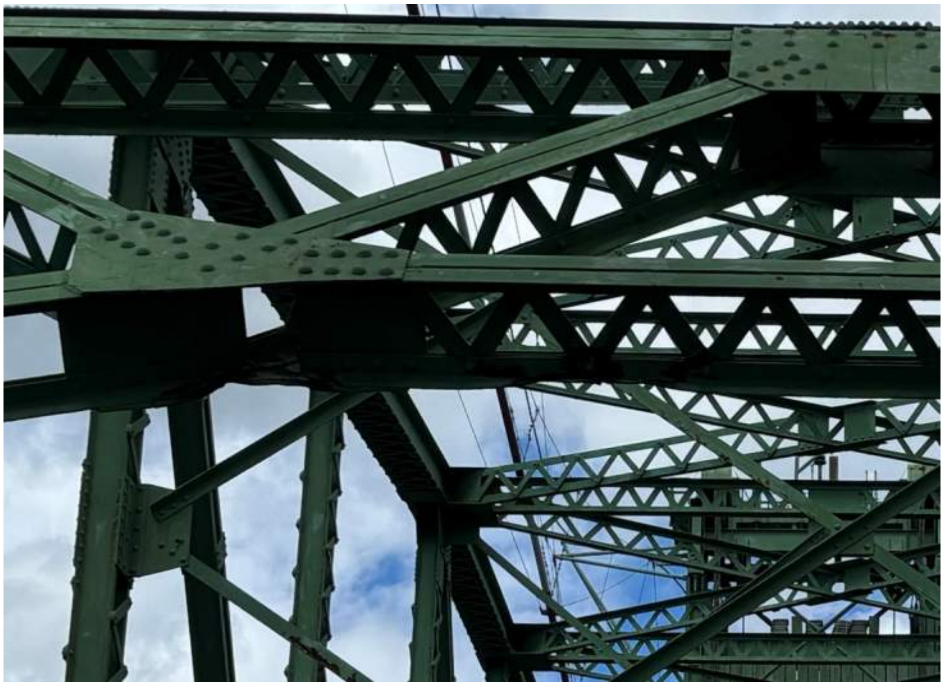
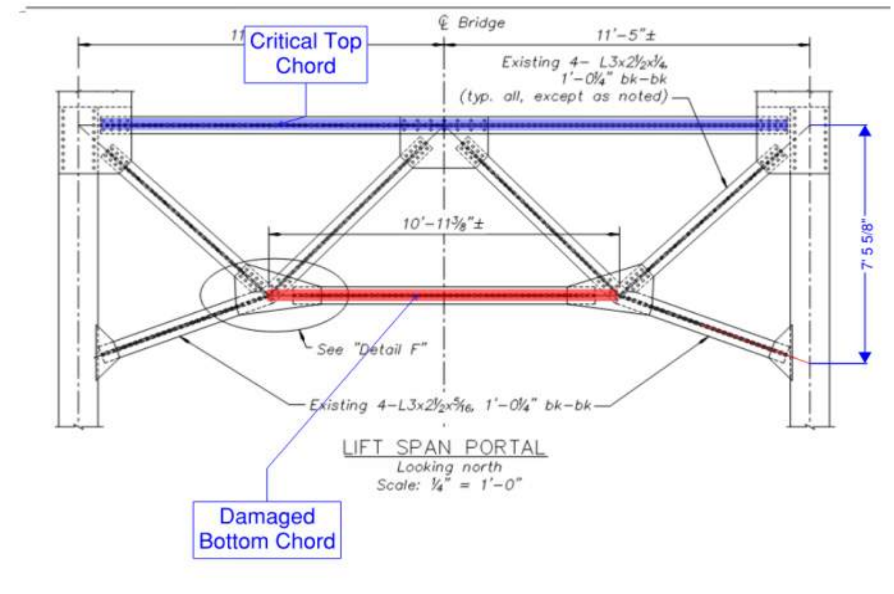
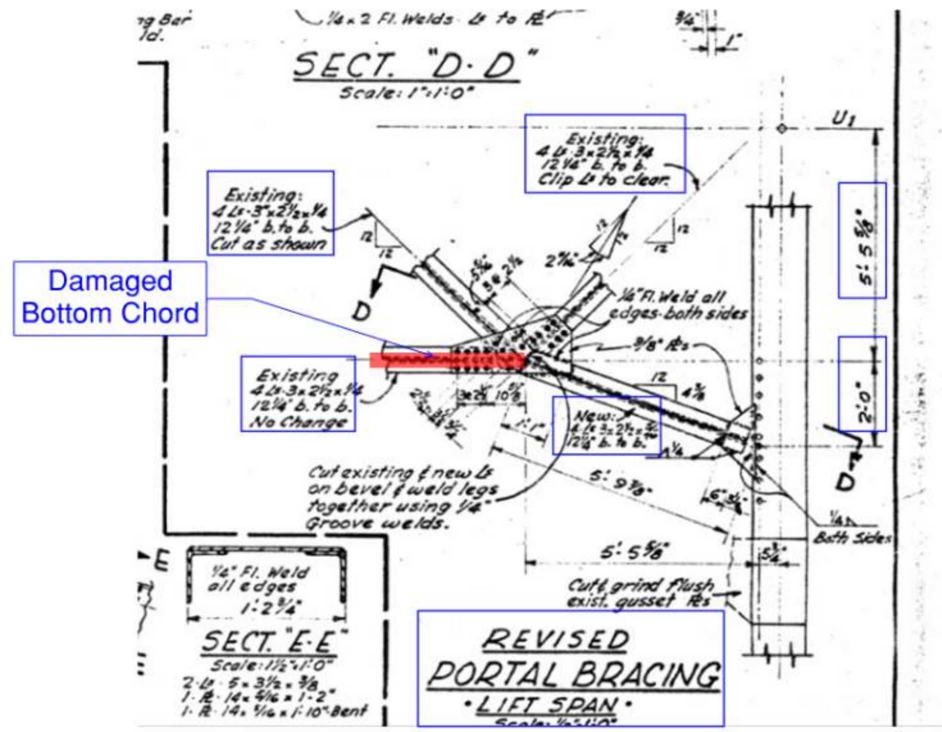


