Memorandum

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Subject	**Firerock Footbridge Condition Assessment**	**Project Name**	Design of CSEP - Rimrock Pump Stations Improvements Project
Attention	Jason Suhr/City of Bend	**Project No.**	Jacobs: D3380200
From	Melissa Moncada, P.E. /Jacobs Nik Gordon, P.E./Jacobs Brittany Hughes, P.E. /Jacobs		
Date	October 8, 2021		
Copies to	File		

1. Introduction

The Rimrock Pump Stations Improvements Project (Project) is located in the Rimrock West neighborhood in northwest Bend, Oregon. The Project has three main components, including:

- Replacement of an existing sewer system along NW Silver Buckle Road (already bid for construction in 2021).
- Upsizing of an existing water main in NW Silver Buckle Road (already bid for construction in 2021)
- Assessment of the condition of a timber pedestrian bridge (Firerock Footbridge) that crosses the Deschutes River.

This Firerock Footbridge Condition Assessment Memorandum documents the condition assessment of the footbridge conducted by Jacobs Engineering Group (Jacobs) which included a site visit, inspection, and structural calculations to assess adequacy of the primary bridge members. A summary of the site observations, noted deficiencies, recommended bridge condition ratings from the inspection and results of the structural analysis are included in the sections below.

2. Firerock Footbridge Background and Description

The Firerock Footbridge is a 143-foot long, 4-foot wide timber pedestrian bridge crossing the Deschutes River at approximately River Mile (RM) 163, one quarter of a mile downstream (north) of the Archie Briggs Road Bridge (see Figure 1). The pedestrian bridge spans the Deschutes River between privately-owned land in the Rimrock West neighborhood on the west side of the river and a publicly owned parcel in the Rimrock Village Subdivision on the east side of the river. There is little existing or historical information available about the bridge and the City does not have any permit or inspection records. The bridge is believed to have been constructed in the mid-1970s by a developer to support a 6-inch-diameter polyvinyl chloride (PVC) waterline that supplied potable water across the Deschutes River. The water system was originally designed to convey water from a well house on the east side of the river to serve the Rimrock West neighborhood on the west side of the river. The waterline is supported under the bridge deck and covered in insulation, but is critically damaged, and not suitable for being returned to service.

The waterline and bridge infrastructure were later turned over to the City to operate and maintain. The City closed the bridge structure to foot traffic in early 2015 due to concerns regarding its structural integrity. Waterline improvements completed within the City's water system in the Rimrock West neighborhood in 2018 and 2019 have since enabled the waterline underneath the bridge to be permanently removed from service.

Figure 1: Firerock Footbridge (photo taken from west riverbank looking northeast)

The bridge is accessible to Rimrock West neighbors (on the west side of the river) from the south via a 5-foot-wide upland and riparian area pedestrian path at the end of the NW Silver Buckle Road cul-de-sac or from the northwest via a 4-foot-wide staircase and similar pedestrian path located east of NW Broken Arrow Road. Access to the bridge from the west side of the river is across private property owned by the Rimrock West Homeowner's Association. There are no public access easements on the west side of the river that provide access down to the bridge. On the east side of the Deschutes River, the bridge is accessible to pedestrians via a short path and timber stairs leading down a steep 30-foot cliff face from Firerock Road, which connects to O.B. Riley Road. Access to the publicly owned parcel on the east side of the river is via a 20-foot-wide existing public easement that is split across two privately-owned parcels. The general layout is shown in Figure 2.

Other than the access easement to the publicly owned parcel on the east riverbank, there are no known public trail systems that connect to the bridge. The Awbrey reach of the Deschutes River Trail is located west of the Rimrock West neighborhood but would require public easements to be acquired through the Rimrock West neighborhood (privately-owned streets and parcels) in order to provide any connection to the Firerock Footbridge from the west. Currently, pedestrians use the Archie Briggs Road bridge to cross the Deschutes River in this area, which is located approximately 1300 feet south of the Firerock Footbridge.

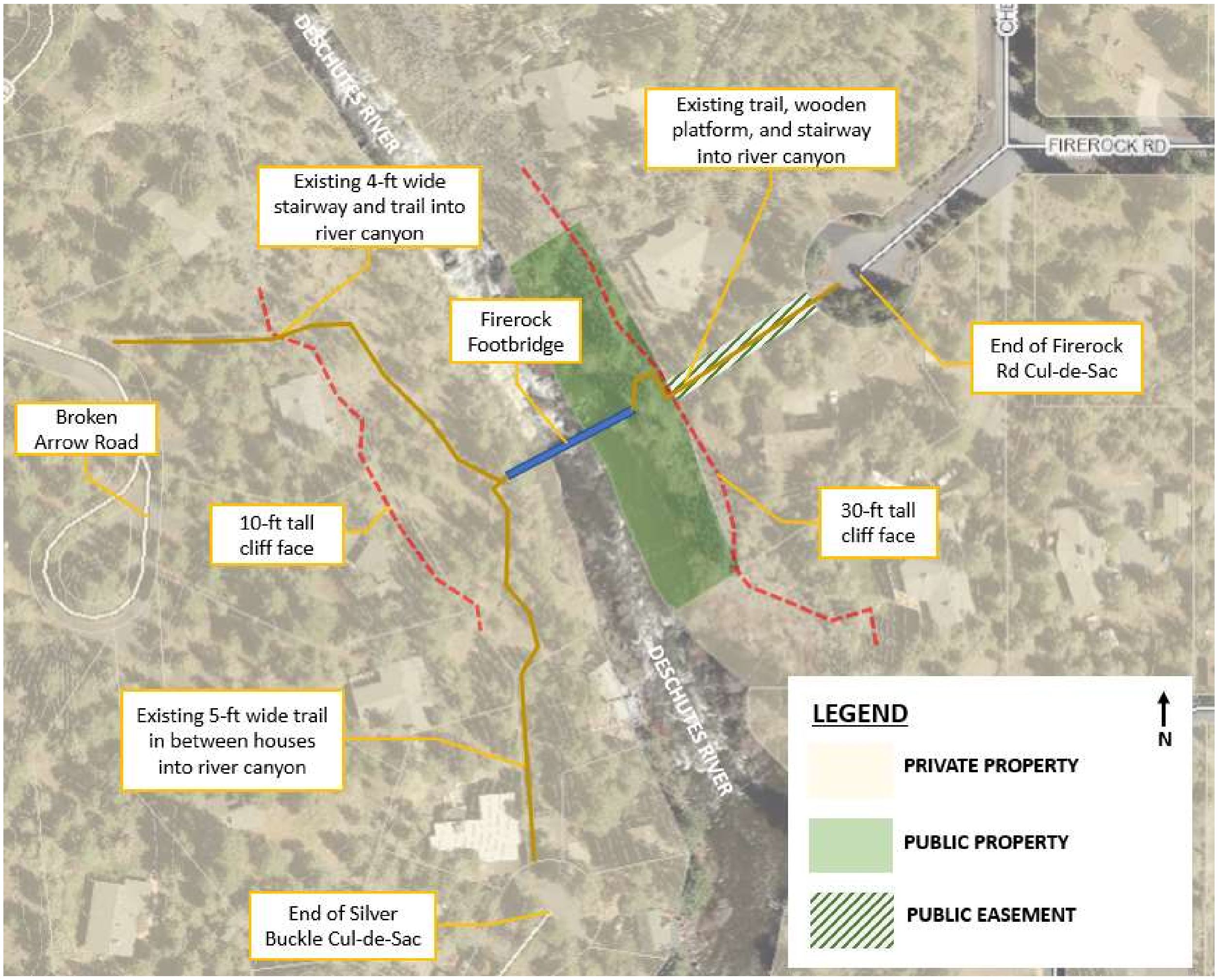

Figure 2: Firerock Footbridge Access

No available engineering design drawings exist for the bridge. Jacobs staff conducted a site visit to take measurements and document the geometry of the bridge, which is described further in the next section.

3. Firerock Footbridge Site Visit and Inspection

A site visit was conducted by Jacobs staff (Dale Wilson and Brittany Hughes) on April 29, 2021 to take measurements, document the geometry of key bridge members, and observe the bridge conditions. Condition ratings that align with the ODOT Load Resistance Factor Rating (LRFR) Manual were estimated and assigned to the bridge members.

3.1 Bridge Geometry

Following is a detailed description of the bridge geometry based on the field measurements taken using a standard steel measurement tape. Measurements were made to a ¼-inch of precision. All members listed are timber, unless noted otherwise. The sizes listed in this section are nominal timber member sizes, except for the timber glulam members which are exact sizes. The actual size of a nominal member is typically a ¼-inch, ½-inch or ¾-inch narrower width and height, depending on the nominal size.

The bridge structure is composed of eight spans, varying in length from about 5-feet long to 43-feet long, for a total bridge length of approximately 143-feet (see Figure 3). The outside bridge width is 4-feet. Supporting the deck planks and railings are two lines of beams, which bear on 4-inch x 6-inch caps (see Figure 4). The caps are directly supported by two 12-inch diameter concrete columns at bents 3 through 6. At bents 2 and 7, the caps are supported by vertical posts which are supported by the 12-inch diameter concrete columns. Bents 1 and 2 are located on the west riverbank, bents 3 through 6 are located in boulders/bedrock in the river, and bents 7 through 9 are located on the east riverbank. Bents 3 through 6 are located below the ordinary high-water mark (OHWM). There is no anchoring of the columns to the rocks that is visible and so it is not known whether they are anchored into the rock. The columns were not scanned for the presence of reinforcing (rebar) as a part of the inspection. Bents 1, 2, and bents 7 through 9 are located above OHWM but are within the floodplain.

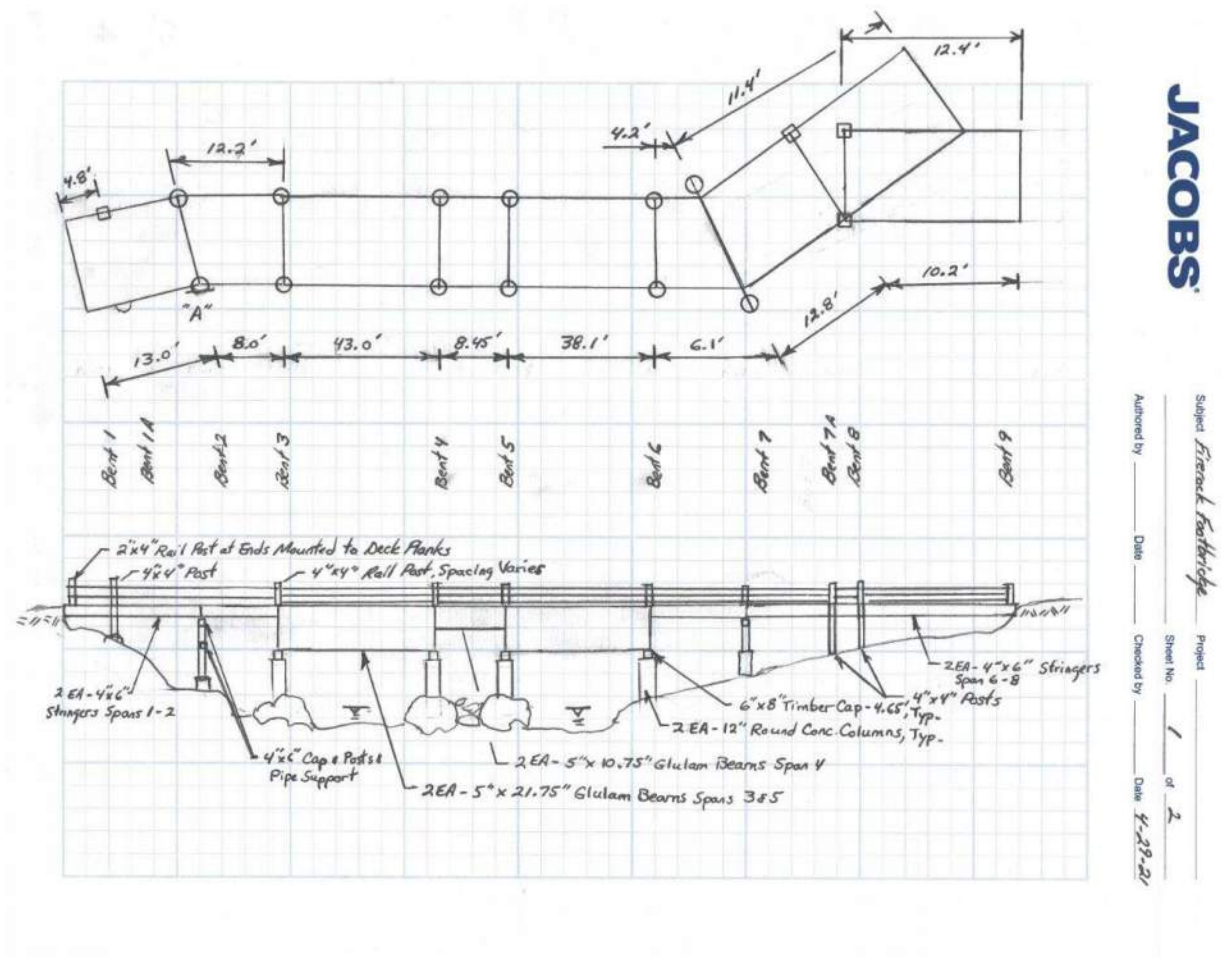

Figure 3: Firerock Footbridge field sketch sheet 1 of 2 (by Dale Wilson/Jacobs)

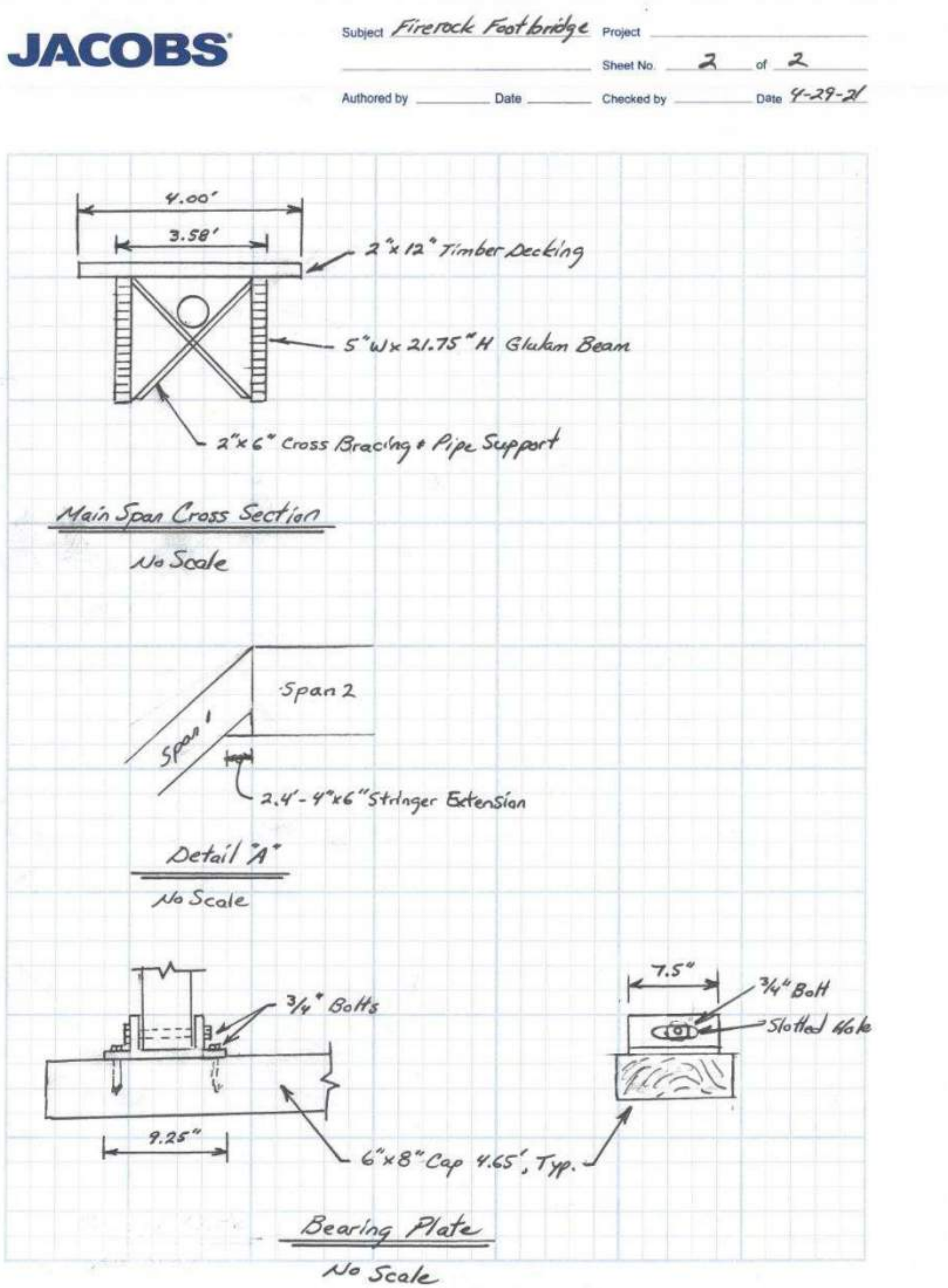

Figure 4: Firerock Footbridge field sketch sheet 2 of 2 (by Dale Wilson / Jacobs)

Spans 3 through 5, which are over water, are supported by 5-inch-wide glulam beams. The beams in spans 3 and 5 are 21.75-inches deep and the beams in span 4 are 10.75-inches deep. The remaining spans are supported by 4-inch x 6-inch beams.

The glulam beams of spans 3 and 5 bear on 6-inch x 8-inch caps which bear directly onto 12-inch round concrete columns below. There is no visible evidence that the caps are anchored to the concrete piers. The glulam beams of span 4 and the beams in span 2 and 6 hang off the ends of the deep glulam beams by hanger brackets. At bents 2, 7, and 8, the beams are hung to the sides of the caps by hanger brackets and supported by buried caps/sills at bents 1 and 9.

The railings consist of 4-inch x 4-inch vertical posts supporting 2-inch x 4-inch longitudinal railing members. The posts are 40-inches tall above deck with a typical spacing of 8-feet. The maximum post spacing is 8-feet 9-inches. There are a variety of shorter post spacings towards the ends of the bridge (see Attachment A). The posts are typically mounted to the sides of the beams. There are a few locations at the approach spans where the vertical posts are supported from the ground in addition to being bolted into the sides of the beams. The rail posts typically support three longitudinal railing members, except at spans 1 and 7, and the south side of span 8 where the posts support just two longitudinal railing members. The vertical openings between longitudinal railing members vary but are typically about 14-inches between the top two rails, about 6-inches between the lower two rails, and about 7-inches between the bottom rail to the top of deck. At locations of two longitudinal railing members the upper and lower vertical openings are typically about 14-inches and 17-inches, respectively. The maximum opening size is about 17 ¾-inches. The height of the topmost longitudinal railing members varies between 32-inches and 39 ½-inches from the walking surface. There is no safety toe rail or curb.