Figure: Damaged bottom chord

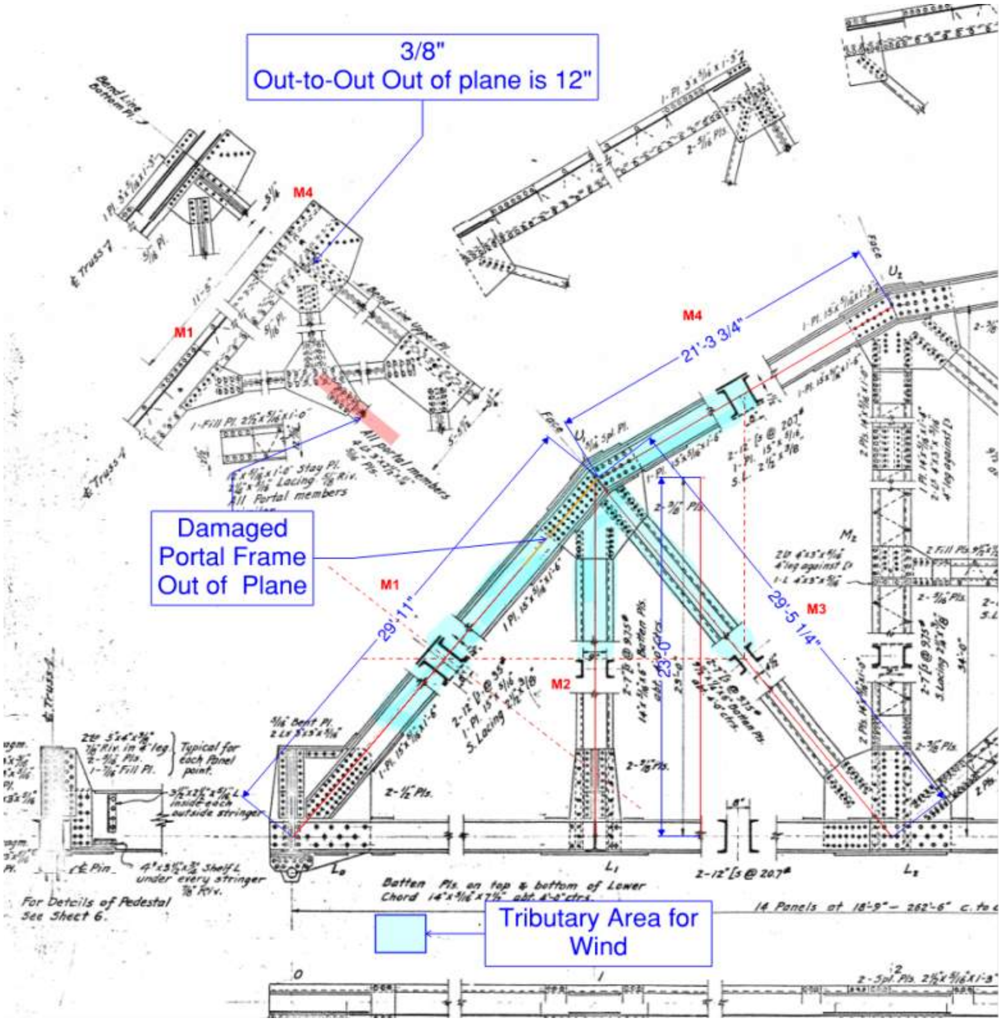
RELEVANT PLANS



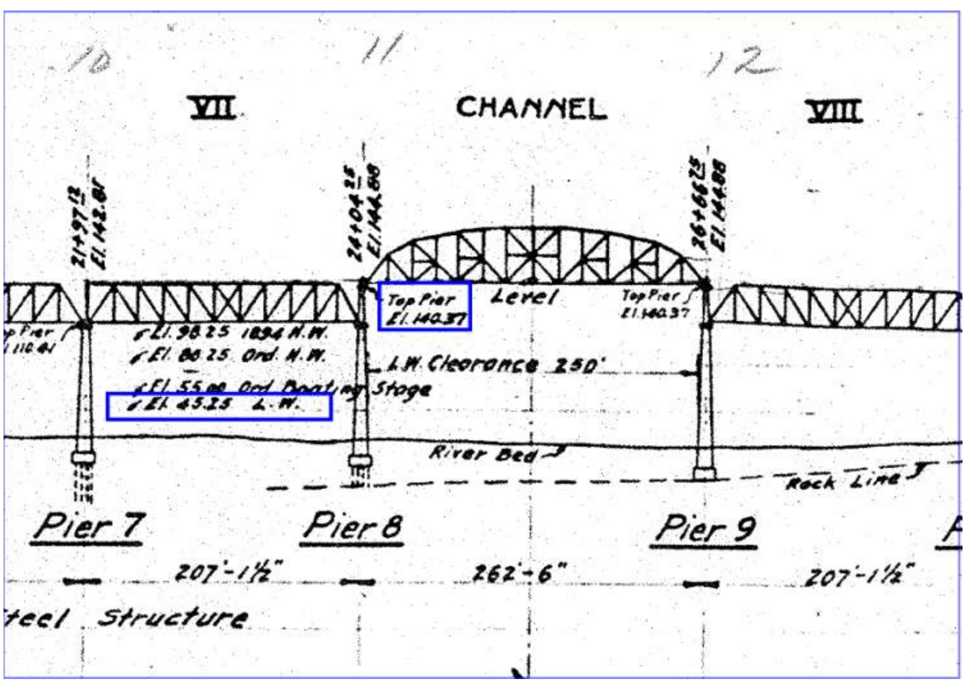
Reference: 2019 Plans (DWG No: 102770)



Reference: 1967 Hood River Bridge Portal Bracing "Revisions" (1 Page)

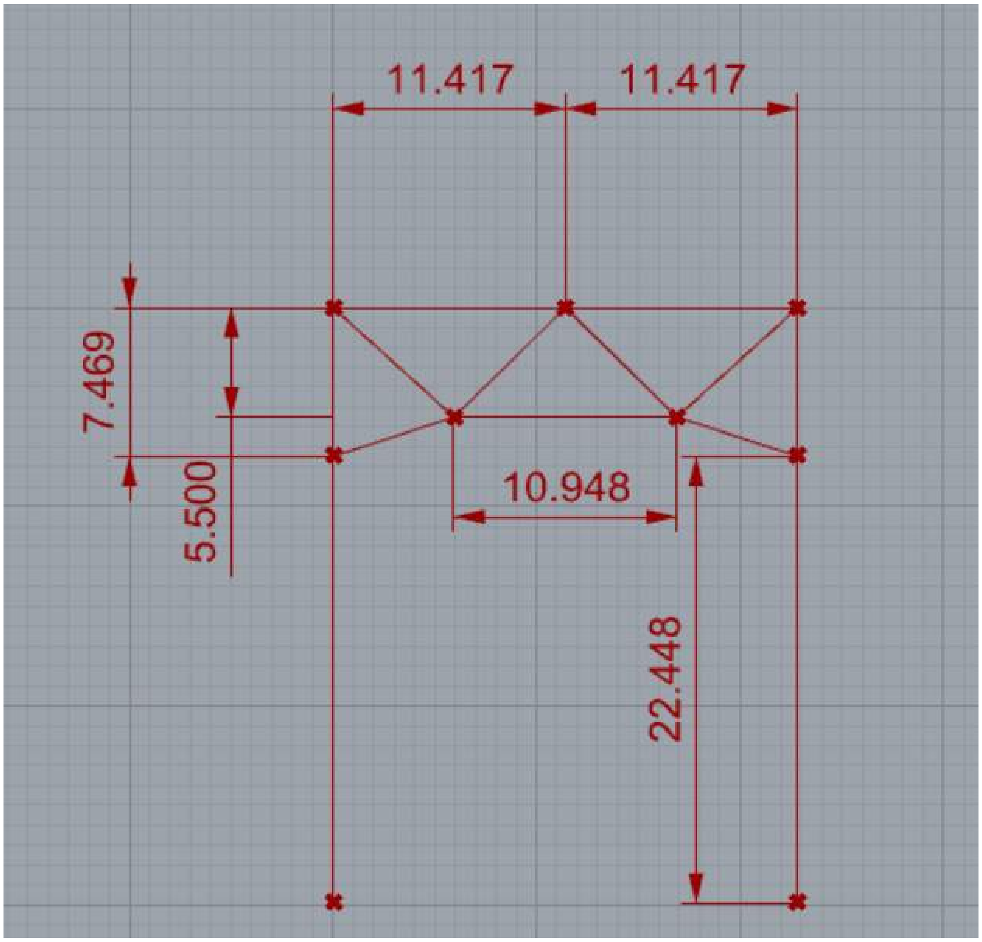


Reference: 1923 Bridge Plans (Page 8 of 9)



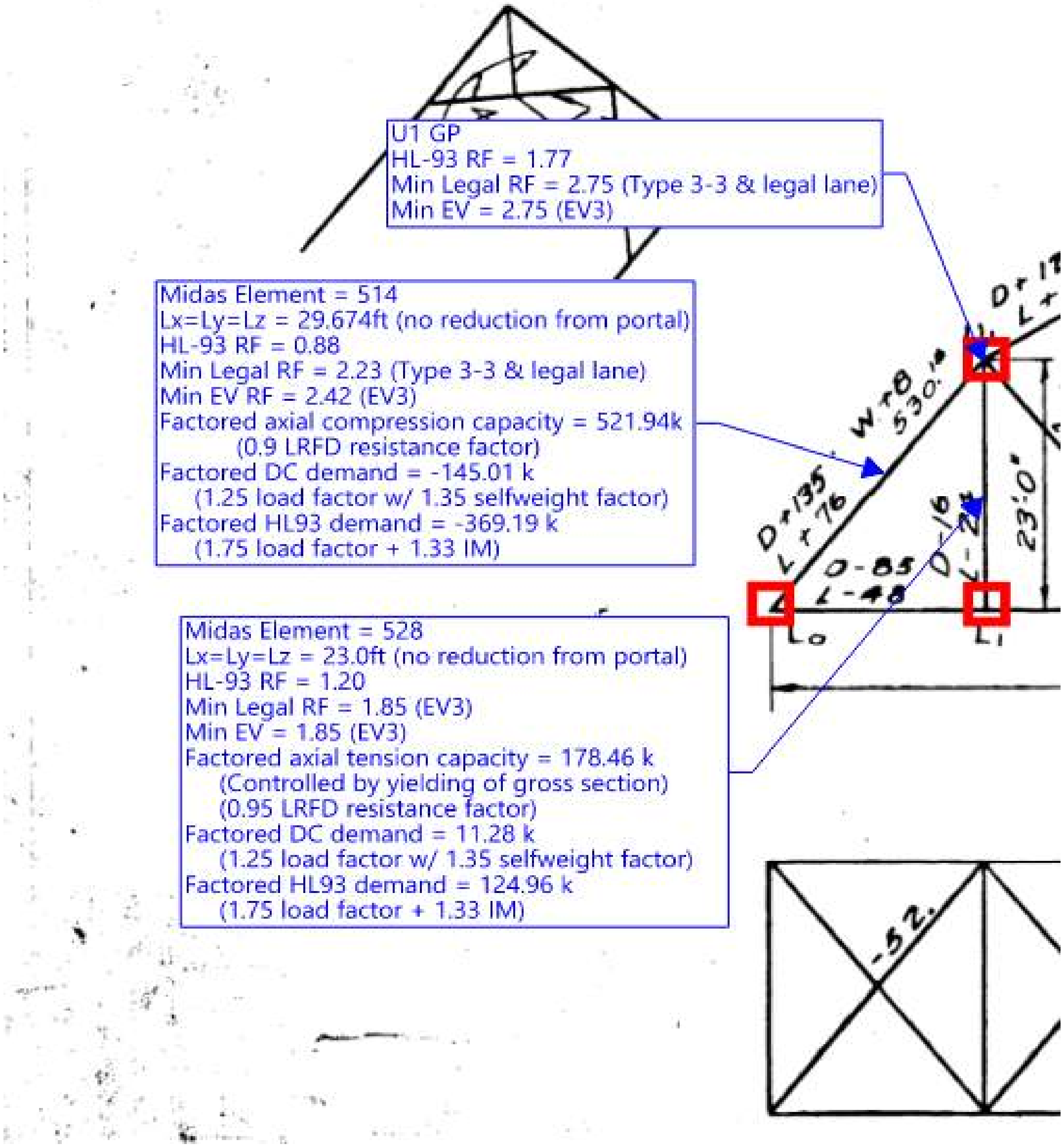
Reference: 1923 Bridge Plans (Page 1 of 9)

MODEL GEOMETRY



BRACING DEMAND

From TL's review of existing load rating files:



Factored Axial Capacity	521.94 k	<<For Midas Element 514
Bracing Force	26.097 k	<<5% of the axial capacity of member 514

Reference:
AASHTO LRFD Bridge Design Specs, 9th Edition

Input Parameters

V:	98	mph	*Design 3-second gust wind speed, ODOT BDM 1.3.9.2*
Expsoure:	D		*Wind Expsosure category per AASHTO (3.8.1.1.5)*
Z	95.14	feet	*Structure height, AASHTO 3.8.1.2.1*

Table 3.8.1.2.1-1—Gust Effect Factor, *G*

Structure Type	Gust Effect Factor, G
Sound Barriers	0.85
All other structures	1.00

G	1	Gust Factor

Table 3.8.1.2.1-2—Drag Coefficient, C_D

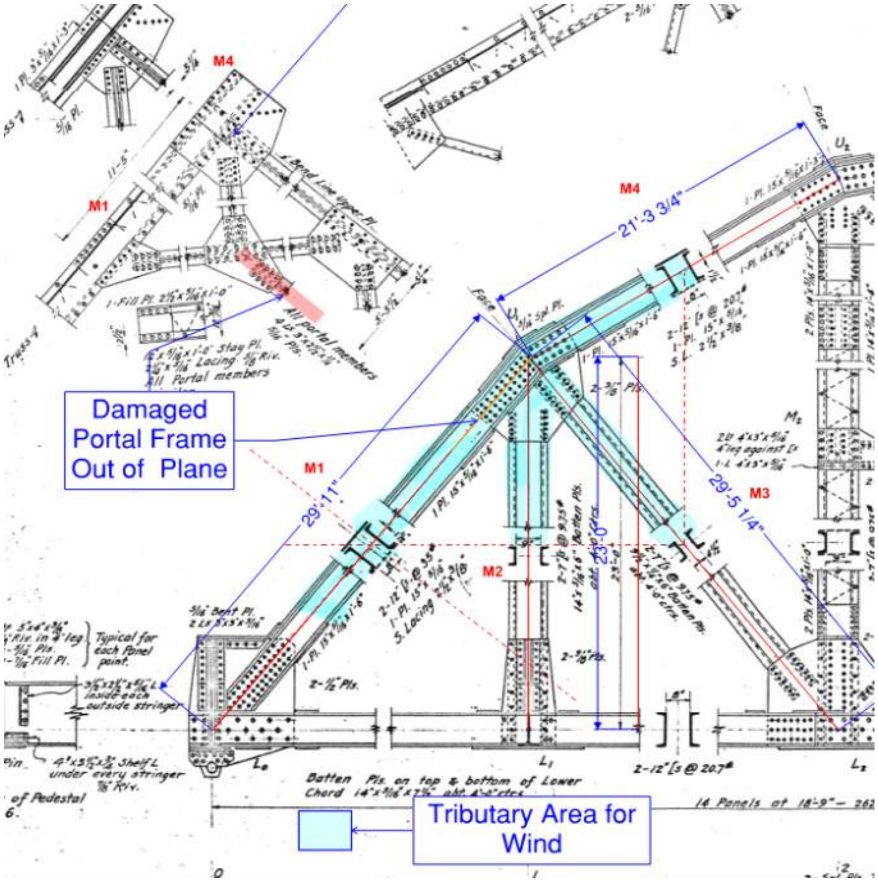
Component		Drag Coefficient, C_D Windward	Leeward
I-Girder and Box-Girder Bridge Superstructures		1.3	N/A
Trusses, Columns, and Arches	Sharp-Edged Member	2.0	1.0
	Round Member	1.0	0.5
Bridge Substructure		1.6	N/A
Sound Barriers		1.2	N/A

$C_{D\_Windward}$	2
$C_{D\_Leeward}$	1

Pressure Exposure and Elevation (Kz) Calculation And Pressure Force

Kz(B)	-
Kz(C)	-
Kz(D)	1.39
Kz	1.39

Pz	**0.069**	**ksf**	Wind pressure on Structure, AASHTO 3.8.1.2.1-1



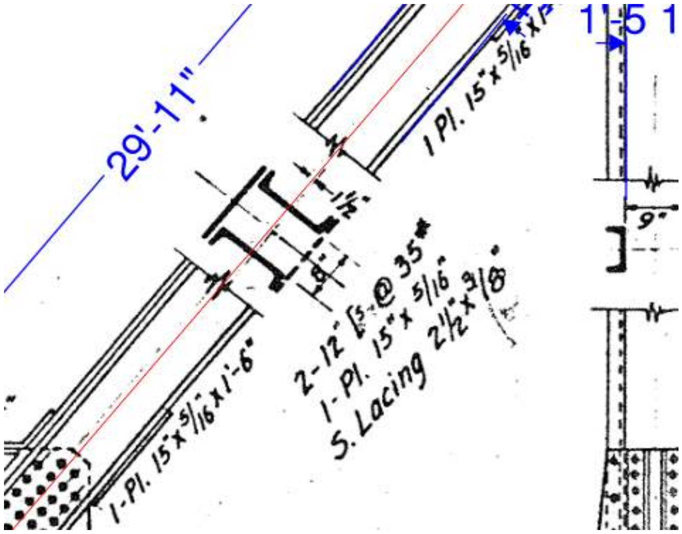
Tributary Area Calculation

	L_Total(ft)	TribFactor	L_Trib (ft)	Width (in)	Area (ft^2)
M1	29.92	0.75	22.4375	12.75	23.84
M2	23.00	0.5	11.5	14.3	13.70
M3	29.44	0.5	14.71875	9.1	11.16
M4	21.31	0.5	10.65625	12.75	11.32
Gusset Area					35.00
					95.03

<<Based on talks with ST, conservative

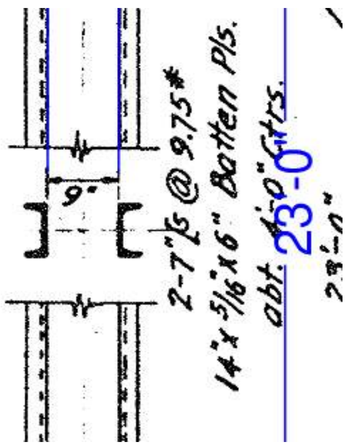
Individual Component

M1: Trib Area



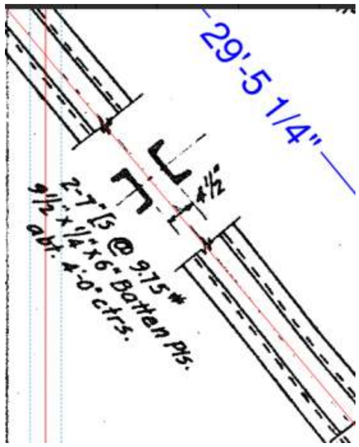
Channel Height	12	in
Lacing Thickness	0.75	in
	12.75	**in**