The walkway is typically 3-feet, 5 1/2-inches between the inside faces of the railings, with some locations of the walkway being 3-feet, 4-inches wide. The walking surface consists of 2-inch x 6-inch deck planks in spans 1 and 2 and spans 6 through 8. The deck planks in spans 3 through 5 are 2-inch x 12-inch. At several locations, 2-inch x 4-inch planks were used as replacement boards. The deck planks span over two lines of longitudinal beams spaced 3 foot-2 inches apart center-to-center. The openings between the timber deck planks vary between ¼-in and 2- inches.

3.2 Bridge Condition

Dale Wilson of Jacobs documented the condition of key bridge members in accordance with the ODOT Load and Resistance Factor Rating (LRFR) Manual, which also aligns with the Federal Highway Administration (FHWA) Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges. Condition ratings between 1 and 9 are used to indicate the condition of the member, with the latter being excellent condition (see Figure 5).

Code	Description
N	NOT APPLICABLE
9	EXCELLENT CONDITION
8	VERY GOOD CONDITION - no problems noted.
7	GOOD CONDITION - some minor problems.
6	SATISFACTORY CONDITION - structural elements show some minor deterioration.
5	FAIR CONDITION - all primary structural elements are sound but may have minor section loss, cracking, spalling or scour.
4	POOR CONDITION - advanced section loss, deterioration, spalling or scour.
3	SERIOUS CONDITION - loss of section, deterioration, spalling or scour have seriously affected primary structural components. Local failures are possible. Fatigue cracks in steel or shear cracks in concrete may be present.
2	CRITICAL CONDITION - advanced deterioration of primary structural elements. Fatigue cracks in steel or shear cracks in concrete may be present or scour may have removed substructure support. Unless closely monitored it may be necessary to close the bridge until corrective action is taken.
1	"IMMINENT" FAILURE CONDITION - major deterioration or section loss present in critical structural components or obvious vertical or horizontal movement affecting structure stability. Bridge is closed to traffic but corrective action may put back in light service.
0	FAILED CONDITION - out of service - beyond corrective action.

Figure 5: Coding guide for condition of bridge members (FHWA)

The deck was documented to be in fair condition. Generally, the deck members were sound, but issues include nails lifting out, moss growth on the ends of span 5, checking (lengthwise fissure of the wood/timber along the grain), splitting, some rotting, some section loss, and significant gaps between boards occurring in all spans. The decking boards were not the full width of the bridge at bent 2 and boards were missing in span 8. A portion of span 7 is built over the top of the west end of span 8.

The railing members were found to be in fair condition. Some of the rail post tops were found to be deteriorating with section loss.

The remaining superstructure members, including the stringers and glulam beams were found to be in good condition. Some localized issues included checking in the stringers of span 1 and the north stringer of span 2. The glulam beams appear structurally sound but have some localized section loss at a few knots in the wood and some minor checking. The bearings were in good condition as well, but the north and south bearing plates of bent 3 are not fully bearing on the cap. At bent 6, the north bearing plate is missing hold down nuts inside and outside and the joist hangers at span 6 were found to be insufficient.

Overall, the substructure was found to be in serious condition due to the lack of bearing support observed for the footings at bents 1A, 3, 7, 7A, 8, and 9. The remaining members of the substructure are rated as being fair to satisfactory condition. Checking and splitting was observed in the caps at bent 3, 5 and 6.

The full inspection report with condition ratings for all parts of the bridge is in Attachment A.

4. Structural Evaluation and Code Checks

The footbridge was evaluated structurally based on the measurement and condition information gathered in the field. Additionally, several aspects of the bridge detailing were compared against current code requirements.

4.1 Structural Evaluation

The structural evaluation of the key members for the footbridge was done in accordance with the following code references:

- ODOT LRFR Manual June 2018 (ODOT)
- AASHTO Manual for Bridge Evaluation, 3rd ed. 2018 with 2019 Interims (MBE)
- AASHTO LRFD Bridge Design Specifications, 9th ed. 2020 (BDS)
- AASHTO LRFD Guide Specifications for the Design of Pedestrian Bridges, December 2009 with 2015 Interim Revisions (PED)
- ODOT Highway Design Manual, 2012 (HDM)

Initially, the primary bridge members were identified for analysis and grouped into categories by member type. These groups consisted of stringers, decking, caps, railing components, timber pier posts, and concrete piers. The stringers, decking, and caps were assumed to be simply supported and the posts were assumed to have fixed connections. Net sections were used for sectional capacities. Where members of the same size were used at multiple locations on the bridge, the maximum span lengths and loadings were used for analysis to determine the controlling rating factors of each member size. Where this analysis produced a rating factor less than 1.0, additional analysis was conducted for the other spans that contain this same member size to determine all members that have deficient rating factors. This was the case for the 4x6 stringers. Analysis of the connections was beyond the scope of this work.

In the absence of as-built documents, material property assumptions were made according to the ODOT LRFR Manual, AASHTO MBE, and AASHTO BDS based on approximate age of the structure and field observations of the existing structure. Dead loads of components were based on the net dimensions of the elements visually present and densities were assumed per AASHTO BDS Table 3.5.1-1. Live loads consisted solely of pedestrian live loading of 90 psf per AASHTO PED 3.1.

Each element was rated using the load rating factor equation presented in the AASHTO MBE, section 6A.4.2.1 (see equation below) for flexural, shear, axial compression, and bearing demands, as applicable. Highway bridges are rated using inventory and operating live load factors. Per the MBE, an inventory rating is for the loading that the bridge may safely sustain over an indefinite period of time while an operating rating is for a maximum permissible load that if not limited, can shorten the life of the bridge. For pedestrian bridges, the code is silent on whether to use an inventory or operating rating. This structure was rated using the live load factor associated with an inventory rating. The resulting rating factors are included in Table 1. A rating factor below 1.0 indicates that the member is not adequate to support the evaluated load.

Load Rating Factor (RF) Equation:

$$RF = \frac{\Phi_c * \Phi_s * \Phi * R_n - \gamma_{DC} * DC_u - \gamma_{DW} * DW_u}{\gamma_{LL} * (LL_u + IM)}$$

Member		Flexure RF	Shear RF	Bearing/(Axial) RF	L/360 Live Load Deflection
Glulam Beams	Span 3	1.88	6.05	3.76	FAIL
	Span 4	15.43	18.33	9.33	PASS
	Span 5	2.43	6.95	4.49	FAIL
4x6 Stringers	Span 1	**0.57**	1.97	4.35	FAIL
	Span 2, north side	**0.58**	2.02	3.44	FAIL
	Span 2, south side	1.50	3.28	5.30	FAIL
	Span 8, north side	**0.52**	1.96	3.36	FAIL
	Span 8, south side	**0.84**	2.46	4.11	FAIL
Decking	All spans	3.15	12.46	118.72	PASS
4x6 Caps	Typical bent	45.23	57.28	118.72	N/A
	Bent 2	**0.19**	**0.71**	2.97	N/A
6x8 Caps		78.78	84.80	186.59	N/A
4x6 Posts		N/A	N/A	7.97 (Axial)	N/A
4x4 Rail Posts		**0.61**	3.52	5.01 (Axial)	N/A
2x4 Rails		**0.24**	2.38	N/A	N/A
12" Round Concrete Columns	< 2'-7"	N/A	N/A	25.53 (Axial)	N/A
	> 2'-7" (Bents 4 and 5, south side)	N/A	N/A	0 (Axial)	N/A

Table 1 – AASHTO LRFR Rating Factors for existing Firerock bridge members

A primary member group found to be structurally inadequate include the 4-inch x 6-inch nominal stringers in spans 2 and 8 for flexural demands, with rating factors below 1.0.

Located at bents 2 and 7 are 4-inch x 6-inch "straddle bents" where the bent caps extend outward from the centerline of the stringer members, thus supporting loads in flexure and shear rather than direct bearing load transfer to the columns below. Bent 2 was analyzed as the controlling of these two locations, by inspection, and was found to have both a flexural rating factor and shear rating factor significantly less than 1.0.

Lastly, the flexural capacities of the 4-inch x 4-inch nominal railing posts and 2-inch x 4-inch nominal longitudinal rail members were found to be significantly less than 1.0.

The round concrete columns were evaluated for axial load and bearing assuming unreinforced concrete. For axial loads and bearing, the unreinforced columns are not adequate at the south side of bents 4 and 5.

4.2 Bridge Detailing Code Requirements

The railing geometry was compared to code requirements in AASHTO BDS 13.8.1. The longitudinal railing members are less than the minimum required height of 42.0-inches above top of walkway. Additionally, the sizes of the openings between longitudinal railing members are significantly greater than the maximum clear opening requirements of 6.0-inches for the lower 27.0-inches of railing and 8.0-inches for the upper portion. Finally, the bridge lacks the required safety toe rail or curb.

The site investigation also noted gaps between deck planks varying up to about 2-inches, which exceeds the ¼-inch maximum spacing for seasoned material according to the ODOT 2021 Standard Specifications section 00570.44 Decking which corresponds to Americans with Disabilities Act (ADA) requirements.

With the walkway width varying between 3-feet, 4-inches and 3-feet, 5 1/2-inches, it does not meet the minimum passage requirement for ADA of 4-feet per the HDM, Appendix L.

5. Conclusion and Recommendations

The Firerock Footbridge is between 40 and 50 years old, an advanced age for this type of timber structure which does not have wood that has been pressure-treated with preservatives. The bridge structural evaluation shows that multiple key members of the existing bridge are structurally deficient and have rating factors significantly below 1.0. The deficient members include the main stringers in three of the eight spans, the cap beam at bent 2, the concrete piers at the south side of bents 4 and 5, and all bridge railing members. The site evaluation noted that the footings are in poor condition and lack bearing support. The concrete piers were not scanned for reinforcing (rebar) as a part of the evaluation. Based on the outcome of the site evaluation, the presence of rebar would not change Jacobs' recommendations.

Additionally, the bridge does not meet current design code detailing requirements for the decking, railing, and multiple superstructure members including the opening size between the railing members, height of railing members, spacing between deck planks, and the walkway width.

Complete reconstruction with revised details is required to provide a pedestrian crossing that meets current code requirements. Reuse of the wood glulam beams is not likely to be cost effective due to the design constraints that would be imposed on the reconstructed bridge and the remaining service life of wood beams.

Based on the results of the site visit, structural evaluation, and comparison with code detailing requirements, Jacobs recommends the bridge remain closed. Use of the existing bridge to cross the Deschutes River poses a safety risk. For the reasons set forth above, reconstruction using the existing bridge components is not likely to be cost effective. Complete replacement of the bridge is recommended if a pedestrian crossing is desired at this location. If replacement of the bridge is something the City chooses to consider, Jacobs recommends the City first explore the feasibility of providing an ADA accessible bridge and acquisition of permanent public easements on the west side of the river to provide a connection to the Deschutes River Trail.

6. Attachments

- Attachment A – Firerock Footbridge Inspection Report with Photos
- Attachment B – Structural Analysis
- Attachment C – Firerock Bridge East Stair Evaluation Site Visit Memo *(Appended to document on November 15, 2023)*

Attachment A
Firerock Footbridge Inspection Report with Photos

Jacobs

BRIDGE NO.
HWY NO.

INSP. FREQ. 24 mo. MILE POST
INSPECTOR Dale Wilson
SIGNATURES Dale Wilson
SIGNATURES

BRIDGE TYPE 702 NAME Firerock Footbridge
CROSSING (OVER, UNDER) Deschutes River COUNTY Deschutes

DISTRICT ____ YEAR BUILT
A.C.(in.) 0 DATE 4/29/21

Bridge Inspection Report

AR = As Repaired
OM = Original Member

SUBSTRUCTURE (60)		Condition Rating AR	Condition Rating OM
1. END BENTS	Cap / Sill		5
	Piles		
	Footings		
	Footing Piles		
	Backwalls, Bulkheads		
	Wings		
2. INTERIOR PIERS OR BENTS	Caps		5
	Column, Posts		6
	Footings		3
	Footing Piles		
	Piles		
	Bracing		6
3. Debris on Seats			7
4. Paint			
5. Collision Damage			8
6. Scour			5
7. Settlement (Footing or Piling)			
INSPECTOR'S CONDITION RATING (60)			**3**

CHANNEL & CHAN. PROTECT (61)	
1. Channel Scour	7
2. Embankment Erosion	7
3. Drift	6
4. Vegetation	6
5. Channel Change	8
6. Fender System	
7. Spur Dikes & Jetties	
8. Riprap	
9. Adequacy of Opening	7
INSPECTOR'S CONDITION RATING (61)	**7**

OBSERVATIONS

SUPERSTRUCTURE (59)		Condition Rating AR	Condition Rating OM
1. Stringers			7
2. Girder or Beams			7
3. Floor beams			
4. TRUSSES	Chords		
	Web Members		
	Portals		
	Bracing		
5. Diaphragms, Bridging			8
6. Bearing Devices			7
7. Paint			
8. Rivets or Bolts			7
9. Welds			
10. Collision Damage			8
11. Deflection under Load			7
12. Alignment of Members			7
13. Vibrations under Load			7
14. Machinery (Movable Spans)			
INSPECTOR'S CONDITION RATING (59)			**7**

DECK (58)	Condition Rating AR	Condition Rating OM
1. Deck - Structural Condition		5
2. Wearing Surface		5
3. Deck Joints		
4. Curbs, Felloe Guards		
5. Sidewalks		
6. Parapet, Concrete Barrier		
7. Railing, Posts		5
8. Median Barrier, Railing		
9. Paint		
10. Drains		
11. Lighting Standards		
12. Utilities - Abandoned		
13. Vibrations in Deck		6
INSPECTOR'S CONDITION RATING (58)		**5**
APPROACH CONDITION (65)		
1. Pavement & Embankment		6
2. Shoulder Embankment		6
3. Relief Joints		
4. Approach Slab		
5. Guardrail		
INSPECTOR'S CONDITION RATING (65)		**6**
SAFETY FEATURES (36)	0000	
APPR. ALINE (72)		4
SIGNING		
1. Posted Loading		
2. Legibility		8
3. Visibility		8
INSPECTOR'S CONDITION RATING		**8**

REMARKS (Key-in to item and number above)

See attached sheets for detailed inspection report.

Jacobs

BRIDGE NO.
HWY NO.

INSP. FREQ. 24 mo. MILE POST
INSPECTOR Dale Wilson
SIGNATURES Dale Wilson
SIGNATURES ____________________

BRIDGE TYPE 702 NAME Firerock Footbridge
CROSSING (OVER, UNDER) Deschutes River COUNTY Deschutes
DISTRICT ____ YEAR BUILT
A.C.(in.) 0 DATE 4/29/21

Bridge Inspection Report

AR = As Repaired
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	Footings		
	Footing Piles		
	Backwalls, Bulkheads		
	Wings		
2. INTERIOR PIERS OR BENTS	Caps		5
	Column, Posts		6
	Footings		3
	Footing Piles		
	Piles		
	Bracing		6
3. Debris on Seats			7
4. Paint			
5. Collision Damage			8
6. Scour			5
7. Settlement (Footing or Piling)			
INSPECTOR'S CONDITION RATING (60)			**3**
CHANNEL & CHAN. PROTECT (61)			
1. Channel Scour			7
2. Embankment Erosion			7
3. Drift			6
4. Vegetation			6
5. Channel Change			8
6. Fender System			
7. Spur Dikes & Jetties			
8. Riprap			
9. Adequacy of Opening			7
INSPECTOR'S CONDITION RATING (61)			**7**

OBSERVATIONS SUPERSTRUCTURE (59)		Condition Rating AR	Condition Rating OM
1. Stringers			7
2. Girder or Beams			7
3. Floor beams			
4. TRUSSES	Chords		
	Web Members		
	Portals		
	Bracing		
5. Diaphragms, Bridging			8
6. Bearing Devices			7
7. Paint			
8. Rivets or Bolts			7
9. Welds			
10. Collision Damage			8
11. Deflection under Load			7
12. Alignment of Members			7
13. Vibrations under Load			7
14. Machinery (Movable Spans)			
INSPECTOR'S CONDITION RATING (59)			**7**

DECK (58)	Condition Rating AR	Condition Rating OM
1. Deck - Structural Condition		5
2. Wearing Surface		5
3. Deck Joints		
4. Curbs, Felloe Guards		
5. Sidewalks		
6. Parapet, Concrete Barrier		
7. Railing, Posts		5
8. Median Barrier, Railing		
9. Paint		
10. Drains		
11. Lighting Standards		
12. Utilities - Abandoned		
13. Vibrations in Deck		6
INSPECTOR'S CONDITION RATING (58		**5**
APPROACH CONDITION (65)		
1. Pavement & Embankment		6
2. Shoulder Embankment		6
3. Relief Joints		
4. Approach Slab		
5. Guardrail		
INSPECTOR'S CONDITION RATING (65		
SAFETY FEATURES (36)	0000	
APPR. ALINE (72)		4
SIGNING		
1. Posted Loading		
2. Legibility		8
3. Visibility		8
INSPECTOR'S CONDITION RATING		**8**

REMARKS (Key-in to item and number above)

See attached sheets for detailed inspection report.

Jacobs

BRIDGE INSPECTION REMARKS

BRIDGE NO.

HWY NO.

BRIDGE TYPE 702 | NAME Firerock Footbridge | INSP. FREQ. 24 mo. MILE POST
CROSSING (OVER, UNDER) Deschutes River | COUNTY Deschutes | INSPECTOR Dale Wilson
DISTRICT | YEAR BUILT | A.C.(in.) 0" | DATE 4/24/21 | SIGNATURES Dale Wilson

58 (DECK)

Timber deck planking is 48" in length and 43" from outside of stringer to outside of stringer. Deck planking sizes are 2"x12" in spans 3 through 5 and 2"x6" in spans 1 through 2 and 6 through 8 with 2"x4"'s used as replacement boards. Flashing placed between decking and glulam beams.

All timber decking has nails popping / lifting out of decking.

Moss growth on the ends of Span 5 decking.

Deck planks checking, splitting, some rotting, some section loss, and wide spacing Spans 1 through 8. Decking boards not full width of bridge and Bent 2 and missing boards in span 8.

Span 7 north end deck section built over the top of the Span 8 west end.

59 (SUPERSTRUCTURE)

Checking in Span 1 and 2 north stringer.

Glulam beams appear structurally sound but have some localized section loss at knots in wood.

Span 1 southside stringer is checking.

Bent 3 north and south bearing plates not fully bearing on cap.

Bent 6 north bearing plate missing hold down nuts inside and outside.

Span 6 joist hangers insufficient.

60 (SUBSTRUCTURE)

Bent 1 and 9 Cap / Sill supports are buried.

Bent 1A northside rail posts extend to pier block and post is loose causing pier block to wobble when pushing on the rail.

Bent 1A southside rail post extends down to ground with rocks piled at the base of the post (no foundation)

4"x6' pipe support has section loss near north post at Bent 2.

Bent 3 concrete columns drilled into rock. Open space beneath rock on south side.

Bent 3 cap checking and splitting.

Bent 5 cap has vertical through split with section loss at southside and checking / split on the northside.

Bent 6 cap checking at the ends.

Bent 7 northside post floating in mid air and southside post is founded on 12" concrete column on large rock.

Bent 7A post founded on rock and drilled in but not fully bearing. Southside rail post doesn't extend to ground.

Bent 8 northside post is floating and not bearing.

Bent 9 posts not bearing and loose.

61 (CHANNEL)

Debris hung up in rocks and around columns at Bents 4 and 5.

65 (APPROACH)

East approach consists of stairs and not inspected.

West approach is narrow.

OTHER

Rail posts tops are deteriorating with section loss and rails are notched into rail posts.

Span 1 west 2"x4" rail posts connected to deck only.

Waterline under bridge broken and abandoned.

BRIDGE MAINTENANCE RECOMMENDATIONS

Jacobs

BRIDGE NO.

HWY NO.

BRIDGE TYPE 702 NAME Firerock Footbridge
INSP. FREQ. 24 mo. MILE POST
CROSSING (OVER, UNDER) Deschutes River COUNTY Deschutes
INSPECTOR Dale Wilson
DISTRICT YEAR BUILT A.C.(in.) 0" DATE 4/24/21
SIGNATURES Dale Wilson

DECK (58)

Replace deck planks with splitting, rotting, and section loss and replace missing planks or planks that don't extend full width of bridge.

Respace deck planking to 1/4" spacing and reattach with lag screws.

Remove moss from ends of deck planks.

Rebuild bridge section that has overlapping section.

SUPERSTRUCTURE (59)

Reset Bent 3 bearing plate to provide full bearing.

Install nuts on Bent 6 northside bearing plate bolts.

Install new joist hangers at Span 6 connection to Span 5 glulam beams.

SUBSTRUCTURE (60)

Reconstruct concrete columns to be founded on bedrock or socketed into the ground at Bents 3 through 7.

Reconstruct intermediate Bents 1A and 7A to be fully bearing on ground and stable.

Monitor checking Bents 3 and 6 caps.

Replace Bent 3 and 5 caps.

APPROACHES (65)

Modify approaches to bring up to ADA specifications.

OTHER

Replace all rail posts and railing to update to current specifications.

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East Approach Looking West

West Approach Looking East

South Elevation Span 1 thru 2

North Elevation Span 3 thru 5

North Elevation Span 6 thru 8

Span 1 West Rail Post Connection

Deck Planking Checking and Splitting

Bent 3 Concrete Column on Rock w/Void Underneath

Bent 3 Bearing Plate Not Fully Bearing

Full Depth Split in Bent 3 Cap

Deck Planking Not Full Width at Span 1 East End

Deck Planking Splitting and Checking

Bent 5 Cap Split at South End

Bent 6 North Bearing Plate Missing Nut

Span 6 Westside Joist Hanger

Bent 7 Post Floating

North Post Not Fully Bearing at Bent 7A

Overlapping Decking at Span 7 to Span 8

Missing Decking at Span 8 West End

Railing Notched into Railposts

Top of Railpost Section Loss

Firerock Footbridge - Railing Geometry

Northside Railing

Post #	Post Spacing (in)	Top of Top Rail from Top of Deck (in)	Top of Intermediate Rail from Top of Deck (in)	Top of Lower Rail from Top of Deck (in)	Notes:
1		39	27	13	
2	49	38 1/4	26 3/4	13	
3	68.75	39 1/2	27 1/4	13 3/8	
4	31	32	18	---	
5	84	37 3/4	20 1/4	11 1/2	
6	59.75	37 1/4	20	9 1/2	
7	96	37 1/4	19 3/4	10 1/4	
8	96	37	19 3/4	10 1/4	
9	95.25	37 3/8	19 7/8	10 3/8	
10	96.5	37 5/8	20 1/4	10	
11	96	37 3/4	20 1/2	10 1/2	
12	93	37 7/8	20 1/2	11 1/4	8" to angle + 23" to post
13	96.5	37 3/4	19 5/8	11	
14	96	37	19 5/8	9 1/2	
15	95.5	37	19 1/4	10 1/2	
16	96	37 3/8	20 1/4	10 1/4	
17	91	37 3/4	20 1/4	10 3/4	
18	76	38	20 5/8	11	
19	72	37 1/2	20	10 3/8	
20	54	37 1/2	20	---	
21	51	37 7/8	20 1/2	---	
22	35	37 3/4	20 1/4	---	2"x4" Sitting on Deck

Firerock Footbridge - Railing Geometry

Southside Railing

Post #	Post Spacing (in)	Top of Top Rail from Top of Deck (in)	Top of Intermediate Rail from Top of Deck (in)	Top of Lower Rail from Top of Deck (in)	Notes:
1		32	15	---	
2	43	32 1/2	15 1/2	---	58" to angle + 3" to post
3	61	35 1/2	18 1/2	---	
4	105	38	32, 20 1/2, 18	10 3/4	
5	84	37	19 1/2	9 3/4	
6	94.75	37	19 3/4	10 1/4	
7	96.75	37 1/4	20	10	
8	97.25	37	19 1/2	9 3/4	
9	96	37 1/4	20	10 1/4	
10	94.75	38	20 3/4	10 3/4	
11	93	37 1/4	19 3/4	10 1/4	
12	96.5	37 3/4	20	10 1/4	
13	96	37	19 3/4	9 3/4	
14	96.5	37 3/4	20 3/8	10 1/2	
15	96.5	37 3/4	20 1/4	10 3/4	
16	91.5	38	19 1/2	10 3/4	
17	76.75	37 3/4	20 1/4	10 5/8	
18	48.5	37 3/4	20 1/4	11	
19	12	37 7/8	20 5/8	---	
20	75.5	37 7/8	20 5/8	---	
21	49.75	37 3/4	20 1/4	---	2"x4" Sitting on Deck

Notes: **All Posts 4"x4" nominally, except when noted.**
All Rails are 2"x4" nominally.
Post Spacing from Center to Center of Post.
Post Numbering Eastside to Westside.

Attachment B
Structural Analysis

D3380200

Design of CSEP - Rimrock Pump Stations Improvements Project

Firerock Footbridge Condition Assessment

Prepared for
City of Bend

Firerock Footbridge Condition Assessment Analysis

Prepared by: Nikolas Gordon, PE

June 2021

Jacobs

377 SW Century Drive, Suite 201
Bend, Oregon, 97702

INDEX TO DESIGN CALCULATIONS
Firerock Footbridge Condition Assessment

Analysis Summary

<u>Overview</u>

The Firerock Footbridge is an eight span, 143-foot long, 4-foot wide timber pedestrian bridge crossing the Deschutes River. The structure consists of two lines of stringers (4x6 timber, 10.75”x5” glulam, and 21.75”x5” glulam members) spaced at 3’-7” out-to-out supporting 2x decking plank materials.

The 4x6 stringers are supported by hanger-brackets attached to the side faces of 4x6 bent caps, or the end faces of the 21.75”x5” glulam stringers. The 4x6 stringers are assumed continuous through bents 1A, 7A, and 8 where they are supported by the 4x4 railing posts via bolted connections. The 10.75”x5” glulam stringers are supported by hanger-brackets attached to the end faces of the 21.75”x5” glulam stringers. The 21.75”x5” glulam stringers are supported by a 6x8 bent cap.

The 4x6 caps are supported by 4x6 timber posts, or bear directly on earth at the abutments. The 6x8 cap is supported by two 12”-diameter concrete piers centered beneath the two lines of glulam stringers above.

The pedestrian railing consists of 4x4 timber posts with two or three lines of 2x4 railing.

<u>Objective</u>

The objective of this evaluation is to determine the structural capacity of the existing structure, in an as-is condition, to serve as a viable pedestrian crossing to support both pedestrian live load and the dead load of the existing structure. The results are provided in the format of load rating values.

The structure is broken down into primary representative structural members, and then grouped by similar loading configurations; see breakdown below. Assumptions on material properties and evaluations are per ODOT LRFR Manual dated June of 2018, AASTHO Manual for Bridge Evaluation 3rd edition dated 2018 with 2019 Interim Revisions, AASHTO LRFD Guide Specifications for the Design of Pedestrian Bridges dated 2009 with 2015 Interim Revisions, and AASHTO LRFD Bridge Design Specifications 9th edition dated 2020.

Analysis Groups

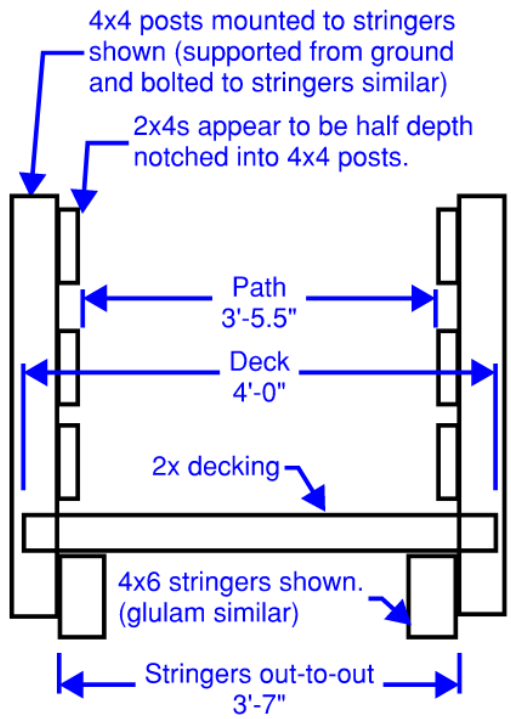
- Stringers
    1) 5″x21.75″ glulam beam, 43.0′ span (Span 3)
    2) 5″x10.75″ glulam beam, 8.45′ span (Span 4)
    3) 4x6 stringers (various lengths; see below)
        3a) 13.0′ continuous 2-span (Span 1)
        3b) 12.4′ span (Span 8 North stringer)
        3c) 10.2′ span (span 8 South stringer)
        3d) 12.2′ span (Span 2 North stringer)
        3e) 8.0′ span (Span 2 South stringer)
    4) 5″x21.75″ glulam beam, 38.1′ span (Span 5)
- Decking
    1) 2x12 decking planks
    2) 2x6 decking planks
    3) 2x4 decking planks
- Caps
    1) 4x6 cap (3′-7″ span)
    2) 4x6 cap (Bent 2)
    3) 6x8 cap (bearing only)
- Railing members
    1) 4x4 rail post
    2) 2x4 Longitudinal rail members
        2a) X-axis (vertical loading along narrow face)
        2b) Y-axis (horizontal loading along wide face)
- Pier Posts
    1) 4x6 post, Bent 2
    2) 4x4 post, Bent 1A
- Concrete Piers
    1) 12″ round concrete columns

In the absence of a more in depth/invasive site evaluation, nor as-built documents, the analysis of the connections is beyond the scope and capacity of this evaluation.

The structure includes several skewed bents along its alignment. However, for the purposes of this analysis, all elements are evaluated member by member where the skew of the bridge is only considered for determining the lengths of individual members.

Major assumptions include the approximate heights of timber posts and concrete piers, as these were not measured in the field, or were inaccessible. The heights of the timber posts and concrete piers were scaled from known geometries in site visit photos, and are therefore approximated. There is no positive connection assumed between cap beams and concrete piles at Bents 3–6; there is no positive connection assumed between timber posts and concrete foundations at north side of Bent 1A; there is no positive connection assumed between timber post and native ground at south side of Bent 1A, north side of Bent 7, Bent 7A, or Bent 8. Timber posts at Bent 2 and south side of Bent 7 appear to be embedded in concrete piles and are evaluated a “fixed” at their base, however Bent 2 timber post controls analysis by inspection (i.e. taller, greater demand) and is analyzed conservatively representatively for Bent 7. Glulam stringers appear centered over concrete piers with negligible eccentricity.

The load combination of Strength I, per AASHTO MBE Table 6A.4.2.2-1 is used to determine demand forces. Per geometry of deck, no pattern of live loading is assumed to produce overturning forces on bents. See typical cross section below.



Typical deck section

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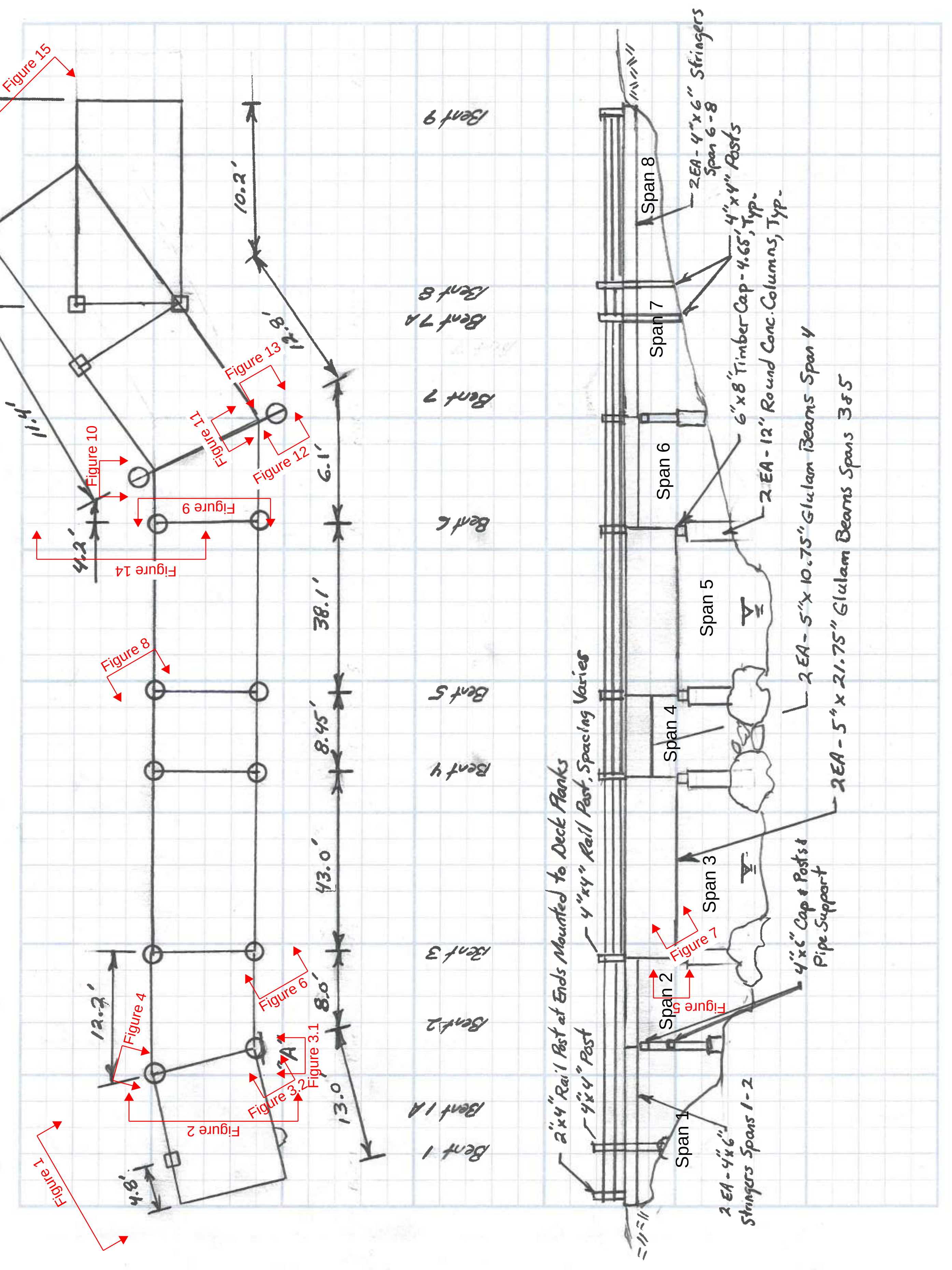
Subject *Firerock Footbridge* Project

Sheet No. *1* of *2*

Authored by Date Checked by Date *4-29-21*

Project Sketches



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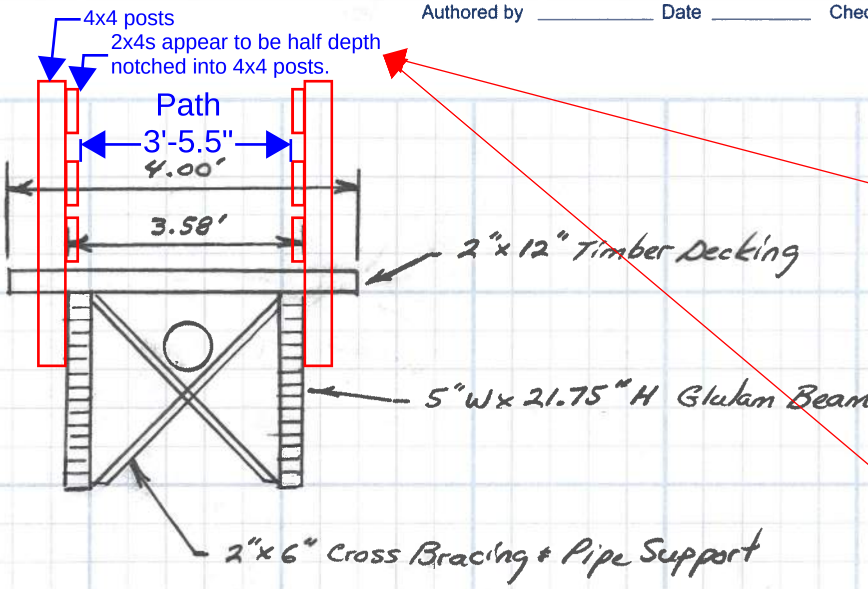
Subject: Firerock Footbridge | Project: Project Sketches

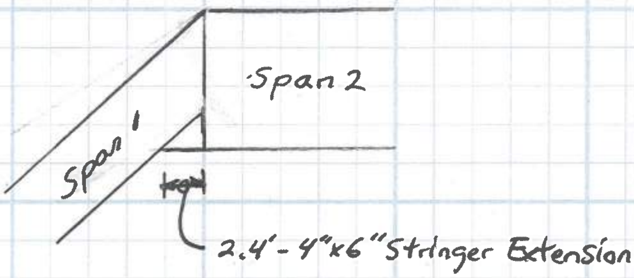
Sheet No. 2 of 2

Authored by ______ Date ______ Checked by ______ Date 4-29-21



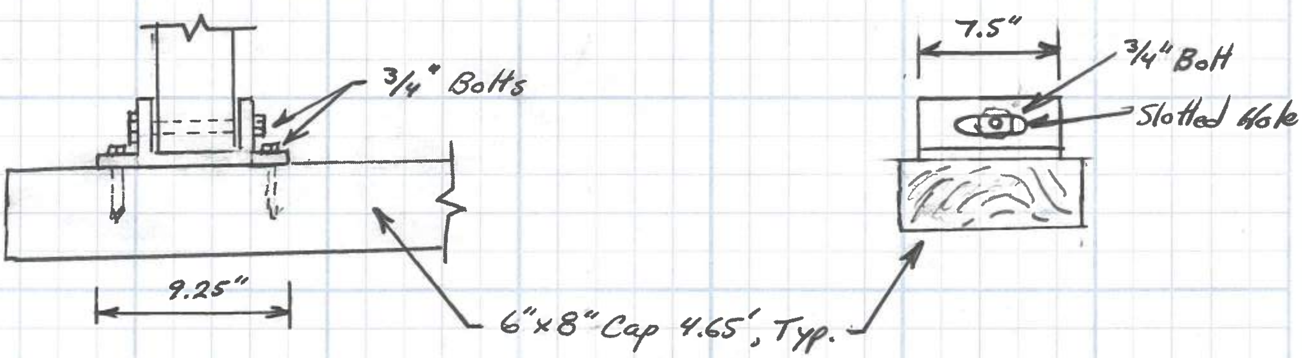
Main Span Cross Section

No Scale



Detail "A"

No Scale



Bearing Plate

No Scale

Reference Photos

Figure 1 - View of bridge looking East

Figure 2 - View of bridge looking East

Figure 3.1 - View of Bent 2 looking North

Figure 3.2 - View of underside of South side of Bent 2

Figure 4 - View of North side Bent 2 looking South

Figure 5 - View of Bent 3 looking North East

Figure 6 - View of Bent 3 looking East

Figure 7 - View of Span 3 underside looking East

Reference Photos

Figure 8 - View of Bent 5 looking South West

Figure 9 - View from Bent 6 looking West

Figure 10 - View of Bent 7 looking South

Figure 11 - View of Bent 7 looking South Post looking South

Figure 12 - View of Bent 7 Post looking East

Figure 13 - View of Bent 7 Post looking West

Figure 14 - View from Bent 6 looking East

Figure 15 - View of bridge looking south west

Bridge Type: Timber and Glulam pedestrian bidge.simple supports for dead load and live load.