M2: Trib Area



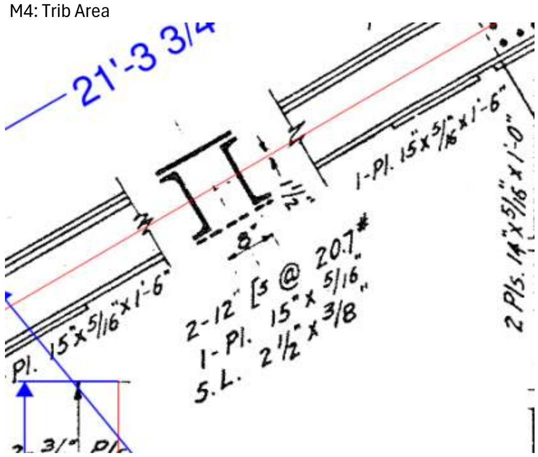
Channel Spacing	9	in	
Channel Flange	2.65	in	*Max flange width for 9" C shape, AISC Table 1-5*
	14.3	in	

M3: Trib Area



Channel Spacing	4.5	in	
Channel Flange	2.3	in	*Max flange width for 7" C shape, AISC Table 1-5*
	9.1	**in**	

M4: Trib Area

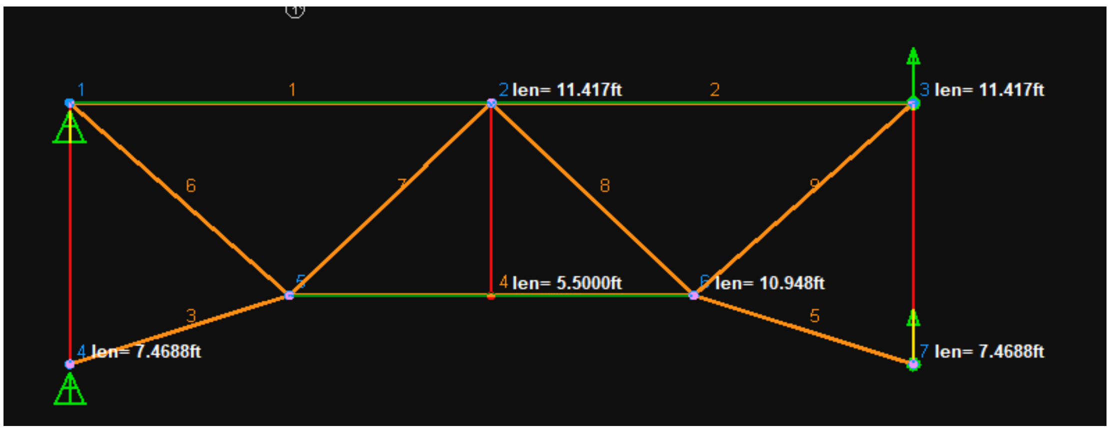


Channel Height	12	in	
Lacing Thickness	0.75	in	*Max flange width for 7" C shape, AISC Table 1-5*
	12.75	**in**	

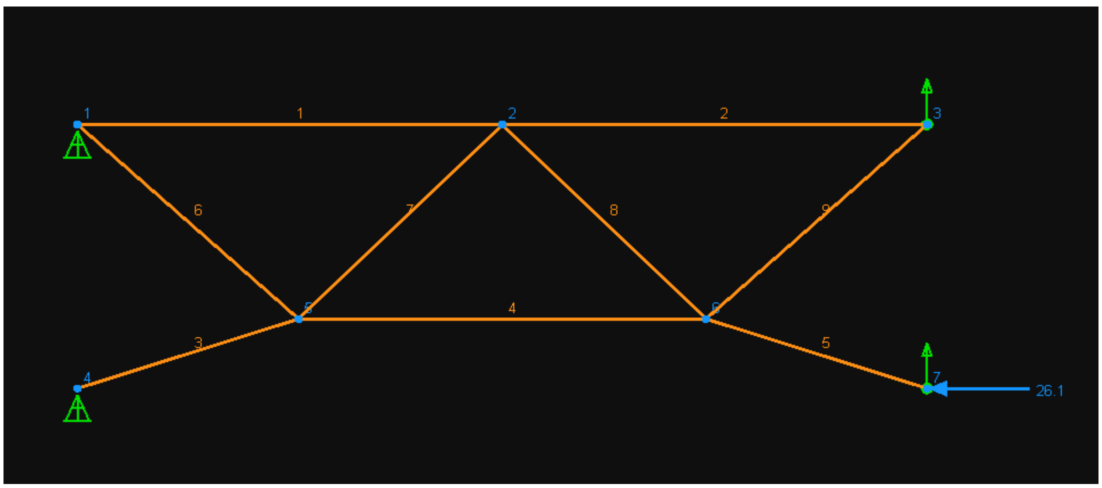
WS-III WIND PRESSURE INDUCED TRUSS LOADS

Wind Pressure	0.069	ksf	Refer Wind Pressure Tab
Amplification Factor	1.5		Guide Spec for Wind Forces During Construbtion, 4.2.2.2
Wind Area	95.03	ft^2	
Total Wind Load	9.78	k	
Load on Top Chord	4.89	k	
Load on Bottom Chord	4.89	k	

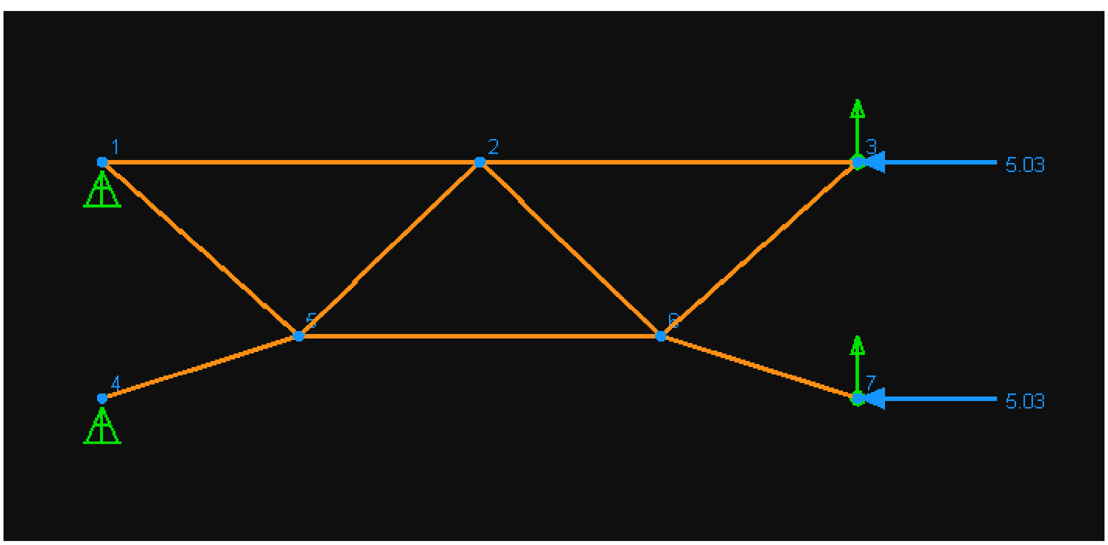
LARSA Model



Bracing Load



WS Loads - STR III



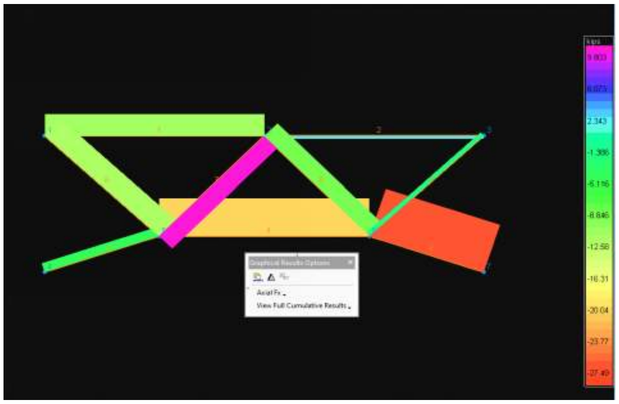
DEMAND

Axial Demand

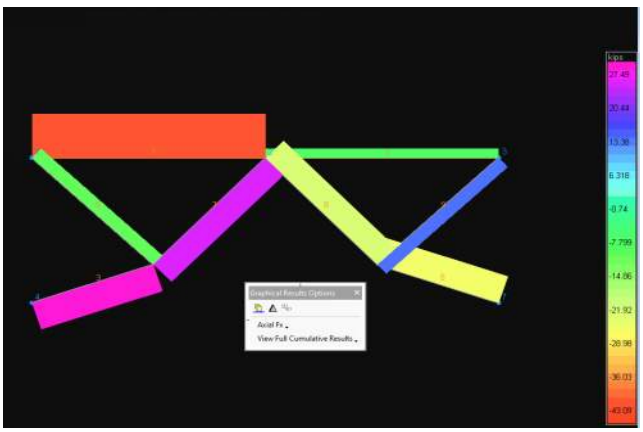
STR-III Wind	4.89	k	Refer WS Tab
Bracing	26.097	k	Refer Bracing Tab