GIRDER/STRINGER ANALYSIS:

References:
AASHTO LRFD BDS 9th Ed. 2020 (AASHTO)
AASHTO MBE 3rd E. 2018 with 2019 Interims (MBE)
AASTHO LRFD Guide Spec for the Design of Pedestrian Bridges 2009 w/ 2015 Interims (PED)
ODOT LRFR Manual June 2018 (ODOT)
ODOT Bridge Design Manual (BDM)

LRFR Strength Limit State:

$$RF = \frac{\phi_c \cdot \phi_s \cdot \phi \cdot (R_n) - (\gamma_{DC}) \cdot (DC) - (\gamma_{DW}) \cdot (DW)}{(\gamma_L) \cdot (LL + IM)}$$ (MBE 6A.4.2.1-1)

Resistance Factors:

$\phi_s := 0.75$	LRFD resistance factor for shear	(AASHTO 8.5.2.2)
$\phi_f := 0.85$	LRFD resistance factor for flexure	(AASHTO 8.5.2.2)
$\phi_{cp} := 0.90$	LRFD resistance factor for compression perpendicular to grain.	(AASHTO 8.5.2.2)
$\phi_{cl} := 0.90$	LRFD resistance factor for compression parallel to grain.	(AASHTO 8.5.2.2)
$\phi_c := 1.00$	Condition factor for superstructure condition rating = 7 (Good)	(MBE T. 6A.4.2.3-1)
$\phi_{sf} := 1.00$	System Factor for Flexure, structure type: "Timber Stringers"	(ODOT 1.4.1.4)
$\phi_{sv} := 1.00$	System Factor for Shear, structure type: "Timber Stringers"	(ODOT 1.4.1.4)
$\phi_{sa} := 1.00$	System Factor for Axial, All other girder bridges and slab bridges	(MBE 6A.4.2.4-1)

Combined Resistance Factors:

For Flexure:	$\Phi_f := \phi_f \cdot (\max(\phi_c \cdot \phi_{sf}, 0.85))$	$\Phi_f = 0.850$
For Shear:	$\Phi_v := \phi_s \cdot (\max(\phi_c \cdot \phi_{sv}, 0.85))$	$\Phi_v = 0.750$
For Axial:	$\Phi_a := \phi_{cl} \cdot (\max(\phi_c \cdot \phi_{sa}, 0.85))$	$\Phi_a = 0.900$
For Bearing	$\Phi_b := \phi_{cp} \cdot (\max(\phi_c \cdot \phi_{sa}, 0.85))$	$\Phi_b = 0.900$

(Note: $\phi_c \phi_s \geq 0.85$ per MBE 6A.4.2.1-3)

Load Factors:

Dead Load Factors γ_{DC}:

$\gamma_{DC.max} := 1.25$	max.	MBE T. 6A.4.2.2-1 for structural components and attachments STR I
$\gamma_{DC.min} := 0.90$	min.	AASHTO T. 3.4.1-2 for structural components and attachments STR I

Live Load Factors γ_L:

$\gamma_{LL} := 1.750$ MBE T. 6A.4.2.2-1, assume pedestrian loading as Inventory

Dynamic load allowance, IM, is not required with pedestrian loading, PED 3.1

Bridge Members

Analysis below shall consist of stringer members:

1) 5x21.75 glulam beams, 43.0' span (Span 3)
2) 5x10.75 glulam beams, 8.45' span (Span 4)
3) 4x6 stringers,
   - 3a) 13.0' continuous 2-span (Span 1)
   - 3b) 12.4' span (Span 8 North)
   - 3c) 10.2' span (Span 8 South)
   - 3d) 12.2' span (Span 2 North)
   - 3e) 8.0' span (Span 2 South)
4) 5x21.75 glulam beams, 38.1' span (Span 5)

Notes:
1) Multiple 4x6 stringer span lengths will be evaluated to determine which spans are structurally adequate.

2) Span 1 is 13.' in length with an intermediate support at 4.8' from bent 1. The negative moment of a 2 span continuous beam shall be evaluated. Positive moment would include pattern pedestrian loading on the 8.2' span, counteracted by DC the full length. Assume the 8.0' span in 3d) is similar and only the negative moment shall be calculated for 3a).

Bridge Geometry:

Number of Girders: $N_g := 2$

Deck Out to Out: $W_{total} := 4ft + 0in = 4.00\,ft$

$S_{s.oto} := 3 \cdot ft + 7 \cdot in$ Spacing stringers, out to out dimension

Path Width: $W_{path} := S_{s.oto} - 0.5 \cdot 2 \cdot (1.5 \cdot in) = 3.46\,ft$

Rail posts are mounted to outside of stringers. 2x4 railing are notched half depth into posts. Path width is between railings.

Span: $L_{span} := (43 \quad 8.45 \quad 13 \quad 12.4 \quad 10.2 \quad 12.2 \quad 8 \quad 38.1)^T \cdot ft$

$i := 1 .. \text{length}(L_{span})$ Set counter

Girder Height: $h_g := (21.75 \quad 10.75 \quad 5.5 \quad 5.5 \quad 5.5 \quad 5.5 \quad 5.5 \quad 21.75)^T\, in$

Girder Width: $b_g := (5 \quad 5 \quad 3.5 \quad 3.5 \quad 3.5 \quad 3.5 \quad 3.5 \quad 5)^T\, in$

Length of bearing: $L_b := (5.5 \quad 2 \quad 2 \quad 2 \quad 2 \quad 2 \quad 2 \quad 5.5)^T \cdot in$

21.75" glulam sits on 4x6 timber cap.
10.75" glulam sits on 2" wide brackets attached to end of 21.75" glulam
4x6 stringer sits on 2" wide brackets attached to end of 21.75" glulam or side of 4x6 caps.

Girder Spacing: $S_g := \left(S_{s.oto} - b_g\right)^T = (3.17 \quad 3.17 \quad 3.29 \quad 3.29 \quad 3.29 \quad 3.29 \quad 3.29 \quad 3.17)\, ft$

Deck Thickness: $t_f := 1.5in$ 2"x12" / 2"x6" / 2"x4" timber decking

Material Properties:

$w_t := 0.050\text{kcf}$ (AASHTO Table 3.5.1-1) (Assumed timber species of douglas fir is softwood)

Glulam (G): (24F Douglas Fir, Assumed symbol is V4) Bending about X-X axis, per ODOT 8.2.4

$F_{bxo} := 2.4\text{ksi}$ (ODOT 8.2.4) Bending stress

$F_{vxo} := 0.265\text{ksi}$ (ODOT 8.2.4) Shear parallel to grain

$F_{epo} := 0.650\text{ksi}$ (ODOT 8.2.4) Compression perpendicular to grain

$E_{xo} := 1800\text{ksi}$ (ODOT 8.2.4) Mod. of Elasticity

Dimensional Lumber (L) (Douglas Fir, Dimension ≥ 2 in. wide, Select Structural), per ODOT 8.2.4

"Select Structural" grade assumed here as values provided in ODOT LRFR match AASHTO Select Structural

$F_{bo} := 1.5\text{ksi}$ (ODOT 8.2.4) Bending stress

$F_{vo} := 0.180\text{ksi}$ (ODOT 8.2.4) Shear parallel to grain

$F_{cpo} := 0.625\text{ksi}$ (ODOT 8.2.4) Compression perpendicular to grain

$F_{co} := 1.700\text{ksi}$ (AASHTO Table 8.4.1.1.4-1) Compression parallel to grain

$E_o := 1900\text{ksi}$ (ODOT 8.2.4) Mod. of Elasticity

Adjustment Factors (AASHTO 8.4.4)

$C_M := 1.0$ (Wet service factor for Glu-Lam less than 16%, and sawn lumber less than 19%, ODOT 8.2.4.3). Unless submerged, timber is considered dry (BDM 1.8.2)

$C_F := 1.0$ Size factor, (ODOT 8.2.4.4)

$j := 1..2$

$$C_{V_i} := \mathrm{Min}\left[1, \left[\left(\frac{12in}{h_{g_i}}\right)\cdot\left(\frac{5.125in}{b_{g_i}}\right)\cdot\left(\frac{21ft}{L_{span_i}}\right)\right]^{0.1}\right]$$

Volume factor, Glulam, ODOT 8.2.4.5

Note: When depth ≤ 12.0 in, or length ≤ 21.0 ft, C.V = 1.0

$$C_V := \left(C_{V_1}\ \ C_{V_2}\ \ C_{V_8}\right)^T \qquad C_V^T = (0.88\ \ 1.00\ \ 0.89)$$

$C_{fu} := 1.0$ (Flat-use factor, ODOT 8.2.4.6)

$C_{i.E} := 0.95$ (Incising factor, for E_o, ODOT 8.2.4.6)

$C_i := 0.80$ Incising factor, for F.bo and F.von (ODOT 8.2.4.7), & F.to and F.co (AASHTO T. 8.4.4.7-1)

$C_{i.cpo} := 1.0$ Incising factor, for F_{cpo} (ODOT 8.2.4.7)

$C_d := 1.0$ Deck factor (ODOT 8.2.4.8)

$C_{\lambda.1} := 0.8$ Time effect factor, Strength Limit State 1 (ODOT 8.2.4.9)

$E_G := (E_{xo})\cdot(C_M)\cdot(C_{i.E}) = 1710\cdot ksi$ (Glulam) (AASHTO 8.4.4.1-6)

$E_L := (E_o)\cdot(C_M)\cdot(C_{i.E}) = 1805\cdot ksi$ (Other) (ODOT 8.2.4.1)

$C_{KF.f.s} := \frac{2.5}{\phi_f} = 2.94$ Format conversion factor, F.b & F.v (ODOT 8.2.4.2)

$C_{KF.cp} := \frac{2.1}{\phi_{cp}} = 2.33$ Format conversion factor, F.cp (compression perpendicular to grain.) (ODOT 8.2.4.2)

$$F_{b.ref} := \left(F_{bxo}\ \ F_{bxo}\ \ F_{bo}\ \ F_{bo}\ \ F_{bo}\ \ F_{bo}\ \ F_{bo}\ \ F_{bxo}\right)^T$$

$$C_{V.F.ref} := \left(C_{V_1}\ \ C_{V_2}\ \ C_F\ \ C_F\ \ C_F\ \ C_F\ \ C_F\ \ C_{V_3}\right)^T$$

$$C_{i.ref} := \left(1.0\ \ 1.0\ \ C_i\ \ C_i\ \ C_i\ \ C_i\ \ C_i\ \ 1.0\right)^T$$

$$F_{vo.ref} := \left(F_{vxo}\ \ F_{vxo}\ \ F_{vo}\ \ F_{vo}\ \ F_{vo}\ \ F_{vo}\ \ F_{vo}\ \ F_{vxo}\right)^T$$

$$F_{co.ref} := \left(F_{epo}\ \ F_{epo}\ \ F_{cpo}\ \ F_{cpo}\ \ F_{cpo}\ \ F_{cpo}\ \ F_{cpo}\ \ F_{epo}\right)^T$$

$$C_{i.cpo.ref} := \left(1.0\ \ 1.0\ \ C_i\ \ C_i\ \ C_i\ \ C_i\ \ C_i\ \ 1.0\right)^T$$

$$E_{ref} := \left(E_G\ \ E_G\ \ E_L\ \ E_L\ \ E_L\ \ E_L\ \ E_L\ \ E_G\right)^T$$

$$F_{b_i} := \left(F_{b.ref_i}\right)\cdot\left(C_{KF.f.s}\right)\cdot\left(C_M\right)\cdot\left(C_{V.F.ref_i}\right)\cdot\left(C_{fu}\right)\cdot\left(C_{i.ref_i}\right)\cdot\left(C_d\right)\cdot C_{\lambda.1}$$ (ODOT 8.2.4.1)

$$F_b^T = (4.97\ \ 5.65\ \ 2.82\ \ 2.82\ \ 2.82\ \ 2.82\ \ 2.82\ \ 5.03)\cdot ksi$$

$$F_{v_i} := \left(F_{vo.ref_i}\right)\cdot\left(C_{KF.f.s}\right)\cdot\left(C_M\right)\cdot\left(C_{i.ref_i}\right)\cdot\left(C_{\lambda.1}\right)$$ (ODOT 8.2.4.1)

$$F_v^T = (0.62\ \ 0.62\ \ 0.34\ \ 0.34\ \ 0.34\ \ 0.34\ \ 0.34\ \ 0.62)\cdot ksi$$

$F_c := (F_{co})\cdot(C_{KF.f.s})\cdot(C_M)\cdot(C_F)\cdot(C_i)\cdot(C_{\lambda.1}) = 3.20\cdot ksi$ (AASHTO 8.4.4.1-4)

$F_{cp_i} := (F_{co.ref_i})\cdot(C_{KF.cp})\cdot(C_M)(C_{i.cpo.ref_i})(C_{\lambda.1})$ (ODOT 8.2.4.1)

$F_{cp}^T = (1.21 \quad 1.21 \quad 0.93 \quad 0.93 \quad 0.93 \quad 0.93 \quad 0.93 \quad 1.21)\cdot ksi$

Beam stability factor, C_L calculated below. F_b will be adjusted to account for C_L. Calculations below are based on AASHTO 8.6.2 / ODOT 8.2.4.10.

Braced := "No" — "Yes" if compression side of beam is continuously braced and beam is braced laterally at supports, Else "No".

$K_{bE.G} := 1.10$ (Euler buckling coefficient for glulam)

$K_{bE.L} := 0.76$ (Euler buckling coefficient for visually graded lumber)

$K_{bE.ref} := (K_{bE.G} \quad K_{bE.G} \quad K_{bE.L} \quad K_{bE.L} \quad K_{bE.L} \quad K_{bE.L} \quad K_{bE.L} \quad K_{bE.G})^T$

$L_u := L_{span}$ Span 3 glulam appears to have 10 braces per field photos, assumed evenly spaced.
Span 5 glulam appears to have 8 baces per field photos, assume evenly spaced; one brace is missing, therefore twice the unbraced distance shall be used here.
Assume 4x6 stringers unbraced length is full span length, except 13.0' span for 3a); 8.2' unbraced length shall be used.

$L_{u_1} := \frac{L_{span_1}}{11} = 3.91\,ft$ $\quad L_{u_8} := 2\frac{L_{span_8}}{9} = 8.47\,ft$ $\quad L_{u_3} := 8.2\cdot ft$

$$L_{e_i} := \begin{cases} (2.06)\cdot(L_{u_i}) & \text{if } \frac{L_{u_i}}{h_{g_i}} < 7 \\ (1.63)\cdot(L_{u_i}) + (3)\cdot(h_{g_i}) & \text{if } 7 \le \frac{(L_{u_i})}{h_{g_i}} \le 14.3 \\ (1.84)\cdot(L_{u_i}) & \text{otherwise} \end{cases}$$ (ODOT 8.2.4.10)

$$L_e = \begin{pmatrix} 8.1 \\ 16.5 \\ 15.1 \\ 22.8 \\ 18.8 \\ 22.4 \\ 14.7 \\ 17.4 \end{pmatrix} ft$$

$$R_{b_i} := \min\left[\sqrt{\frac{(L_{e_i})\cdot(h_{g_i})}{(b_{g_i})^2}}, 50\right]$$ $R_b^T = (9.17 \quad 9.22 \quad 9.02 \quad 11.09 \quad 10.06 \quad 11.00 \quad 8.91 \quad 13.49)$ (ODOT 8.2.4.10)

$$F_{bE_i} := \frac{(K_{bE.ref_i})\cdot(E_{ref_i})}{(R_{b_i})^2}$$ $F_{bE}^T = (22.4 \quad 22.1 \quad 16.9 \quad 11.2 \quad 13.6 \quad 11.3 \quad 17.3 \quad 10.3)\cdot ks$ (ODOT 8.2.4.10)

$$A_i := \frac{F_{bE_i}}{F_{b_i}}$$ $A^T = (4.51 \quad 3.92 \quad 5.98 \quad 3.95 \quad 4.80 \quad 4.02 \quad 6.13 \quad 2.06)$ (ODOT 8.2.4.10)

$$C_{L_i} := \begin{cases} 1 & \text{if } \text{Braced} = \text{"Yes"} \\ \dfrac{1 + A_i}{1.9} - \sqrt{\dfrac{(1 + A_i)^2}{3.61} - \dfrac{A_i}{0.95}} & \text{otherwise} \end{cases}$$

(Beam Stability factor, ODOT 8.2.4.10)

$$C_L^T = (0.99 \quad 0.98 \quad 0.99 \quad 0.98 \quad 0.99 \quad 0.98 \quad 0.99 \quad 0.96)$$

$$F_{b_i} := \left(F_{b_i}\right) \cdot \left(C_{L_i}\right) \qquad F_b^T = (4.897 \quad 5.554 \quad 2.796 \quad 2.778 \quad 2.787 \quad 2.778 \quad 2.797 \quad 4.815) \cdot ksi$$

Capacities:

$$S_i := \frac{\left(b_{g_i}\right) \cdot \left(h_{g_i}\right)^2}{6} \qquad S^T = (394.2 \quad 96.3 \quad 17.6 \quad 17.6 \quad 17.6 \quad 17.6 \quad 17.6 \quad 394.2) \cdot in^3$$

Section modulus

$$M_{n_i} := F_{b_i} \cdot S_i \qquad M_n^T = (160.9 \quad 44.6 \quad 4.1 \quad 4.1 \quad 4.1 \quad 4.1 \quad 4.1 \quad 158.2) \cdot kip \cdot ft$$

Nominal Moment Capacity, AASHTO 8.6.3-1

$$V_{n_i} := \frac{F_{v_i} \cdot b_{g_i} \cdot h_{g_i}}{1.5} \qquad V_n^T = (45.2 \quad 22.3 \quad 4.3 \quad 4.3 \quad 4.3 \quad 4.3 \quad 4.3 \quad 45.2) \cdot kip$$

Nominal Shear Capacity, AASHTO 8.7-2

$$A_{b_i} := \left(b_{g_i}\right) \cdot \left(L_{b_i}\right) \qquad A_b^T = (27.50 \quad 10.00 \quad 7.00 \quad 7.00 \quad 7.00 \quad 7.00 \quad 7.00 \quad 27.50) \cdot in^2$$

Bearing area

$$C_b := 1.0$$

Bearing adjustment factor, AASHTO 8.8.3

$$R_{n_i} := F_{cp_i} \cdot A_{b_i} \cdot C_b \qquad R_n^T = (33.37 \quad 12.13 \quad 6.53 \quad 6.53 \quad 6.53 \quad 6.53 \quad 6.53 \quad 33.37) \cdot kip$$

Nominal compression capacity perpendicular to grain, AASHTO 8.8.3-1

Component Dead Loads (DC):

Deadload of decking

$W_{deck} := (w_t)\cdot(t_f)\cdot(W_{total})$ $\quad\quad W_{deck} = 0.025\cdot klf$

Deadload of stringer

$W_{g_i} := (w_t)\cdot(h_{g_i})\cdot(b_{g_i})$ $\quad\quad W_g^{\,T} = (0.038 \;\; 0.019 \;\; 0.007 \;\; 0.007 \;\; 0.007 \;\; 0.007 \;\; 0.007 \;\; 0.038)\cdot klf$

Diaphragms (10 cross frames [assume 2x6 nominal members] along 21.75" glulam 43.0' span)

$P_{dia} := 2(w_t)\cdot(1.5\cdot in)\cdot(5.5in)\cdot\sqrt{(21.75\cdot in)^2 + (3\cdot ft + 7\cdot in - 2\cdot 5in)^2}$ $\quad\quad P_{dia} = 0.019\cdot k$

$$W_{dia} := \frac{10\cdot P_{dia}}{43ft} = 4.39\cdot plf$$

Rails: 3ea 2x4 rails each side with 4x4 posts at approx 8' spacings max. Assume post 42" tall + depth to bottom of stringer (conservative). A distributed load shall be calculated for the long glulam span. A rail post is located at the mid span of the 8.2' span of the 13.0' span 1; a rail post is located at the mid span of the 12.4' span of span 8; and a rail post is located at the mid span of the 10.2' span of span 8. All other spans have posts located at piers and do not load spans.

$$W_{rail_i} := (w_t)\cdot\left[3(1.5in)\cdot(3.5in) + \frac{(3.5in)\cdot(3.5in)\cdot\left(42\cdot in + t_f + h_{g_i}\right)}{Min\left(8\cdot ft, 0.5\cdot L_{span_i}\right)}\right]$$

Post is distributed by 8-ft, or half the span length.