Analysis Results: Assuming middle bottom chord goes out of service

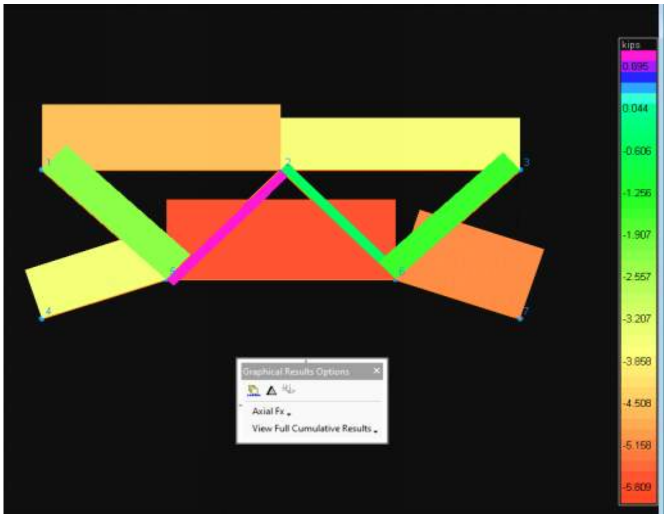
Bracing Force before damage



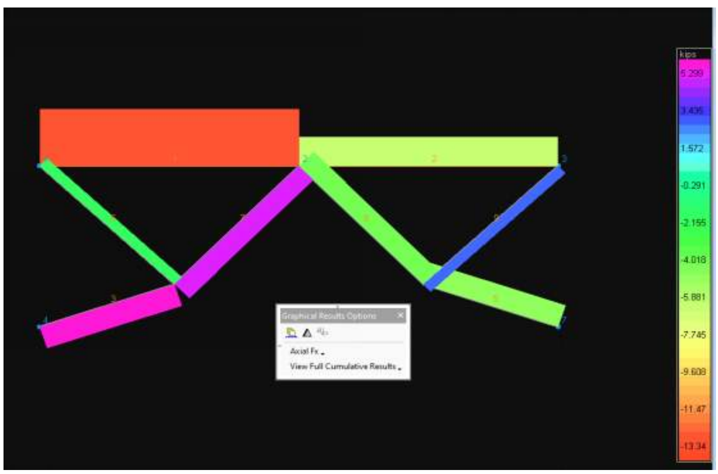
Bracing force after damage



WS-STRIII before damage



WS-STRIII after damage



Note: Top chord is the critical member for checking.

Ancticipated Maximum Force	-56.43	kips	<<Bracing+WS-STRIII, Post damage

Hood River-White Salmon Bridge Truss Tower Damaged Portal Frame Capacity Evaluation

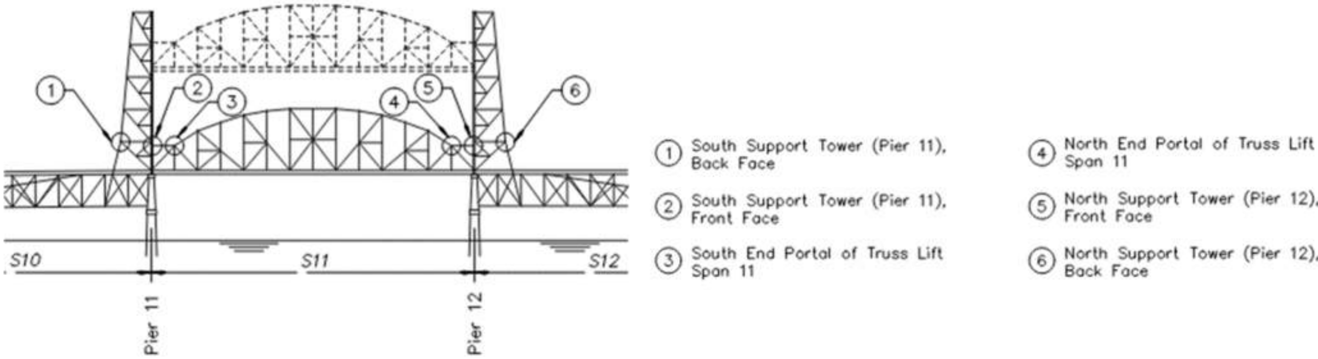
HDR Engineering, Inc.

6/27/2024

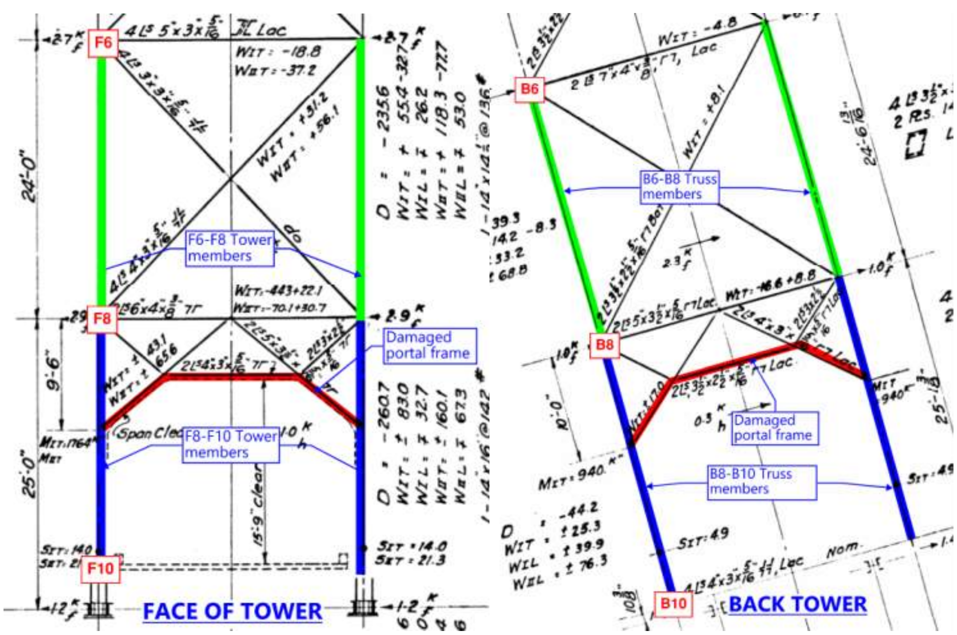
Evaluation by: Timothy Link, PE

Checked by: Mikal Mitchell, PE

Overview



The front and back portal frames of the North and South lift span support towers of the Hood River-White Salmon Bridge were damaged by a vehicle on 6/27/2024. An analysis of the primary tower members of the back and front towers of the lift span was performed assuming the portal frames were no longer bracing the towers. The slenderness ratio and associated compressive capacity of these primary tower members were compared to the unaffected members above to determine if the reduced capacity controlled the vertical capacity of the towers. The figures below show the face and back of the tower with the substantially damaged portal frame highlighted in red, primary tower members with increased unbraced length in blue, and the primary tower members in the bay above in green.

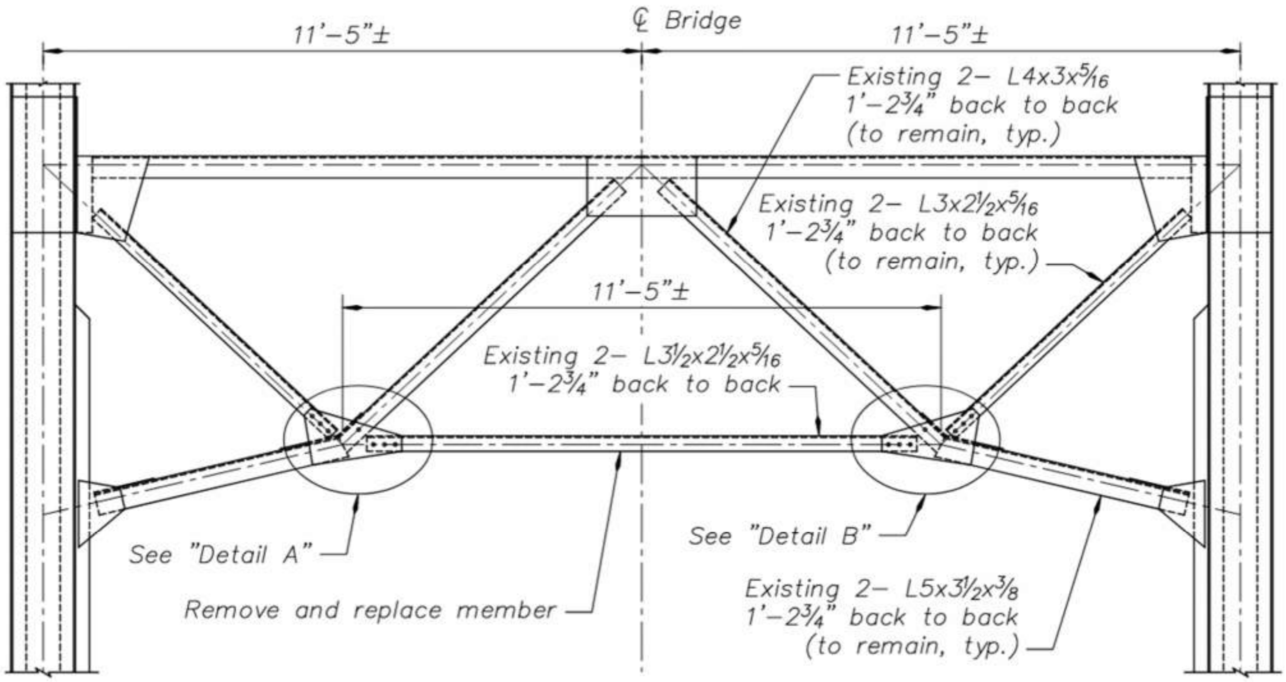


Analysis Methodology

The primary tower members braced by the damaged portal frames (F8-F10, front tower, and B8-B10, back tower) were evaluated with and without considering the lateral bracing provided by the portal frames. The similar truss members above (F6-F8 and B6-B8) that are unaffected by the damaged portal frames were also evaluated for comparison. The evaluation was based on the slenderness ratio (KL/r), the associated elastic flexural buckling resistance, and the resulting factored axial compressive resistance. The increase in slenderness ratio and associated reduction in capacity of the F8-F10 and B8-B10 members were compared against the unaffected F6-F8 and B6-B8 to determine if the overall capacity of the towers was reduced. The axial capacity of the tower members were determined using the ODOT Truss_Element_LRFR_v3.7.xlsm tool. The input and output files are included at the end of this document.

Analysis Assumptions

- The slenderness ratios and associated axial capacity in the direction perpendicular to the portal frames has not changed due to the damaged portal frames



NORTH TOWER BACK FACE PORTAL
Looking north
Scale: ¼" = 1'–0"

- Using the ODOT tool, the unbraced lengths in the perpendicular direction and torsional unbraced length were set to 1 foot to consider buckling only in the plane braced by the portal frames.
- Back of tower member assumptions:
  - Single lacing are 3/8" x 2 ½" flat bars
  - Angle to top/bottom plate rivet spacing is 6 inches
  - Rivets are 5/8" diameter
- Front of tower member assumptions:
  - The two 3.5"x3.5"x0.75" angles attached to exterior face of the wide flange beam were ignored in the calculations for all elements

Analysis Results

Front Tower Members

The slenderness ratio and elastic flexural buckling resistance, Pe, decrease when assuming the portal frame no longer braces the primary F8-F10 front tower members. The factored axial compressive resistance, Pr, is larger than the F6-F8 members above due to the larger gross area of the member. Therefore, the reduced capacity of member F8-F10 does not control the capacity of the tower. Member F8-F10 (#1) represents the primary tower member connected to the front portal frame and F8-F10* (#2) is the same member, but without the portal frame brace point. Member F6-F8 (#3) is the primary front tower member above the portal frame.

Member Location	Member #	Shape	Gross area (in^2)	Unbraced length (ft)	Slenderness ratio (KL/r)	Pe (kips)	Pr (kips)
F8-F10	1	WF14x16 @142# w/ (2) L3.5x3.5x0.75	41.54	15.500	25.74	17942	1195
F8-F10* (damaged)	2	WF14x16 @142# w/ (2) L3.5x3.5x0.75	41.54	25.000	41.52	6897	1135
F6-F8	3	WF14x14.5 @136# w/ (2) L3.5x3.5x0.75	39.67	24.000	39.91	7127	1091

Back Tower Members

The slenderness ratio and elastic flexural buckling resistance, Pe, decrease when assuming the portal frame no longer braces the primary B8-B10 back tower members. The factored axial compressive resistance, Pr, is larger than the similar B6-B8 members above due to the larger gross area of the member. Therefore, the reduced capacity of member B8-B10 does not control the capacity of the tower. Member B8-B10 (#4) represents the primary tower member connected to the back portal frame and B8-B10* (#5) is the same member, but without the portal frame brace point. Member B6-B8 (#6) is the primary back tower member above the portal frame.

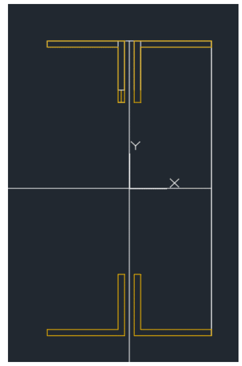
Member Location	Member #	Shape	Gross area (in^2)	Unbraced length (ft)	Slenderness ratio (KL/r)	Pe (kips)	Pr (kips)
B8-B10	4	Built-up box: (4) L3.5x3.5x3/8 w/ 14x3/8 side pls & SL top & bot	20.44	15.115	29.66	6650	582
B8-B10* (damaged)	5	Built-up box: (4) L3.5x3.5x3/8 w/ 14x3/8 side pls & SL top & bot	20.44	25.115	49.28	2408	540
B6-B8	6	Built-up box: (4) L3.5x3.5x5/16 w/ 14x5/16 side pls & SL top & bot	17.11	24.568	48.03	2122	455

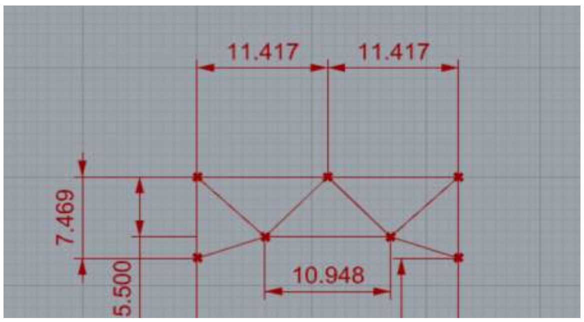
The member numbers in the tables above correspond to the input and output text file names.

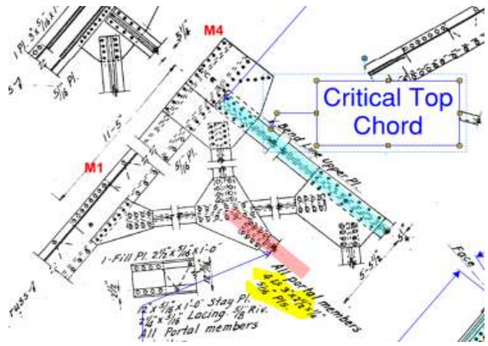
COMPRESSION CAPACITY CHECK FOR TOP CHORD

```
 ----------------   REGIONS   ----------------

Area:                     5.2500
Perimeter:                44.0000
Bounding box:          X: -3.1875  --  3.1875
                       Y: -6.0000  --  6.0000
Centroid:              X: 0.0000
                       Y: 0.0000
Moments of inertia:    X: 152.6406
                       Y: 11.0244
Product of inertia:   XY: 0.0000
Radii of gyration:     X: 5.3921
                       Y: 1.4491
Principal moments and X-Y directions about centroid:
                       I: 152.6406 along [1.0000 0.0000]
                       J: 11.0244 along [0.0000 1.0000]
```







A	5.25	in^2			<<Cross sectional area
Ix	152.64	in^4			<<MOI Major Axis
Iy	11.02	in^4			<<MOI Minor Axis
rx	5.39	in			<<Radius of Gyration
kx	1.00				<<Effective length factor
lx	137.00	in			<<Unbraced length of the top chord
rx	5.39				
kx*lx/rx	25.41	<=	120		<<AASHTO 6.9.3
DC	**0.21**		OK		

Critical Compression Stress (AISC)

AISC Table 4-14 for Fy=36ksi

Lc/r	phi_c*Fcr (ksi)
25	31.4
26	31.3

Lc/r	-31.36	ksi	<<	Compression Capacity, Stress
fc	-10.75	ksi	<<	Compression Demand, Stress
DC	**0.34**		<<	OK

HDR

Tower Transverse Lateral Force Capacity

Overview

The purpose of this section is to assess the ability of the tower to resist lateral wind loads.