$W_{rail}^{\,T} = (8.36 \;\; 10.02 \;\; 8.14 \;\; 8.27 \;\; 8.87 \;\; 8.32 \;\; 9.81 \;\; 8.36)\cdot plf$

Utility: Assume 6" diam. SCH 80 PVC pipe

$W_{util} := 5.42\cdot plf$ $\quad\quad$ Utility shall be assumed as part of DC, per ODOT 2.2.7

Wearing Surface Dead Loads (DW):

N/A

Live Loads (LL):

Pedestrian Loading

$W_{LL} := 90\cdot psf\cdot W_{path} = 0.31\cdot klf$

Analysis Sections:

Spans are simply supported. Max bending moment assumed at mid span. Max shear assumed at a distance from face of support equal to depth of the component (per AASHTO 8.7). Max bearing assumed at location of bearing.

Distributed loads

$$W_{dia.ref} := \left(W_{dia}\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ W_{dia}\right)^T$$

$$W_{STRI.DC_i} := \gamma_{DC.max} \cdot \left(0.5 \cdot W_{deck} + W_{g_i} + 0.5 W_{dia.ref_i} + 0.5 \cdot W_{util}\right)$$

$$W_{STRI.DC}^T = (0.07\ \ 0.04\ \ 0.03\ \ 0.03\ \ 0.03\ \ 0.03\ \ 0.03\ \ 0.07) \cdot klf$$

$$W_{STRI.LL} := \gamma_{LL} \cdot \left(0.5 W_{LL}\right) = 0.27 \cdot klf$$

$$W_{STRI_i} := W_{STRI.DC_i} + W_{STRI.LL}$$

$$W_{STRI}^T = (0.341\ \ 0.315\ \ 0.300\ \ 0.300\ \ 0.300\ \ 0.300\ \ 0.300\ \ 0.341) \cdot klf$$

Demands due to railing DC

$$M_{u.DC.rail.1} := \frac{\gamma_{DC.max} \cdot W_{rail_1} \cdot \left(L_{span_1}\right)^2}{8} = 2.42 \cdot k \cdot ft$$ Rail flexural demand at member 1)

$$V_{u.DC.rail.1} := \frac{\gamma_{DC.max} \cdot W_{rail_1} \left(0.5 \cdot L_{span_1}\right)}{2} = 0.11 \cdot kip$$ Rail shear demand at member 1)

$$M_{u.DC.rail.4} := \frac{\gamma_{DC.max} \cdot W_{rail_8} \cdot \left(L_{span_8}\right)^2}{8} = 1.90 \cdot k \cdot ft$$ Rail flexural demand at member 4)

$$V_{u.DC.rail.4} := \frac{\gamma_{DC.max} \cdot W_{rail_8} \left(0.5 \cdot L_{span_8}\right)}{2} = 0.10 \cdot kip$$ Rail shear demand at member 4)

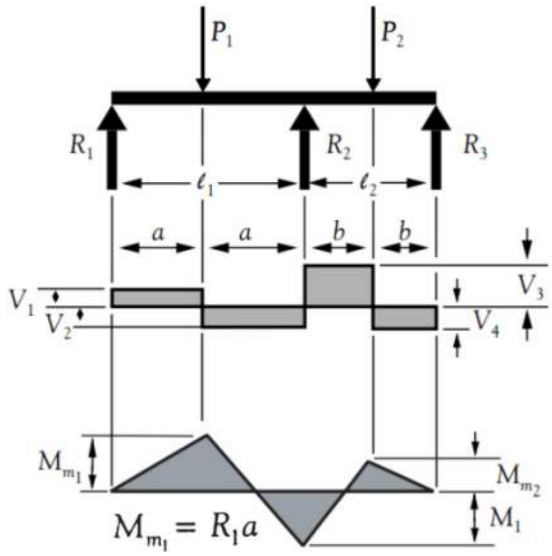
$$M_{u.DC.rail.3b} := \frac{\gamma_{DC.max} \cdot W_{rail_4} \cdot 0.5 \left(L_{span_4}\right)^2}{4} = 0.20 \cdot k \cdot ft$$ Rail flexural demand at member 3b) for positive moment. M = P*L/4 = (W*0.5*L)*L/4 = W*0.5*L^2/4

$$V_{u.DC.rail.3b} := \frac{\gamma_{DC.max} \cdot W_{rail_4} \left(0.5 \cdot L_{span_4}\right)}{2} = 0.03 \cdot kip$$ Rail shear demand at member 3b)

$$M_{u.DC.rail.3c} := \frac{\gamma_{DC.max} \cdot W_{rail_5} \cdot 0.5 \left(L_{span_5}\right)^2}{4} = 0.14 \cdot k \cdot ft$$ Rail flexural demand at member 3c) for positive moment. M = P*L/4 = (W*0.5*L)*L/4 = W*0.5*L^2/4

$$V_{u.DC.rail.3c} := \frac{\gamma_{DC.max} \cdot W_{rail_5} \left(0.5 \cdot L_{span_5}\right)}{2} = 0.03 \cdot kip$$ Rail shear demand at member 3b)

Fig 1:



$$R_1 \; . \; . \; . \; = \frac{M_1}{\ell_1} + \frac{P_1}{2}$$

$$R_2 \; . \; . \; . \; = P_1 + P_2 - R_1 - R_3$$

$$R_3 \; . \; . \; . \; = \frac{M_1}{\ell_2} + \frac{P_2}{2}$$

$$V_1 \; . \; . \; . \; = R_1$$

$$V_2 \; . \; . \; . \; = P_1 - R_1$$

$$V_3 \; . \; . \; . \; = P_2 - R_3$$

$$V_4 \; . \; . \; . \; = R_3$$

$$M_1 \; . \; . \; . \; = -\frac{3}{16}\left(\frac{P_1\ell_1^{\,2} + P_2\ell_2^{\,2}}{\ell_1 + \ell_2}\right)$$

Positive Moment

Rail flexural demand at member 3a) for negative moment. See Fig 1 above (NDS Beam Design Formulas)

$$M_{1.DC.rail.3a.pos} := -\gamma_{DC.max} \cdot \frac{3}{16} \cdot \left[\frac{\left(W_{rail_3} \cdot 0.5 \cdot 8.2 \cdot ft\right) \cdot (8.2 \cdot ft)^2}{L_{span_3}}\right] = -0.04 \cdot k \cdot ft$$

$$R_{1.DC.rail.3a.pos} := \frac{M_{1.DC.rail.3a.pos}}{8.2ft} + \frac{\left(W_{rail_3} \cdot 0.5 \cdot 8.2 \cdot ft\right)}{2} = 11.75\,lbf$$

$$M_{m1.DC.rail.3a.pos} := R_{1.DC.rail.3a.pos} \cdot (0.5 \cdot 8.2ft) = 0.05 \cdot k \cdot ft$$ Rail positive flexural demand at member 3a)

Negative Moment

$$M_{u.DC.rail.3a.neg} := -M_{1.DC.rail.3a.pos} = 0.04 \cdot k \cdot ft$$ Rail negative flexural demand at member 3a)

Shear

$$V_{1.DC.rail.3a} := R_{1.DC.rail.3a.pos} = 11.75\,lbf$$

$$V_{2.DC.rail.3a} := \left(W_{rail_3} \cdot 0.5 \cdot 8.2 \cdot ft\right) - R_{1.DC.rail.3a.pos} = 21.62\,lbf$$

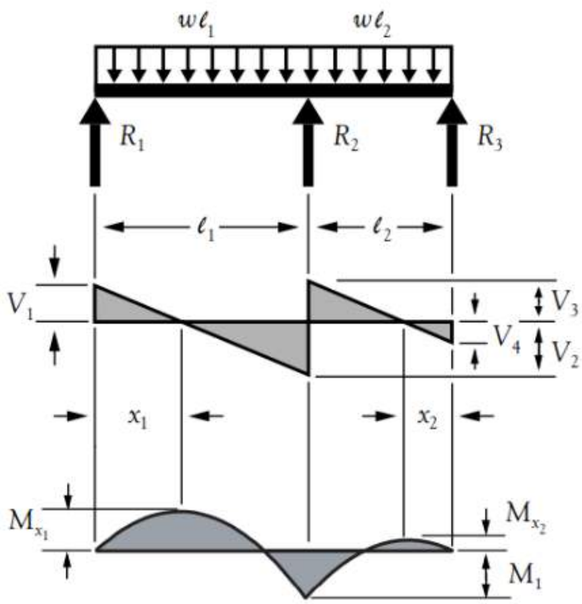
$$R_{3.DC.rail.3a} := \frac{M_{1.DC.rail.3a.pos}}{4.8ft} + 0 = -8.43\,lbf$$

$$V_{3.DC.rail.3a} := -R_{3.DC.rail.3a} = 8.43\,lbf$$

$$V_{4.DC.rail.3a} := R_{3.DC.rail.3a} = -8.43\,lbf$$

Other DC demands at Span 1, member 3a)

Fig 2:



$$R_1 \ldots = \frac{M_1}{\ell_1} + \frac{w\ell_1}{2}$$

$$R_2 \ldots = w\ell_1 + w\ell_2 - R_1 - R_3$$

$$R_3 = V_4 = \frac{M_1}{\ell_2} + \frac{w\ell_2}{2}$$

$$V_1 \ldots = R_1$$

$$V_2 \ldots = w\ell_1 - R_1$$

$$V_3 \ldots = w\ell_2 - R_3$$

$$V_4 \ldots = R_3$$

$$M_1 \ldots = -\frac{w\ell_2^{\,3} + w\ell_1^{\,3}}{8(\ell_1 + \ell_2)}$$

$$M_{x_1}\left(\text{when } x_1 = \frac{R_1}{w}\right) = R_1 x_1 - \frac{w x_1^2}{2}$$

Positive Moment

Other (non rail) DC flexural demand at member 3a) for moment. See Fig 2 above (NDS Beam Design Formulas)

$$\gamma_{DC.max} = 1.25$$

$$\gamma_{DC.min} = 0.90$$

$$Min_{max} := \frac{\gamma_{DC.min}}{\gamma_{DC.max}} = 0.72$$

$$M_{1.DC.3a.pos} := -W_{STRI.DC_3} \frac{\left[Min_{max} \cdot (4.8 \cdot ft)^3 + (8.2 \cdot ft)^3\right]}{8 \cdot L_{span_3}} = -0.17 \cdot k \cdot ft$$

$$R_{1.DC.3a.pos} := \frac{M_{1.DC.3a.pos}}{8.2ft} + \frac{W_{STRI.DC_3} \cdot 8.2ft}{2} = 0.09 \cdot kip$$

$$x_{1.DC.3a.pos} := \frac{R_{1.DC.3a.pos}}{W_{STRI.DC_3}} = 3.36\,ft$$

This is not the midle of the 8.2' clear span, but it will be assumed as the value at mid span to coincide with rail and live load demands.

$$M_{x1.DC.3a.pos} := R_{1.DC.3a.pos} \cdot x_{1.DC.3a.pos} - \frac{W_{STRI.DC_3} \cdot x_{1.DC.3a.pos}^{\;2}}{2} = 0.15 \cdot k \cdot ft$$

Negative Moment

$$M_{1.DC.3a.neg} := W_{STRI.DC_3} \frac{\left[(4.8 \cdot ft)^3 + (8.2 \cdot ft)^3\right]}{8 \cdot L_{span_3}} = 0.17 \cdot kip \cdot ft$$

Shear

$$V_{1.DC.3a} := R_{1.DC.3a.pos} = 91.96\ lbf$$

$$V_{2.DC.3a} := W_{STRI.DC_3} \cdot 8.2ft - R_{1.DC.3a.pos} = 132.46\ lbf$$

$$R_{3.DC.3a} := \frac{M_{1.DC.3a.pos}}{4.8ft} + \frac{W_{STRI.DC_3} \cdot Min_{max} \cdot 4.8ft}{2} = 12.70\ lbf$$

$$V_{3.DC.3a} := W_{STRI.DC_3} \cdot Min_{max} \cdot 4.8ft - R_{3.DC.3a} = 81.88\ lbf$$

$$V_{4.DC.3a} := R_{3.DC.3a} = 12.70\ lbf$$

<u>Live Loads at Span 1, member 3a)</u>

LL flexural demand at member 3a) for moment. See Fig 2 above (NDS Beam Design Formulas)
Positive moment will pattern load the larger of the two spans. Negative moment shall load both spans

Positive Moment

$$M_{1.LL.3a.pos} := -W_{STRI.LL} \frac{\left[(0 \cdot ft)^3 + (8.2 \cdot ft)^3\right]}{8 \cdot L_{span_3}} = -1.44 \cdot k \cdot ft$$

$$R_{1.LL.3a.pos} := \frac{M_{1.LL.3a.pos}}{8.2ft} + \frac{W_{STRI.LL} \cdot 8.2ft}{2} = 0.94 \cdot kip$$

$$x_{1.LL.3a.pos} := \frac{R_{1.LL.3a.pos}}{W_{STRI.LL}} = 3.45\ ft$$

This is not the midle of the 8.2' clear span, but it will be assumed as the value at mid span to coinside with rail and live load demands.

$$M_{x1.LL.3a.pos} := R_{1.LL.3a.pos} \cdot x_{1.LL.3a.pos} - \frac{W_{STRI.LL} \cdot x_{1.LL.3a.pos}^{\ 2}}{2} = 1.62 \cdot k \cdot ft$$

Negative Moment

$$M_{1.LL.3a.neg} := M_{1.DC.3a.neg} \cdot \frac{W_{STRI.LL}}{W_{STRI.DC_3}} = 1.73 \cdot kip \cdot ft$$

Controlling Flexural case for Span 1, member 3a)

$M_{u.3a.pos} := M_{m1.DC.rail.3a.pos} + M_{x1.DC.3a.pos} + M_{x1.LL.3a.pos} = 1.83 \cdot k \cdot ft$

$M_{u.3a.neg} := M_{u.DC.rail.3a.neg} + M_{u.DC.rail.3a.neg} + M_{1.LL.3a.neg} = 1.81 \cdot k \cdot ft$

$M_{3a} := \text{If}\left(M_{u.3a.pos} > M_{u.3a.neg}, \text{"Pos"}, \text{"Neg"}\right) = \text{"Pos"}$

$M_{u.DC.rail.3a} := \text{If}\left(M_{3a} = \text{"Pos"}, M_{m1.DC.rail.3a.pos}, M_{u.DC.rail.3a.neg}\right) = 0.05 \cdot k \cdot ft$

$M_{u.DC.other.3a} := \text{If}\left(M_{3a} = \text{"Pos"}, M_{x1.DC.3a.pos}, M_{u.DC.rail.3a.neg}\right) = 0.15 \cdot k \cdot ft$

$Mu_{LL.3a} := \text{If}\left(M_{3a} = \text{"Pos"}, M_{x1.LL.3a.pos}, M_{1.LL.3a.neg}\right) = 1.62 \cdot k \cdot ft$

Shear in positive moment configuration

$$R_{1.LL.3a.pos} := \frac{M_{1.LL.3a.pos}}{8.2ft} + \frac{W_{STRI.LL} \cdot 8.2ft}{2} = 0.94 \cdot kip$$

$V_{1.LL.3a.pos} := R_{1.LL.3a.pos} = 940.53\,lbf$

$V_{2.LL.3a.pos} := W_{STRI.LL} \cdot 8.2ft - R_{1.LL.3a.pos} = 1292.69\,lbf$

$$R_{3.LL.3a.pos} := \frac{M_{1.LL.3a.pos}}{4.8ft} + 0 = -300.80\,lbf$$

$V_{3.LL.3a.pos} := W_{STRI.LL} \cdot 4.8ft - R_{3.LL.3a.pos} = 1608.05\,lbf$

$V_{4.LL.3a.pos} := R_{3.LL.3a.pos} = -300.80\,lbf$

Shear in negative moment configuration

$$R_{1.LL.3a.neg} := \frac{M_{1.LL.3a.neg}}{8.2ft} + \frac{W_{STRI.LL} \cdot 8.2ft}{2} = 1.33 \cdot kip$$

$V_{1.LL.3a.neg} := R_{1.LL.3a.neg} = 1328.01\,lbf$

$V_{2.LL.3a.neg} := W_{STRI.LL} \cdot 8.2ft - R_{1.LL.3a.neg} = 905.21\,lbf$

$$R_{3.LL.3a.neg} := \frac{M_{1.LL.3a.neg}}{4.8ft} + \frac{W_{STRI.LL} \cdot 4.8ft}{2} = 1014.76\,lbf$$