LOADS CONSIDERED:

- Wind with the lift span in the lowered position

LOADS NOT CONSIDERED:

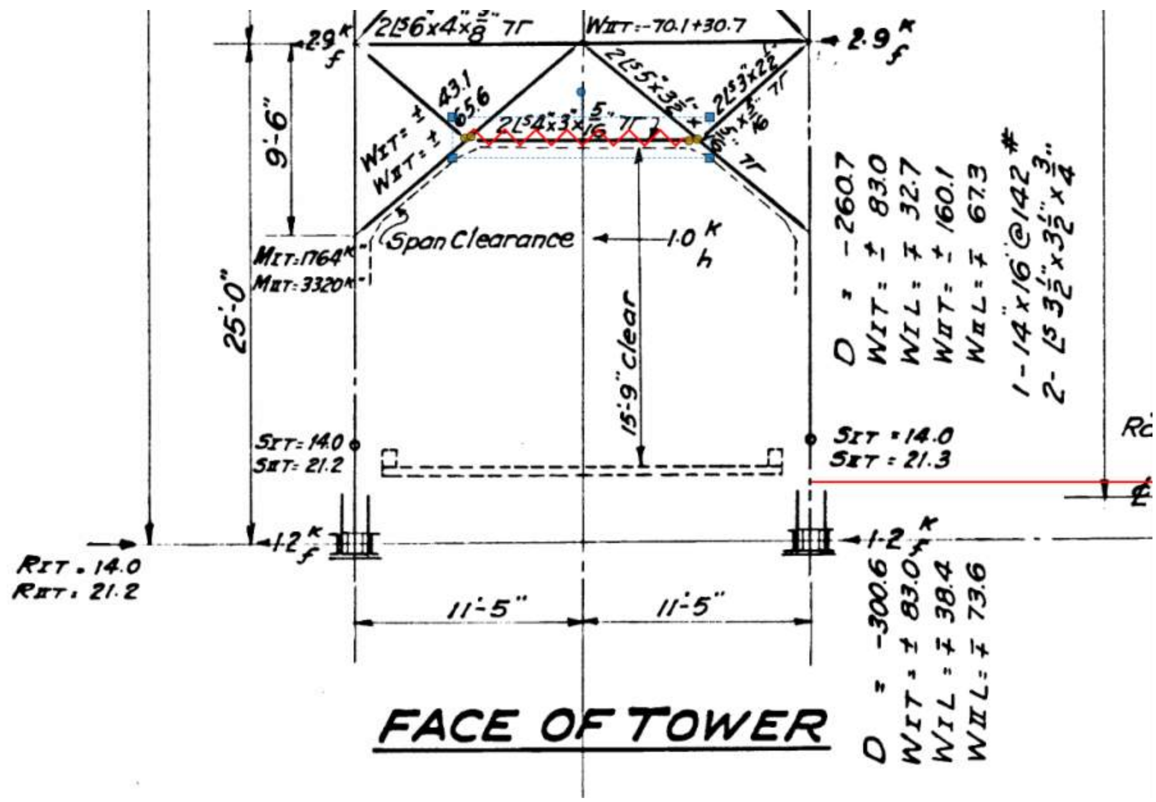
- Seismic
- Wind with the lift span in the lifted position

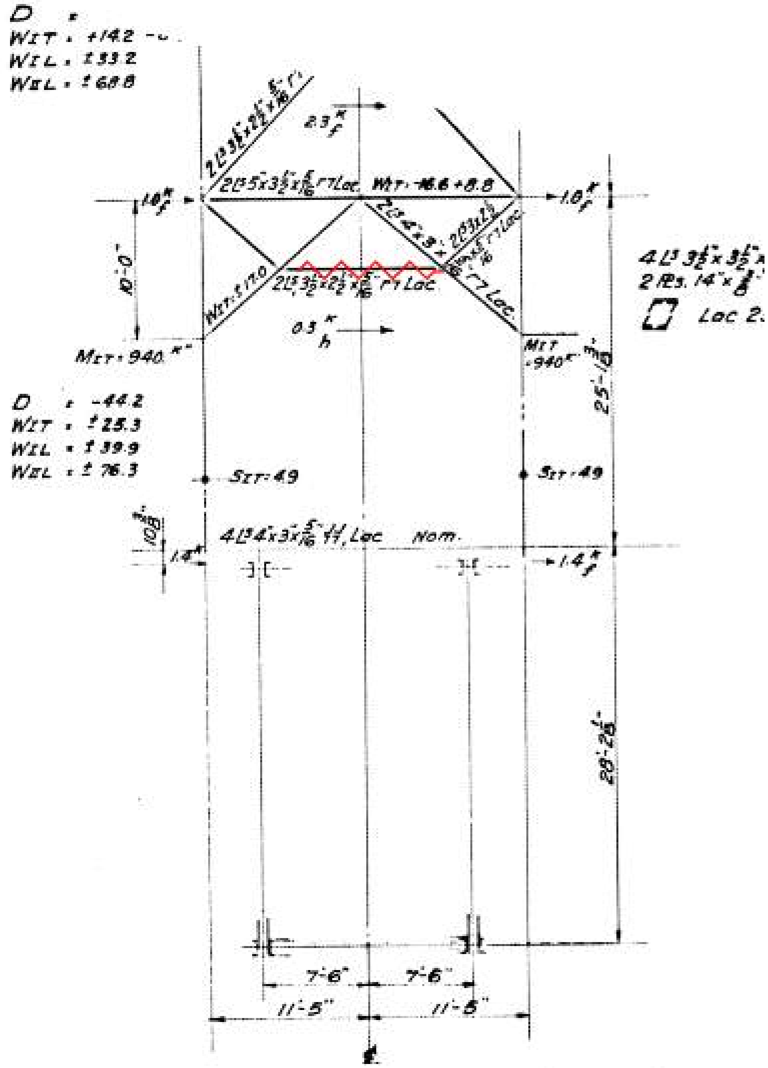
REFERENCES:

- Original 1937 design plans
- Revised configuration in 1967 plans

Damaged Member

The substantially damaged member for this analysis is shown below. The analysis appears to show that this bottom chord member of the portal truss can be completely removed with no adverse impact on the structure's ability to resist wind loads.





BACK OF TOWER

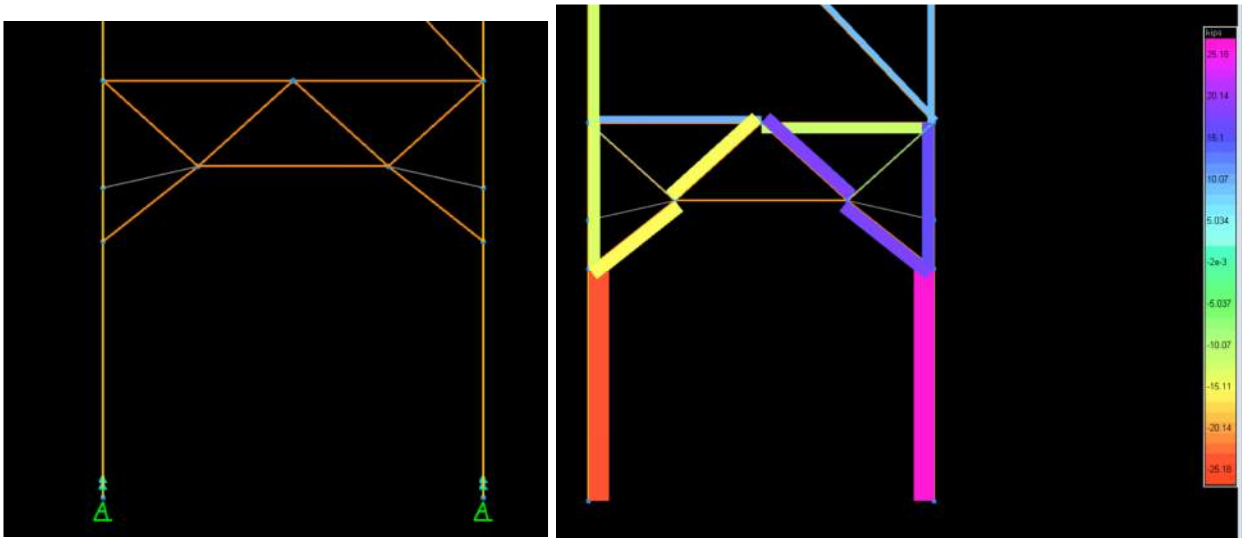
(DEVELOPED).