$V_{3.LL.3a.neg} := W_{STRI.LL} \cdot 4.8ft - R_{3.LL.3a.neg} = 292.49\,lbf$

$V_{4.LL.3a.neg} := R_{3.LL.3a.neg} = 1014.76\,lbf$

<u>Controlling Shear case for Span 1, member 3a)</u>

$$V_{u.3a.1.pos} := V_{1.DC.rail.3a} + V_{1.DC.3a} + V_{1.LL.3a.pos} = 1.04 \cdot kip$$
$$V_{u.3a.2.pos} := V_{2.DC.rail.3a} + V_{2.DC.3a} + V_{2.LL.3a.pos} = 1.45 \cdot kip$$
$$V_{u.3a.3.pos} := V_{3.DC.rail.3a} + V_{3.DC.3a} + V_{3.LL.3a.pos} = 1.70 \cdot kip$$
$$V_{u.3a.4.pos} := V_{4.DC.rail.3a} + V_{4.DC.3a} + V_{4.LL.3a.pos} = -0.30 \cdot kip$$

$$V_{u.3a.4.pos} := -V_{u.3a.4.pos}$$ Changing sign

$$V_{u.3a.pos.max} := \max\left(V_{u.3a.1.pos}, V_{u.3a.2.pos}, V_{u.3a.3.pos}, V_{u.3a.4.pos}\right) = 1.70 \cdot kip$$

$$V_{u.3a.1.neg} := V_{1.DC.rail.3a} + V_{1.DC.3a} + V_{1.LL.3a.neg} = 1.43 \cdot kip$$
$$V_{u.3a.2.neg} := V_{2.DC.rail.3a} + V_{2.DC.3a} + V_{2.LL.3a.neg} = 1.06 \cdot kip$$
$$V_{u.3a.3.neg} := V_{3.DC.rail.3a} + V_{3.DC.3a} + V_{3.LL.3a.neg} = 0.38 \cdot kip$$
$$V_{u.3a.4.neg} := V_{4.DC.rail.3a} + V_{4.DC.3a} + V_{4.LL.3a.neg} = 1.02 \cdot kip$$

$$V_{u.3a.neg.max} := \max\left(V_{u.3a.1.neg}, V_{u.3a.2.neg}, V_{u.3a.3.neg}, V_{u.3a.4.neg}\right) = 1.43 \cdot kip$$

$$V_{u.3a.max} := \mathrm{Max}\left(V_{u.3a.pos.max}, V_{u.3a.neg.max}\right) = 1.70 \cdot kip$$

$$V_{u.DC.rail.3a} = 8.43\,lbf$$
$$V_{u.DC.3a} = 81.88\,lbf$$

$$V_{u.LL.3a} = 1.61 \cdot kip$$

Flexure

Combine all loads

$$Mu_{DC.Rail} := \left(M_{u.DC.rail.1}\ \ 0\ \ M_{u.DC.rail.3a}\ \ M_{u.DC.rail.3b}\ \ M_{u.DC.rail.3c}\ \ 0\ \ 0\ \ M_{u.DC.rail.4}\right)^T$$

$$Mu_{DC.Rail}{}^T = (2.42\ \ 0.00\ \ 0.05\ \ 0.20\ \ 0.14\ \ 0.00\ \ 0.00\ \ 1.90)\cdot k\cdot ft$$

$$M_{u.DC_i} := \frac{W_{STRI.DC_i}\cdot\left(L_{span_i}\right)^2}{8} \qquad M_{u.DC_3} := M_{u.DC.other.3a}$$ Replacing the value for member 3a)

$$M_{u.DC_i} := M_{u.DC_i} + Mu_{DC.Rail_i}$$ Adding the DC demands for Railing calculated above.

$$M_{u.DC}{}^T = (18.35\ \ 0.38\ \ 0.20\ \ 0.72\ \ 0.50\ \ 0.51\ \ 0.22\ \ 14.41)\cdot kip\cdot ft$$

$$M_{u.LL_i} := \frac{W_{STRI.LL}\cdot\left(L_{span_i}\right)^2}{8} \qquad M_{u.LL_3} := Mu_{LL.3a}$$ Replacing the value for member 3a)

$$M_{u.LL}{}^T = (62.95\ \ 2.43\ \ 5.75\ \ 5.23\ \ 3.54\ \ 5.07\ \ 2.18\ \ 49.42)\cdot kip\cdot ft$$

$$M_{u_i} := \frac{W_{STRI_i}\cdot\left(L_{span_i}\right)^2}{8} \qquad M_u{}^T = (78.88\ \ 2.81\ \ 6.33\ \ 5.76\ \ 3.90\ \ 5.58\ \ 2.40\ \ 61.93)\cdot kip\cdot ft$$

$$Bending_{Ratio_i} := \frac{\phi_f\cdot M_{n_i}}{M_{u_i}} \qquad Bending_{Ratio}{}^T = (1.73\ \ 13.49\ \ 0.55\ \ 0.60\ \ 0.89\ \ 0.62\ \ 1.46\ \ 2.17)$$

$$RF_{M.u_i} := \frac{\Phi_f\cdot\left(M_{n_i}\right) - M_{u.DC_i}}{M_{u.LL_i}} \qquad RF_{M.u}{}^T = (1.88\ \ 15.43\ \ 0.57\ \ 0.52\ \ 0.84\ \ 0.58\ \ 1.50\ \ 2.43)$$

Note: RFs for 10.75" and 21.75" glulams, and 4x6 stringers 8' and less > 1.0. Say OK

RFs for 4x6 stringers with simple spans greater than 8' or having rail posts not located solely at supports < 1.0. No good.

Live load deflection

$$I_i := \frac{b_{g_i} \cdot \left(h_{g_i}\right)^3}{12} \qquad I^T = (4287.13 \quad 517.62 \quad 48.53 \quad 48.53 \quad 48.53 \quad 48.53 \quad 48.53 \quad 4287.13) \cdot in^4$$

$$\Delta_i := \frac{5 \cdot W_{STRI.LL} \cdot \left(L_{span_i}\right)^4}{384 \cdot E_{ref_i} \cdot I_i} \qquad \Delta^T = (2.86 \quad 0.04 \quad 2.00 \quad 1.65 \quad 0.76 \quad 1.55 \quad 0.29 \quad 1.76) \cdot in$$

$$\Delta_{limit} := \frac{L_{span}}{360} \qquad \Delta_{limit}^T = (1.43 \quad 0.28 \quad 0.43 \quad 0.41 \quad 0.34 \quad 0.41 \quad 0.27 \quad 1.27) \cdot in$$ (BDM 1.8.2)

$$\Delta_{check_i} := \text{If}\left(\Delta_i \le \Delta_{limit_i}, \text{"OK"}, \text{"NG"}\right)$$

$$\Delta_{check}^T = (\text{"NG"} \quad \text{"OK"} \quad \text{"NG"} \quad \text{"NG"} \quad \text{"NG"} \quad \text{"NG"} \quad \text{"NG"} \quad \text{"NG"})$$

$$Deflection_{RF_i} := \frac{\Delta_{limit_i}}{\Delta_i} \qquad Deflection_{RF}^T = (0.50 \quad 7.98 \quad 0.22 \quad 0.25 \quad 0.45 \quad 0.26 \quad 0.93 \quad 0.72)$$

Shear

<u>Combine all loads</u>

$$Vu_{DC.Rail} := \left(V_{u.DC.rail.1} \quad 0 \quad V_{u.DC.rail.3a} \quad V_{u.DC.rail.3b} \quad V_{u.DC.rail.3c} \quad 0 \quad 0 \quad V_{u.DC.rail.4}\right)^T$$

$$Vu_{DC.Rail}^T = (112.33 \quad 0.00 \quad 8.43 \quad 32.05 \quad 28.29 \quad 0.00 \quad 0.00 \quad 99.53) \cdot lbf$$

$$V_{u.DC_i} := W_{STRI.DC_i} \cdot \left(\frac{L_{span_i}}{2} - h_{g_i}\right) \qquad V_{u.DC_3} := V_{u.DC.3a}$$ Replacing the value for member 3a)

$$V_{u.DC_i} := V_{u.DC_i} + Vu_{DC.Rail_i}$$ Adding the DC demands for Railing calculated above.

$$V_{u.DC}^T = (1469.90 \quad 140.96 \quad 90.31 \quad 189.18 \quad 155.32 \quad 154.40 \quad 96.93 \quad 1288.16)\,lbf$$

$$V_{u.LL_i} := W_{STRI.LL} \cdot \left(\frac{L_{span_i}}{2} - h_{g_i}\right) \qquad V_{u.LL_3} := V_{u.LL.3a}$$ Replacing the value for member 3a)

$$V_{u.LL}^T = (5.36 \quad 0.91 \quad 1.61 \quad 1.56 \quad 1.26 \quad 1.54 \quad 0.96 \quad 4.69) \cdot kip$$

$$V_{u_i} := W_{STRI_i} \cdot \left(\frac{L_{span_i}}{2} - h_{g_i}\right) \qquad V_u^T = (6.72 \quad 1.05 \quad 1.81 \quad 1.72 \quad 1.39 \quad 1.69 \quad 1.06 \quad 5.88) \cdot kip$$

$$Shear_{Ratio_i} := \frac{\phi_s \cdot V_{n_i}}{V_{u_i}} \qquad Shear_{Ratio}^T = (5.05 \quad 16.00 \quad 1.80 \quad 1.90 \quad 2.34 \quad 1.93 \quad 3.07 \quad 5.76)$$

$$RF_{V.u_i} := \frac{\Phi_v \cdot \left(V_{n_i}\right) - V_{u.DC_i}}{V_{u.LL_i}} \qquad RF_{V.u}^T = (6.05 \quad 18.33 \quad 1.97 \quad 1.96 \quad 2.46 \quad 2.02 \quad 3.28 \quad 6.95)$$

Note: Rating factors for all members > 1.0. Say OK
C/D ratios for all members > 1.0. Say OK

Bearing

Spans adjacent to 21.75" glulams are supported by 21.75" glulams, thus bearing will include tributary load of adjacent spans.

$w_{STRI.DC_3} \cdot \frac{12.2 \cdot ft}{2} = 0.17 \cdot kip \qquad w_{STRI.DC_2} \cdot \frac{8.45 \cdot ft}{2} = 0.18 \cdot kip$

$$Adjust_{1.DC} := Max\left(w_{STRI.DC_3} \cdot \frac{12.2 \cdot ft}{2}, w_{STRI.DC_2} \cdot \frac{8.45 \cdot ft}{2}\right) = 178.89\, lbf$$

$$Adjust_{2.DC} := Max\left(w_{STRI.DC_3} \cdot \frac{6.1 \cdot ft}{2}, w_{STRI.DC_2} \cdot \frac{8.45 \cdot ft}{2}\right) = 178.89\, lbf$$

$$Wt_{Adj.DC} := \left(Adjust_{1.DC} \quad 0 \quad 0 \quad 0 \quad 0 \quad 0 \quad 0 \quad Adjust_{2.DC}\right)^T$$

$$Wt_{Adj.DC}^{\;T} = (0.18 \quad 0.00 \quad 0.00 \quad 0.00 \quad 0.00 \quad 0.00 \quad 0.00 \quad 0.18) \cdot kip$$

$w_{STRI.LL} \cdot \frac{12.2 \cdot ft}{2} = 1.66 \cdot kip \qquad w_{STRI.LL} \cdot \frac{8.45 \cdot ft}{2} = 1.15 \cdot kip$

$$Adjust_{1.LL} := Max\left(w_{STRI.LL} \cdot \frac{12.2 \cdot ft}{2}, w_{STRI.LL} \cdot \frac{8.45 \cdot ft}{2}\right) = 1661.30\, lbf$$

$$Adjust_{2.LL} := Max\left(w_{STRI.LL} \cdot \frac{6.1 \cdot ft}{2}, w_{STRI.LL} \cdot \frac{8.45 \cdot ft}{2}\right) = 1150.65\, lbf$$

$$Wt_{Adj.LL} := \left(Adjust_{1.LL} \quad 0 \quad 0 \quad 0 \quad 0 \quad 0 \quad 0 \quad Adjust_{2.LL}\right)^T$$

$$Wt_{Adj.LL}^{\;T} = (1.66 \quad 0.00 \quad 0.00 \quad 0.00 \quad 0.00 \quad 0.00 \quad 0.00 \quad 1.15) \cdot kip$$

$R_{u.DC_i} := w_{STRI.DC_i} \cdot \frac{L_{span_i}}{2} + Wt_{Adj.DC_i} \qquad R_{u.DC_3} := R_{1.DC.3a.pos}$ Replacing the value for member 3a)

$R_{u.DC_i} := R_{u.DC_i} + Vu_{DC.Rail_i}$ Adding the DC demands for Railing calculated above.

$$R_{u.DC}^{\;T} = (1.77 \quad 0.18 \quad 0.10 \quad 0.20 \quad 0.17 \quad 0.17 \quad 0.11 \quad 1.59) \cdot kip$$

$R_{u.LL_i} := w_{STRI.LL} \cdot \frac{L_{span_i}}{2} + Wt_{Adj.LL_i} \qquad R_{u.LL_3} := Max\left(R_{1.LL.3a.pos}, R_{1.LL.3a.neg}\right) = 1.33 \cdot kip$

Replacing the value for member 3a)

$$R_{u.LL}^{\;T} = (7.52 \quad 1.15 \quad 1.33 \quad 1.69 \quad 1.39 \quad 1.66 \quad 1.09 \quad 6.34) \cdot kip$$

$R_{u_i} := R_{u.DC_i} + R_{u.LL_i}$ $\qquad$ $R_u^T = (9.29 \;\; 1.33 \;\; 1.43 \;\; 1.89 \;\; 1.56 \;\; 1.83 \;\; 1.20 \;\; 7.93) \cdot kip$

$\text{Bearing}_{\text{Ratio}_i} := \dfrac{\phi_{cp} \cdot R_{n_i}}{R_{u_i}}$ $\qquad$ $\text{Bearing}_{\text{Ratio}}^T = (3.23 \;\; 8.21 \;\; 4.12 \;\; 3.11 \;\; 3.78 \;\; 3.22 \;\; 4.90 \;\; 3.79)$

$RF_{R.u_i} := \dfrac{\Phi_b \cdot \left(R_{n_i}\right) - R_{u.DC_i}}{R_{u.LL_i}}$ $\qquad$ $RF_{R.u}^T = (3.76 \;\; 9.33 \;\; 4.35 \;\; 3.36 \;\; 4.11 \;\; 3.44 \;\; 5.30 \;\; 4.49)$

Note: Rating factors for all members > 1.0. Say OK
C/D ratios for all members > 1.0. Say OK

Bridge Type: Timber and Glulam pedestrian bidge.simple supports for dead load and live load.

DECKING ANALYSIS:

References:
AASHTO LRFD BDS 9th Ed. 2020 (AASHTO)
AASHTO MBE 3rd E. 2018 with 2019 Interims (MBE)
AASTHO LRFD Guide Spec for the Design of Pedestrian Bridges 2009 w/ 2015 Interims (PED)
ODOT LRFR Manual June 2018 (ODOT)
ODOT Bridge Design Manual (BDM)
AISC Steel Construction Manual (AISC)

LRFR Strength Limit State:

$$RF = \frac{\phi_c \cdot \phi_s \cdot \phi \cdot (R_n) - (\gamma_{DC}) \cdot (DC) - (\gamma_{DW}) \cdot (DW)}{(\gamma_L) \cdot (LL + IM)}$$ (MBE 6A.4.2.1-1)

Resistance Factors:

$\phi_s := 0.75$	LRFD resistance factor for shear	(AASHTO 8.5.2.2)
$\phi_f := 0.85$	LRFD resistance factor for flexure	(AASHTO 8.5.2.2)
$\phi_{cp} := 0.90$	LRFD resistance factor for compression perpendicular to grain.	(AASHTO 8.5.2.2)
$\phi_{cl} := 0.90$	LRFD resistance factor for compression parallel to grain.	(AASHTO 8.5.2.2)
$\phi_c := 0.95$	Condition factor for deck condition rating = 5 (Fair)	(MBE T. 6A.4.2.3-1)
$\phi_{sf} := 1.00$	System Factor for Flexure, structure type: "Timber Stringers"	(ODOT 1.4.1.4)
$\phi_{sv} := 1.00$	System Factor for Shear, structure type: "Timber Stringers"	(ODOT 1.4.1.4)
$\phi_{sa} := 1.00$	System Factor for Axial, All other girder bridges and slab bridges	(MBE 6A.4.2.4-1)

Combined Resistance Factors:

For Flexure:	$\Phi_f := \phi_f \cdot (\max(\phi_c \cdot \phi_{sf}, 0.85))$	$\Phi_f = 0.807$
For Shear:	$\Phi_v := \phi_s \cdot (\max(\phi_c \cdot \phi_{sv}, 0.85))$	$\Phi_v = 0.712$
For Bearing	$\Phi_b := \phi_{cp} \cdot (\max(\phi_c \cdot \phi_{sa}, 0.85))$	$\Phi_b = 0.855$

(Note: $\phi_c \phi_s \geq 0.85$ per MBE 6A.4.2.1-3)

Load Factors:

Dead Load Factors γ_{DC}:

$\gamma_{DC.max} := 1.25$ max. MBE T. 6A.4.2.2-1 for structural components and attachments STR I

Live Load Factors γ_L:

$\gamma_{LL} := 1.750$ MBE T. 6A.4.2.2-1, assume pedestrian loading as Inventory

Dynamic load allowance, IM, is not required with pedestrian loading, PED 3.1

Bridge Members

Analysis below shall consist of decking members:

1) 2x12 decking
2) 2x6 decking.
3) 2x4 decking,

Bridge Geometry:

Deck Out to Out: $W_{total} := 4ft + 0in = 4.00\,ft$

$S_{s.oto} := 3 \cdot ft + 7 \cdot in$ Spacing stringers, out to out dimension

Path Width: $W_{path} := S_{s.oto} - 0.5 \cdot 2 \cdot (1.5 \cdot in) = 3.46\,ft$

Span: $L_{span} := \left(S_{s.oto} \quad S_{s.oto} \quad S_{s.oto} \right)^T$

$i := 1 .. \text{length}\left(L_{span}\right)$ Set counter

Member Height: $h_g := (1.5 \quad 1.5 \quad 1.5)^T\,in$

Member Width: $b_g := (11.25 \quad 5.5 \quad 3.5)^T\,in$

Length of bearing: $L_b := (3.5 \quad 3.5 \quad 3.5)^T \cdot in$

Decking bearing controlled by 4x6 stringers.

Deck Thickness: $t_f := 1.5in$ 2"x12" / 2"x6" / 2"x4" timber decking

Left Cantilever: $L_{Cant} := 0.5 \cdot \left(W_{total} - S_{s.oto}\right) = 0.21\,ft$

Right Cantilever: $R_{Cant} := L_{Cant} = 0.21\,ft$

Material Properties:

$w_t := 0.050\text{kcf}$ (AASHTO Table 3.5.1-1) (Assumed timber species of douglas fir is softwood)

Dimensional Lumber (L) (Douglas Fir, Dimension ≥ 2 in. wide, Select Structural), per ODOT 8.2.4

"Select Structural" grade assumed here as values provided in ODOT LRFR match AASHTO Select Structural

$F_{bo} := 1.5\text{ksi}$ (ODOT 8.2.4) Bending stress

$F_{vo} := 0.180\text{ksi}$ (ODOT 8.2.4) Shear parallel to grain

$F_{cpo} := 0.625\text{ksi}$ (ODOT 8.2.4) Compression perpendicular to grain

$F_{co} := 1.700\text{ksi}$ (AASHTO Table 8.4.1.1.4-1) Compression parallel to grain

$E_o := 1900\text{ksi}$ (ODOT 8.2.4) Mod. of Elasticity

Adjustment Factors (AASHTO 8.4.4)

$C_M := 1.0$ (Wet service factor for Glu-Lam less than 16%, and sawn lumber less than 19%, ODOT 8.2.4.3). Unless submerged, timber is considered dry (BDM 1.8.2)

$C_{F.Fbo} := 0.86$ Size factor, (AASHTO T. 8.4.4.4-2)

$C_{F.Eo} := 1.0$

$C_{F.o} := 1.0$

$C_{fu} := (1.20 \quad 1.15 \quad 1.10)^T$ (Flat-use factor, AASHTO T. 8.4.4.6-1)

$C_{i.E} := 0.95$ (Incising factor, for E_o, ODOT 8.2.4.6)

$C_i := 0.80$ Incising factor, for F.bo and F.von (ODOT 8.2.4.7), & F.to and F.co (AASHTO T. 8.4.4.7-1)

$C_{i.cpo} := 1.0$ Incising factor, for F_{cpo} (ODOT 8.2.4.7)

$C_d := 1.0$ (Deck factor, AASHTO 8.4.4.8, per ODOT 8.2.4.8)

$C_{\lambda.1} := 0.8$ Time effect factor, Strength Limit State 1 (ODOT 8.2.4.9)

$E_i := (E_o)\cdot(C_M)\cdot(C_{i.E})$ $\qquad E^T = (1805 \quad 1805 \quad 1805)\cdot\text{ksi}$ (ODOT 8.2.4.1)

$C_{KF.f} := \frac{2.5}{\phi_f} = 2.94$ Format conversion factor, F.b (ODOT 8.2.4.2)

$C_{KF.s} := \frac{2.5}{\phi_s} = 3.33$ Format conversion factor, F.v (ODOT 8.2.4.2)

$C_{KF.cp} := \frac{2.1}{\phi_{cp}} = 2.33$ Format conversion factor, F.cp (compression perpendicular to grain.) (ODOT 8.2.4.2)

$F_{b_i} := (F_{bo})\cdot(C_{KF.f})\cdot(C_M)\cdot(C_{F.Fbo})\cdot(C_{fu_i})\cdot(C_i)\cdot(C_d)\cdot C_{\lambda.1}$ (ODOT 8.2.4.1)

$F_b{}^T = (2.91 \quad 2.79 \quad 2.67)\cdot ksi$

$F_{v_i} := (F_{vo})\cdot(C_{KF.s})\cdot(C_M)\cdot(C_i)\cdot(C_{\lambda.1})$ (ODOT 8.2.4.1)

$F_v{}^T = (0.38 \quad 0.38 \quad 0.38)\cdot ksi$

$F_c := (F_{co})\cdot(C_{KF.s})\cdot(C_M)\cdot(C_{F.o})\cdot(C_i)\cdot(C_{\lambda.1}) = 3.63\cdot ksi$ (AASHTO 8.4.4.1-4)

$F_{cp_i} := (F_{cpo})\cdot(C_{KF.cp})\cdot(C_M)(C_i)(C_{\lambda.1})$ (ODOT 8.2.4.1)

$F_{cp}{}^T = (0.93 \quad 0.93 \quad 0.93)\cdot ksi$

Beam stability factor, C_L calculated below. F_b will be adjusted to account for C_L. Calculations below are based on AASHTO 8.6.2 / ODOT 8.2.4.10.

Braced := "No" — "Yes" if compression side of beam is continuously braced and beam is braced laterally at supports, Else "No".

$K_{bE} := 0.76$ (Euler buckling coefficient for visually graded lumber)

$L_u := L_{span}$ Assume laterally braced at stringers

(ODOT 8.2.4.10)

$$L_{e_i} := \begin{cases} (2.06)\cdot(L_{u_i}) & \text{if } \dfrac{L_{u_i}}{h_{g_i}} < 7 \\ (1.63)\cdot(L_{u_i}) + (3)\cdot(h_{g_i}) & \text{if } 7 \le \dfrac{(L_{u_i})}{h_{g_i}} \le 14.3 \\ (1.84)\cdot(L_{u_i}) & \text{otherwise} \end{cases}$$

$L_e{}^T = (6.59 \quad 6.59 \quad 6.59)\, ft$

$$R_{b_i} := \min\left[\sqrt{\frac{(L_{e_i})\cdot(h_{g_i})}{(b_{g_i})^2}}, 50\right] \qquad R_b{}^T = (0.97 \quad 1.98 \quad 3.11)$$ (ODOT 8.2.4.10)

$$F_{bE_i} := \frac{(K_{bE})\cdot(E_i)}{(R_{b_i})^2} \qquad F_{bE}{}^T = (1462.9 \quad 349.7 \quad 141.6)\cdot ksi$$ (ODOT 8.2.4.10)

$$A_i := \frac{F_{bE_i}}{F_{b_i}} \qquad A^T = (502.05 \quad 125.21 \quad 53.01)$$ (ODOT 8.2.4.10)

$$C_{L_i} := \begin{cases} 1 & \text{if Braced = "Yes"} \\ \frac{1 + A_i}{1.9} - \sqrt{\frac{(1 + A_i)^2}{3.61} - \frac{A_i}{0.95}} & \text{otherwise} \end{cases}$$

(Beam Stability factor, ODOT 8.2.4.10)

$$C_L^T = (1.00 \quad 1.00 \quad 1.00)$$

$$F_{b_i} := (F_{b_i}) \cdot (C_{L_i}) \qquad F_b^T = (2.914 \quad 2.791 \quad 2.668) \cdot ksi$$

Capacities:

$$S_i := \frac{(b_{g_i}) \cdot (h_{g_i})^2}{6} \qquad S^T = (4.2 \quad 2.1 \quad 1.3) \cdot in^3$$

Section modulus

$$M_{n_i} := F_{b_i} \cdot S_i \quad M_n^T = (1.0 \quad 0.5 \quad 0.3) \cdot kip \cdot ft$$

Nominal Moment Capacity, AASHTO 8.6.3-1

$$V_{n_i} := \frac{F_{v_i} \cdot b_{g_i} \cdot h_{g_i}}{1.5} \qquad V_n^T = (4.3 \quad 2.1 \quad 1.3) \cdot kip$$

Nominal Shear Capacity, AASHTO 8.7-2

$$A_{b_i} := (b_{g_i}) \cdot (L_{b_i}) \qquad A_b^T = (39.38 \quad 19.25 \quad 12.25) \cdot in^2$$

Bearing area

$$C_b := 1.0$$

Bearing adjustment factor, AASHTO 8.8.3

$$R_{n_i} := F_{cp_i} \cdot A_{b_i} \cdot C_b$$

Nominal compression capacity perpendicular to grain, AASHTO 8.8.3-1

$$R_n^T = (36.75 \quad 17.97 \quad 11.43) \cdot kip$$

Component Dead Loads (DC):

Deadload of decking

$$W_{deck} := (w_t) \cdot (t_f) \cdot (b_g) \qquad W_{deck}^T = (5.86 \quad 2.86 \quad 1.82) \cdot plf$$

Wearing Surface Dead Loads (DW):

N/A

Live Loads (LL):

Pedestrian Loading

$$W_{LL} := (90 \cdot psf) \cdot (b_g) \qquad W_{LL}^T = (84.38 \quad 41.25 \quad 26.25) \cdot plf$$

Analysis Sections:

Spans are simply supported. Max bending moment assumed at mid span. Max shear assumed at a distance from face of support equal to depth of the component (per AASHTO 8.7). Max bearing assumed at location of bearing. For decking, neglect overhangs as they reduce max positive moment.

Distributed loads

$w_{STRI.DC_i} := \gamma_{DC.max} \cdot \left(w_{deck_i}\right)$ $\qquad w_{STRI.DC}^T = (7.32 \quad 3.58 \quad 2.28) \cdot plf$

$w_{STRI.LL_i} := \gamma_{LL} \cdot \left(w_{LL_i}\right)$ $\qquad w_{STRI.LL}^T = (0.15 \quad 0.07 \quad 0.05) \cdot klf$

$w_{STRI_i} := w_{STRI.DC_i} + w_{STRI.LL_i}$ $\qquad w_{STRI}^T = (0.155 \quad 0.076 \quad 0.048) \cdot klf$

Flexure

$$M_{u.DC_i} := \frac{w_{STRI.DC_i} \cdot \left(L_{span_i}\right)^2}{8} \qquad M_{u.DC}^T = (11.8 \quad 5.7 \quad 3.7) \cdot lbf \cdot ft$$

$$M_{u.LL_i} := \frac{w_{STRI.LL_i} \cdot \left(L_{span_i}\right)^2}{8} \qquad M_{u.LL}^T = (0.24 \quad 0.12 \quad 0.07) \cdot kip \cdot ft$$

$$M_{u_i} := M_{u.DC_i} + M_{u.LL_i} \qquad M_u^T = (0.25 \quad 0.12 \quad 0.08) \cdot kip \cdot ft$$

$$Bending_{Ratio_i} := \frac{\phi_f \cdot M_{n_i}}{M_{u_i}} \qquad Bending_{Ratio}^T = (3.50 \quad 3.35 \quad 3.21)$$

$$RF_{M.u_i} := \frac{\Phi_f \cdot \left(M_{n_i}\right) - M_{u.DC_i}}{M_{u.LL_i}} \qquad RF_{M.u}^T = (3.44 \quad 3.29 \quad 3.15)$$

Note: Rating factors for all members > 1.0. Say OK
C/D ratios for all members > 1.0. Say OK

Live load deflection

$I_i := \frac{b_{g_i} \cdot \left(h_{g_i}\right)^3}{12}$ $\quad I^T = (3.16 \quad 1.55 \quad 0.98) \cdot in^4$

$\Delta_i := \frac{5 \cdot \left(w_{STRI.LL_i}\right) \cdot \left(L_{span_i}\right)^4}{384 \cdot E_i \cdot I_i}$ $\quad \Delta^T = (0.10 \quad 0.10 \quad 0.10) \cdot in$

$\Delta_{limit} := \frac{L_{span}}{360}$ $\quad {\Delta_{limit}}^T = (0.12 \quad 0.12 \quad 0.12) \cdot in$ (BDM 1.8.2)

$\Delta_{check_i} := \text{If}\left(\Delta_i \le \Delta_{limit_i}, \text{"OK"}, \text{"NG"}\right)$ $\quad {\Delta_{check}}^T = (\text{"OK"} \quad \text{"OK"} \quad \text{"OK"})$

$Deflection_{RF_i} := \frac{\Delta_{limit_i}}{\Delta_i}$ $\quad {Deflection_{RF}}^T = (1.25 \quad 1.25 \quad 1.25)$

Shear

$V_{u.DC_i} := w_{STRI.DC_i} \cdot \left(\frac{L_{span_i}}{2} - h_{g_i}\right)$ $\quad {V_{u.DC}}^T = (12.2 \quad 6.0 \quad 3.8) \cdot lbf$

$V_{u.LL_i} := w_{STRI.LL_i} \cdot \left(\frac{L_{span_i}}{2} - h_{g_i}\right)$ $\quad {V_{u.LL}}^T = (0.25 \quad 0.12 \quad 0.08) \cdot kip$

$V_{u_i} := V_{u.DC_i} + V_{u.LL_i}$ $\quad {V_u}^T = (0.26 \quad 0.13 \quad 0.08) \cdot kip$

$Shear_{Ratio_i} := \frac{\phi_s \cdot V_{n_i}}{V_{u_i}}$ $\quad {Shear_{Ratio}}^T = (12.54 \quad 12.54 \quad 12.54)$

$RF_{V.u_i} := \frac{\Phi_V \cdot \left(V_{n_i}\right) - V_{u.DC_i}}{V_{u.LL_i}}$ $\quad {RF_{V.u}}^T = (12.46 \quad 12.46 \quad 12.46)$

Note: Rating factors for all members > 1.0. Say OK
C/D ratios for all members > 1.0. Say OK

Bearing

$$R_{u.DC_i} := W_{STRI.DC_i} \cdot \frac{L_{span_i}}{2} \qquad R_{u.DC}^{T} = (13.12 \quad 6.42 \quad 4.08) \cdot lbf$$

$$R_{u.LL_i} := W_{STRI.LL_i} \cdot \frac{L_{span_i}}{2} \qquad R_{u.LL}^{T} = (0.26 \quad 0.13 \quad 0.08) \cdot kip$$

$$R_{u_i} := R_{u.DC_i} + R_{u.LL_i} \qquad R_u^{T} = (0.28 \quad 0.14 \quad 0.09) \cdot kip$$

$$Bearing_{Ratio_i} := \frac{\phi_{cp} \cdot R_{n_i}}{R_{u_i}} \qquad Bearing_{Ratio}^{T} = (119.11 \quad 119.11 \quad 119.11)$$

$$RF_{R.u_i} := \frac{\Phi_b \cdot \left(R_{n_i}\right) - R_{u.DC_i}}{R_{u.LL_i}} = \ldots \qquad RF_{R.u}^{T} = (118.72 \quad 118.72 \quad 118.72)$$

Note: Rating factors for all members > 1.0. Say OK
C/D ratios for all members > 1.0. Say OK

Bridge Type: Timber and Glulam pedestrian bidge.simple supports for dead load and live load.

CAPS ANALYSIS:

References:
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ODOT LRFR Manual June 2018 (ODOT)
ODOT Bridge Design Manual (BDM)
AISC Steel Construction Manual (AISC)

LRFR Strength Limit State:

$$RF = \frac{\phi_c \cdot \phi_s \cdot \phi \cdot (R_n) - (\gamma_{DC}) \cdot (DC) - (\gamma_{DW}) \cdot (DW)}{(\gamma_L) \cdot (LL + IM)}$$ (MBE 6A.4.2.1-1)

Resistance Factors:

$\phi_s := 0.75$	LRFD resistance factor for shear	(AASHTO 8.5.2.2)
$\phi_f := 0.85$	LRFD resistance factor for flexure	(AASHTO 8.5.2.2)
$\phi_{cp} := 0.90$	LRFD resistance factor for compression perpendicular to grain.	(AASHTO 8.5.2.2)
$\phi_{cl} := 0.90$	LRFD resistance factor for compression parallel to grain.	(AASHTO 8.5.2.2)
$\phi_c := 0.95$	Condition factor for caps condition rating = 5 (Fair)	(MBE T. 6A.4.2.3-1)
$\phi_{sf} := 1.00$	System Factor for Flexure, structure type: "Timber Stringers"	(ODOT 1.4.1.4)
$\phi_{sv} := 1.00$	System Factor for Shear, structure type: "Timber Stringers"	(ODOT 1.4.1.4)
$\phi_{sa} := 1.00$	System Factor for Axial, All other girder bridges and slab bridges	(MBE 6A.4.2.4-1)

Combined Resistance Factors:

For Flexure: $\Phi_f := \phi_f \cdot (\max(\phi_c \cdot \phi_{sf}, 0.85))$ $\Phi_f = 0.807$

For Shear: $\Phi_v := \phi_s \cdot (\max(\phi_c \cdot \phi_{sv}, 0.85))$ $\Phi_v = 0.712$

For Bearing $\Phi_b := \phi_{cp} \cdot (\max(\phi_c \cdot \phi_{sa}, 0.85))$ $\Phi_b = 0.855$

(Note: $\phi_c \phi_s \geq 0.85$ per MBE 6A.4.2.1-3)

Load Factors:

Dead Load Factors γ_{DC}:

$\gamma_{DC.max} := 1.25$ max. MBE T. 6A.4.2.2-1 for structural components and attachments STR I

Live Load Factors γ_L:

$\gamma_{LL} := 1.750$ MBE T. 6A.4.2.2-1, assume pedestrian loading as Inventory

Dynamic load allowance, IM, is not required with pedestrian loading, PED 3.1

Bridge Members

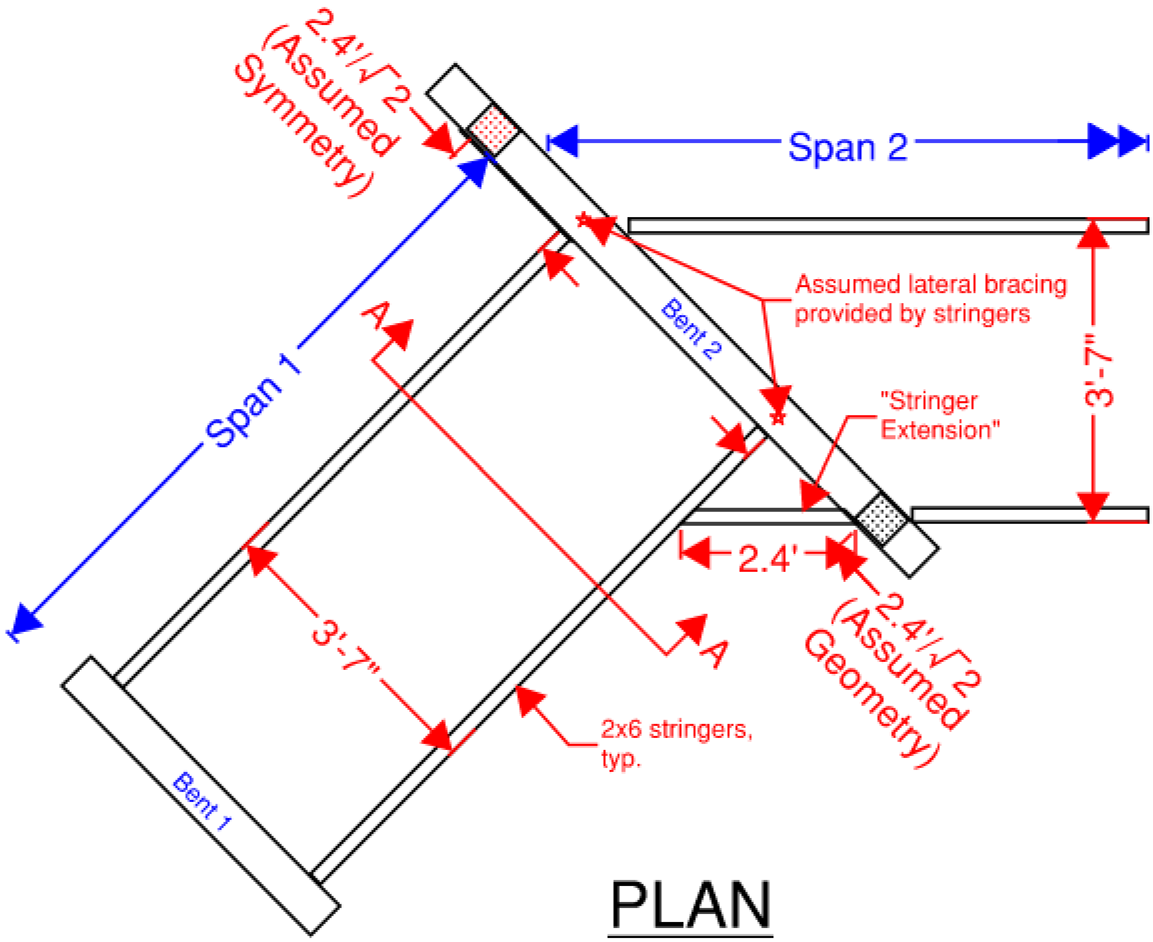
Analysis below shall consist of cap members:

1) 4x6 cap (3'-7" span)
2) 4.6 cap (Bent 2)
3) 6x8 cap (Bearing only)

Assume Bent 2 span length = 4'-0" + 2*(2.4' / √2) = 7.4 ft

Per field inspection, 2.4' 4x6 stringer extension makes approx right triangle geometry at inside corner. Assume eqal distance offset on both ends of bent 2 out from width of deck.