SCALE: 1/8" = 1'-0"

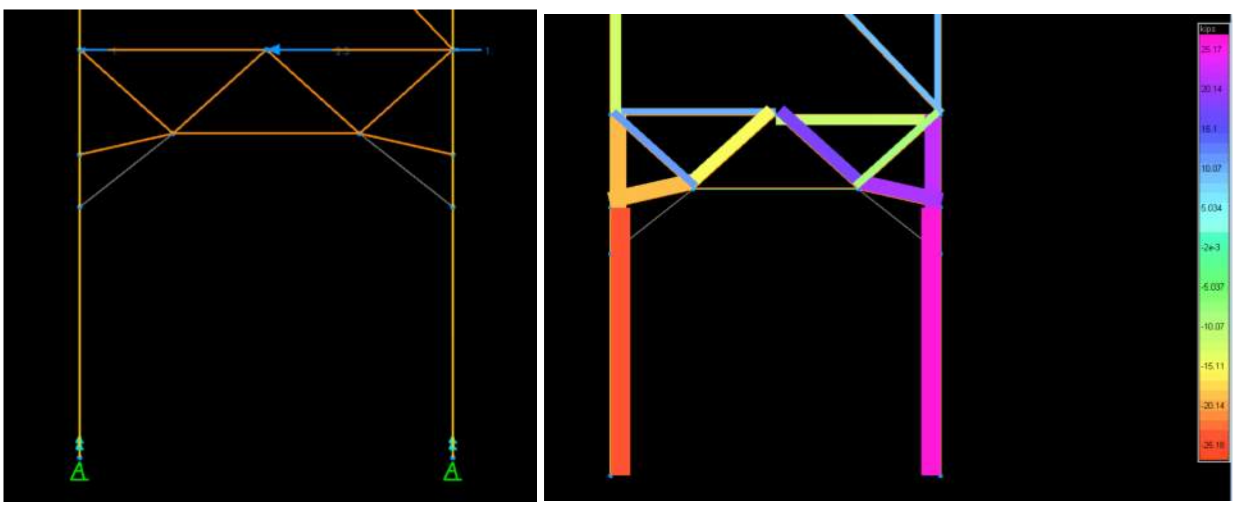
Evaluation of wind loads for Back Truss

The back truss was evaluated using LARSA models of the isolated back truss with loads applied per the original design plans. Loading is not independently calculated. Construction staging is used in the model to change the portal bracing scheme from the original geometry to the 1967 geometry. Staging is also used to remove the damaged member from the analysis.

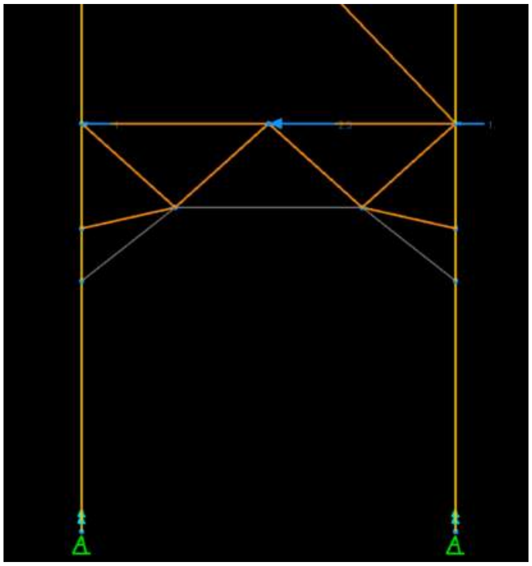
For simplicity, only the results of the analysis are shown below. The member forces are in reasonable agreement with the values in the original plans. Only the area around the damaged portal brace is shown.

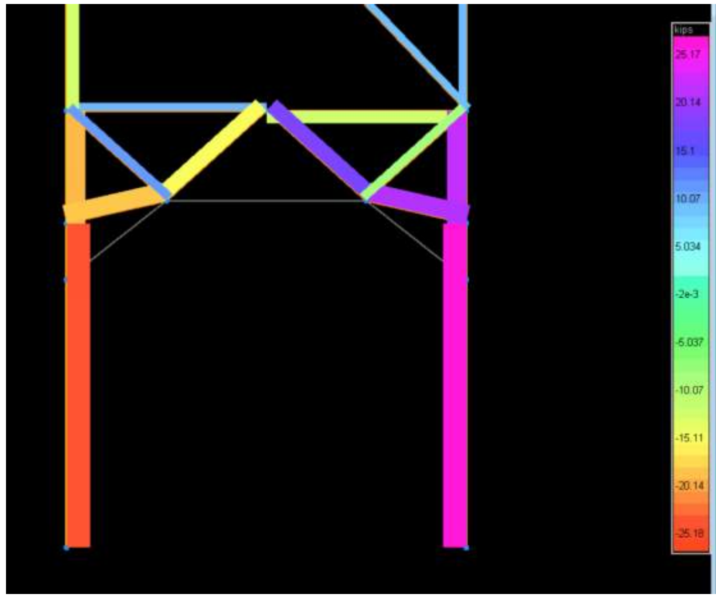


Original configuration (left) with member axial forces (right). Note the damaged bottom chord is a zero force member under this loading.



Modified 1967 configuration (left) with member axial forces (right). Note the damaged bottom chord is effectively still a zero force member carrying less than 100lbs of force.





Modified 1967 configuration with damaged member removed (left) with member axial forces (right). Note the remaining member forces appear to be essentially unchanged, with only minor differences resulting from re-distribution of the approximate 100-lb force previously present in the bottom chord that was removed. Maximum member axial force increases were less than 1% when re-analyzed in this condition.

Evaluation of wind loads for Front Truss

The Front Truss is a similar configuration to the back truss. Due to the reverse curvature moment being resisted by the portal truss and the damaged member being located at a location with zero moment, the damaged member should have zero force by inspection similar to the back truss analysis.

Conclusion

The damaged bottom chord of the portal truss appears to be a zero force member under this loading. The portal brace is subject to reverse curvature bending from transverse wind forces and an inflection point for moment is expected at the mid point of the system. The bottom chord is a zero force member as a result of there being zero moment at the transverse mid point of the portal system. Note that this analysis is also valid for the front truss.

Port of Hood River Commission Meeting

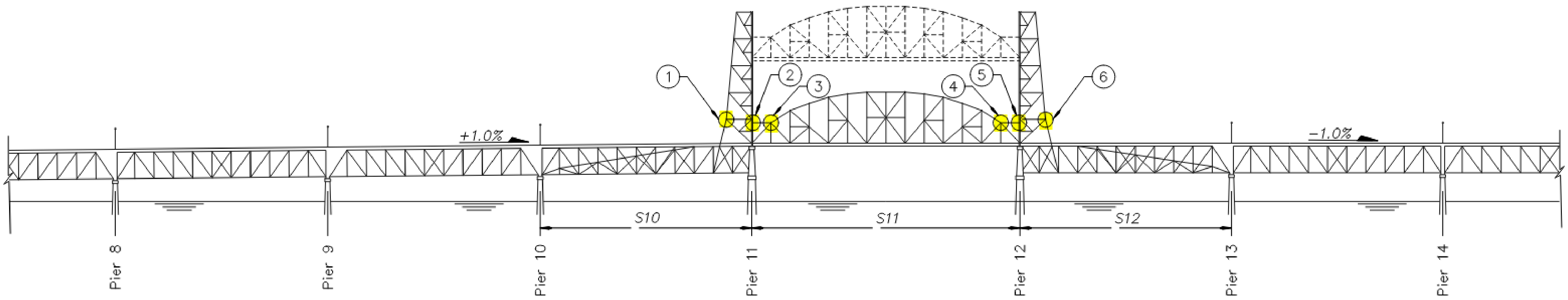
June 30, 2024

Justin Doornink, PE (HDR)
Eric Rau, PE (HDR)

Bridge Assessment Field Observations

- Six (6) damage locations to overhead portal braces were identified
    - South Support Tower (Pier 11), Back Face
    - South Support Tower (Pier 11), Front Face
    - South End Portal of Truss Lift Span 11
    - North Support Tower (Pier 12), Back Face
    - North Support Tower (Pier 12), Front Face
    - North End Portal of Truss Lift Span 11

Bridge Assessment Field Observations



1. South Support Tower (Pier 11), Back Face
2. South Support Tower (Pier 11), Front Face
3. South End Portal of Truss Lift Span 11
4. North End Portal of Truss Lift Span 11
5. North Support Tower (Pier 12), Front Face
6. North Support Tower (Pier 12), Back Face

Bridge Assessment in Current Damaged State

- Lift Support Tower (Vertical & Horizontal Load Assessment)
    - 4 locations
- Lift Truss Span (Vertical & Horizontal Load Assessment)
    - 2 locations

Recommendations

- Temporary Condition: The existing bridge lift span should remain in place and not be raised until all damaged portal bracing is repaired.
- At the discretion of the Port, marine traffic under the bridge may continue provided the Port can determine that there is sufficient vertical clearance to do so without raising the lift span.
- At the discretion of the Port, reopening of the bridge to vehicular traffic may proceed provided that vehicles are compliant with the posted load rating of the bridge.
    - Implementation: Limit traffic to cars to mitigate the potential for another accidental strike to the overhead bracing while the bridge is in a damaged state.
- Design of final bridge repair plans, specifications, and estimate should be expedited with the resulting repairs to follow immediately thereafter.

Next Steps

- HDR to continue discussions between Kiewit on best means, methods, and constructability (already in progress)
- Continue with final bridge repair plans, specifications (HDR)
- Construct bridge repairs (Kiewit)