See sketch below. Section A is shown in "Analysis" section.



PLAN

Bridge Geometry:

Deck Out to Out: $W_{total} := 4ft + 0in = 4.00\,ft$

$S_{s.oto} := 3 \cdot ft + 7 \cdot in$ Spacing stringers, out to out dimension

Path Width: $W_{path} := S_{s.oto} - 0.5 \cdot 2 \cdot (1.5 \cdot in) = 3.46\,ft$

Span: $L_{span} := \left(S_{s.oto} \quad 7.4 \cdot ft \quad S_{s.oto} \right)^T$

$i := 1 .. length\left(L_{span}\right)$ Set counter

Member Height: $h_g := (5.5 \quad 5.5 \quad 7.25)^T in$

Member Width: $b_g := (3.5 \quad 3.5 \quad 5.5)^T in$

Length of bearing: $L_b := (3.5 \quad 3.5 \quad 5.5)^T \cdot in$

Decking bearing controlled by 4x6 stringers.
4x6 caps supported by 4x4 posts
6x8 caps assume 5.5x5.5 support on top of concrete piers.

Deck Thickness: $t_f := 1.5in$ 2"x12" / 2"x6" / 2"x4" timber decking

Left Cantilever: $L_{Cant} := 0.5 \cdot \left(W_{total} - S_{s.oto}\right) = 0.21\,ft$

Right Cantilever: $R_{Cant} := L_{Cant} = 0.21\,ft$

Material Properties:

$w_t := 0.050kcf$ (AASHTO Table 3.5.1-1) (Assumed timber species of douglas fir is softwood)

Dimensional Lumber (L) (Douglas Fir, Dimension ≥ 2 in. wide, Select Structural), per ODOT 8.2.4

"Select Structural" grade assumed here as values provided in ODOT LRFR match AASHTO Select Structural

$F_{bo} := 1.5ksi$ (ODOT 8.2.4) Bending stress

$F_{vo} := 0.180ksi$ (ODOT 8.2.4) Shear parallel to grain

$F_{cpo} := 0.625ksi$ (ODOT 8.2.4) Compression perpendicular to grain

$F_{co} := 1.700ksi$ (AASHTO Table 8.4.1.1.4-1) Compression parallel to grain

$E_o := 1900ksi$ (ODOT 8.2.4) Mod. of Elasticity

Adjustment Factors (AASHTO 8.4.4)

$C_M := 1.0$ (Wet service factor for Glu-Lam less than 16%, and sawn lumber less than 19%, ODOT 8.2.4.3). Unless submerged, timber is considered dry (BDM 1.8.2)

$C_F := 1.0$ Size factor, (ODOT 8.2.4.4)

$C_{fu} := 1.0$ (Flat-use factor, ODOT 8.2.4.6)

$C_{i.E} := 0.95$ (Incising factor, for E_o, ODOT 8.2.4.6)

$C_i := 0.80$ Incising factor, for F.bo and F.von (ODOT 8.2.4.7), & F.to and F.co (AASHTO T. 8.4.4.7-1)

$C_{i.cpo} := 1.0$ Incising factor, for F_{cpo} (ODOT 8.2.4.7)

$C_d := 1.0$ Deck factor (ODOT 8.2.4.8)

$C_{\lambda.1} := 0.8$ Time effect factor, Strength Limit State 1 (ODOT 8.2.4.9)

$E_i := (E_o)\cdot(C_M)\cdot(C_{i.E})$ $\quad E^T = (1805 \quad 1805 \quad 1805)\cdot ksi$ (ODOT 8.2.4.1)

$C_{KF.f} := \dfrac{2.5}{\phi_f} = 2.94$ Format conversion factor, F.b (ODOT 8.2.4.2)

$C_{KF.s} := \dfrac{2.5}{\phi_s} = 3.33$ Format conversion factor, F.v (ODOT 8.2.4.2)

$C_{KF.cp} := \dfrac{2.1}{\phi_{cp}} = 2.33$ Format conversion factor, F.cp (compression perpendicular to grain.) (ODOT 8.2.4.2)

$F_{b_i} := (F_{bo})\cdot(C_{KF.f})\cdot(C_M)\cdot(C_{fu})\cdot(C_i)\cdot(C_d)\cdot C_{\lambda.1}$ (ODOT 8.2.4.1)

$F_b^T = (2.82 \quad 2.82 \quad 2.82)\cdot ksi$

$F_{v_i} := (F_{vo})\cdot(C_{KF.s})\cdot(C_M)\cdot(C_i)\cdot(C_{\lambda.1})$ (ODOT 8.2.4.1)

$F_v^T = (0.38 \quad 0.38 \quad 0.38)\cdot ksi$

$F_c := (F_{co})\cdot(C_{KF.s})\cdot(C_M)\cdot(C_F)\cdot(C_i)\cdot(C_{\lambda.1}) = 3.63\cdot ksi$ (AASHTO 8.4.4.1-4)

$F_{cp_i} := (F_{cpo})\cdot(C_{KF.cp})\cdot(C_M)(C_i)(C_{\lambda.1})$ (ODOT 8.2.4.1)

$F_{cp}^T = (0.93 \quad 0.93 \quad 0.93)\cdot ksi$

Beam stability factor, C_L calculated below. F_b will be adjusted to account for C_L. Calculations below are based on AASHTO 8.6.2 / ODOT 8.2.4.10.

$Braced := \text{"No"}$ — "Yes" if compression side of beam is continuously braced and beam is braced laterally at supports, Else "No".

$K_{bE} := 0.76$ — (Euler buckling coefficient for visually graded lumber)

$L_u := L_{span}$ $\quad L_{u_2} := 3ft + 7in$ — Assume laterally braced at ends only. Bent 2 unbraced length is assumed between stringers

(ODOT 8.2.4.10)

$$L_{e_i} := \begin{cases} (2.06)\cdot\left(L_{u_i}\right) & \text{if } \dfrac{L_{u_i}}{h_{g_i}} < 7 \\ (1.63)\cdot\left(L_{u_i}\right) + (3)\cdot\left(h_{g_i}\right) & \text{if } 7 \le \dfrac{\left(L_{u_i}\right)}{h_{g_i}} \le 14.3 \\ (1.84)\cdot\left(L_{u_i}\right) & \text{otherwise} \end{cases}$$

$$L_e^T = (7.22 \quad 7.22 \quad 7.38)\,ft$$

$$R_{b_i} := \min\left[\sqrt{\frac{\left(L_{e_i}\right)\cdot\left(h_{g_i}\right)}{\left(b_{g_i}\right)^2}}, 50\right] \qquad R_b^T = (6.24 \quad 6.24 \quad 4.61)$$

(ODOT 8.2.4.10)

$$F_{bE_i} := \frac{\left(K_{bE}\right)\cdot\left(E_i\right)}{\left(R_{b_i}\right)^2} \qquad F_{bE}^T = (35.3 \quad 35.3 \quad 64.6)\cdot ksi$$

(ODOT 8.2.4.10)

$$A_i := \frac{F_{bE_i}}{F_{b_i}} \qquad A^T = (12.50 \quad 12.50 \quad 22.88)$$

(ODOT 8.2.4.10)

(Beam Stability factor, ODOT 8.2.4.10)

$$C_{L_i} := \begin{cases} 1 & \text{if } Braced = \text{"Yes"} \\ \dfrac{1 + A_i}{1.9} - \sqrt{\dfrac{(1 + A_i)^2}{3.61} - \dfrac{A_i}{0.95}} & \text{otherwise} \end{cases}$$

$$C_L^T = (1.00 \quad 1.00 \quad 1.00)$$

$$F_{b_i} := \left(F_{b_i}\right)\cdot\left(C_{L_i}\right) \qquad F_b^T = (2.811 \quad 2.811 \quad 2.817)\cdot ksi$$

Capacities:

$$S_i := \frac{(b_{g_i}) \cdot (h_{g_i})^2}{6} \qquad S^T = (17.6 \quad 17.6 \quad 48.2) \cdot in^3$$

Section modulus

$$M_{n_i} := F_{b_i} \cdot S_i \qquad M_n^T = (4.1 \quad 4.1 \quad 11.3) \cdot kip \cdot ft$$

Nominal Moment Capacity, AASHTO 8.6.3-1

$$V_{n_i} := \frac{F_{v_i} \cdot b_{g_i} \cdot h_{g_i}}{1.5} \qquad V_n^T = (4.9 \quad 4.9 \quad 10.2) \cdot kip$$

Nominal Shear Capacity, AASHTO 8.7-2

$$A_{b_i} := (b_{g_i}) \cdot (L_{b_i}) \qquad A_b^T = (12.25 \quad 12.25 \quad 30.25) \cdot in^2$$

Bearing area

$$C_b := 1.0$$

Bearing adjustment factor, AASHTO 8.8.3

$$R_{n_i} := F_{cp_i} \cdot A_{b_i} \cdot C_b$$

Nominal compression capacity perpendicular to grain, AASHTO 8.8.3-1

$$R_n^T = (11.43 \quad 11.43 \quad 28.23) \cdot kip$$

Component Dead Loads (DC):

Deadload of decking

$$W_{deck} := (w_t) \cdot (t_f) \cdot (b_g) \qquad W_{deck}^T = (1.82 \quad 1.82 \quad 2.86) \cdot plf$$

Wearing Surface Dead Loads (DW):

N/A

Live Loads (LL):

Pedestrian Loading

$$W_{LL} := (90 \cdot psf) \cdot (b_g) \qquad W_{LL}^T = (26.25 \quad 26.25 \quad 41.25) \cdot plf$$

Analysis Sections:

Spans are simply supported. Max bending moment assumed at mid span. Max shear assumed at a distance from face of support equal to depth of the component (per AASHTO 8.7). Max bearing assumed at location of bearing. For decking, neglect overhangs as they reduce max positive moment.

Distributed loads

$w_{STRI.DC_i} := \gamma_{DC.max} \cdot \left(w_{deck_i}\right)$ $\quad w_{STRI.DC}^T = (2.28 \quad 2.28 \quad 3.58) \cdot plf$

$w_{STRI.LL_i} := \gamma_{LL} \cdot \left(w_{LL_i}\right)$ $\quad w_{STRI.LL}^T = (0.05 \quad 0.05 \quad 0.07) \cdot klf$

$w_{STRI_i} := w_{STRI.DC_i} + w_{STRI.LL_i}$ $\quad w_{STRI}^T = (0.048 \quad 0.048 \quad 0.076) \cdot klf$

4x6 bents will bear the tributary span loads. Distributed demands are copied from Stringer calcs.

$w_{STRI.DC.Spans} := 27.37\ plf$ Superstructure DC loading on 4x6 spans, per Stringer, per Stringer calcs.

$w_{STRI.LL.Spans} := 272.34\ plf$ Superstructure LL loading on 4x6 spans, per Stringer, per Stringer calcs.

$V_{u.DC.2} := 1.34 \cdot kip$ Shear demand at bent 2 per stringer of span 1, per stringer calcs

$V_{u.LL.2} := 1.34 \cdot kip$ Shear demand at bent 2 per stringer of span 1, per stringer calcs

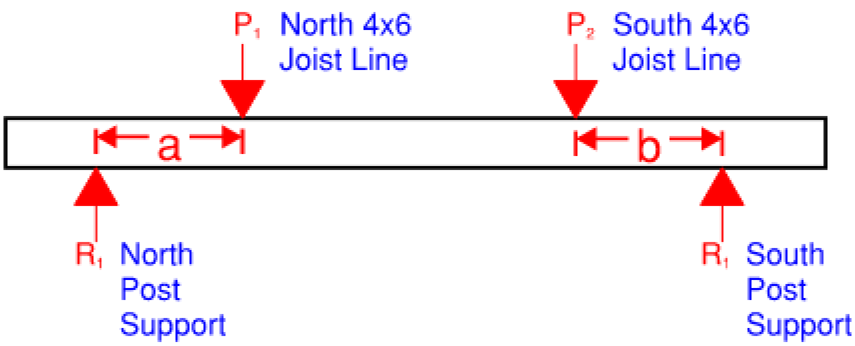
$w_{rail} := 8.36 \cdot plf$ Distributed load of railing with 4x4 posts at 8'-0" spacing, ave.

$V_{u.DC.2.addl.Rail.N} := \gamma_{DC.max} \cdot w_{rail} \cdot (0.5 \cdot 8.2 + 4)ft = 84.64\ lbf$ Additional DC at bent 2 from rail posts at bent 2 along north stringer line.

$V_{u.DC.2.addl.Rail.S} := \gamma_{DC.max} \cdot w_{rail} \cdot (0.5 \cdot 8.2)ft = 42.84\ lbf$ Additional DC at bent 2 from rail posts at bent 2 along south stringer line. (Span 2 south stringer frames aligned with post and does not load bent 2 cap span).

Flexure

Bent 2's 4x6 cap includes span 1 stringers tying into the cap in span, and the outside stringer of span 2 tying into the cap in span. AISC Table 3-23 item 11 will be used to evaluate bending moment at mid span, and shear.



Section A-A/Load Diagram

$$a_1 := \frac{2.4 \cdot ft}{\sqrt{2}} = 1.70\,ft \qquad b_2 := a_1$$

$$P_{1.DC} := V_{u.DC.2} + V_{u.DC.2.addl.Rail.N} + W_{STRI.DC.Spans} \cdot 0.5 \cdot 12ft = 1.59 \cdot kip$$

$$P_{1.LL} := V_{u.LL.2} + W_{STRI.LL.Spans} \cdot 0.5 \cdot 12ft = 2.97 \cdot kip$$

$$P_{2.DC} := V_{u.DC.2} + V_{u.DC.2.addl.Rail.N} = 1.42 \cdot kip$$

$$P_{2.LL} := V_{u.LL.2} = 1.34 \cdot kip$$

$$M_{max.DC} := 0.5\left[\left[\frac{P_{1.DC} \cdot (L_{span_2} - a_1) + P_{2.DC} \cdot b_2}{L_{span_2}} \cdot a_1\right] + \left[\frac{P_{1.DC} \cdot a_1 + P_{2.DC} \cdot (L_{span_2} - b_2)}{L_{span_2}} \cdot b_2\right]\right]$$

$$M_{max.LL} := 0.5\left[\left[\frac{P_{1.LL} \cdot (L_{span_2} - a_1) + P_{2.LL} \cdot b_2}{L_{span_2}} \cdot a_1\right] + \left[\frac{P_{1.LL} \cdot a_1 + P_{2.LL} \cdot (L_{span_2} - b_2)}{L_{span_2}} \cdot b_2\right]\right]$$

$$M_{u.DC.ref} := \begin{pmatrix} 0 & M_{max.DC} & 0 \end{pmatrix}^T \qquad M_{u.DC.ref}^T = \begin{pmatrix} 0.00 & 2.56 & 0.00 \end{pmatrix} \cdot kip \cdot ft$$

$$M_{u.LL.ref} := \begin{pmatrix} 0 & M_{max.LL} & 0 \end{pmatrix}^T \qquad M_{u.LL.ref}^T = \begin{pmatrix} 0.00 & 3.66 & 0.00 \end{pmatrix} \cdot kip \cdot ft$$

$$M_{u.DC_i} := \frac{w_{STRI.DC_i} \cdot \left(L_{span_i}\right)^2}{8} + M_{u.DC.ref_i} \qquad M_{u.DC}^T = (3.7 \quad 2572.6 \quad 5.7) \cdot lbf \cdot ft$$

$$M_{u.LL_i} := \frac{w_{STRI.LL_i} \cdot \left(L_{span_i}\right)^2}{8} + M_{u.LL.ref_i} \qquad M_{u.LL}^T = (0.07 \quad 3.98 \quad 0.12) \cdot kip \cdot ft$$

$$M_{u_i} := M_{u.DC_i} + M_{u.LL_i} \qquad M_u^T = (0.08 \quad 6.55 \quad 0.12) \cdot kip \cdot ft$$

$$Bending_{Ratio_i} := \frac{\phi_f \cdot M_{n_i}}{M_{u_i}} \qquad Bending_{Ratio}^T = (45.41 \quad 0.54 \quad 79.06)$$

$$RF_{M.u_i} := \frac{\Phi_f \cdot \left(M_{n_i}\right) - M_{u.DC_i}}{M_{u.LL_i}} \qquad RF_{M.u}^T = (45.23 \quad 0.19 \quad 78.78)$$

Note: 4x6 cap at bent 2 would experience partial distributed loading at mid span (i.e. between loads P.1 and P.2). Conservatively analyzed here with full span (R.1 to R.2) distributed loading for simplicity (Distributed live load accounts for ~1% of total moment compared to joist loading).

6x8 cap would not experience distributed loading. Conservatively analyzed here.

Rating factors for all members other than 4x6 cap at bent 2 > 1.0. Say OK for flexure
Rating factors for 4x6 bent 2 cap < 1.0. No goodfor flexure
Bent 7 appears similar to bent 2. Assume no good. for flexure

C/D ratios for all members other than 4x6 cap at bent 2 > 1.0. Say OK for flexure
C/D ratios for 4x6 bent 2 cap < 1.0. No good. for flexure
Bent 7 appears similar to bent 2. Assume no good. for flexure

Shear

$$V_{u.DC.ref} := \left[0 \quad \frac{P_{1.DC} \cdot \left(L_{span_2} - a_1\right) + P_{2.DC} \cdot b_2}{L_{span_2}} \quad 0 \right]^T \qquad V_{u.DC.ref}{}^T = (0 \quad 1.55 \quad 0) \cdot kip$$

$$V_{u.LL.ref} := \left[0 \quad \frac{P_{1.LL} \cdot \left(L_{span_2} - a_1\right) + P_{2.LL} \cdot b_2}{L_{span_2}} \quad 0 \right]^T \qquad V_{u.LL.ref}{}^T = (0 \quad 2.6 \quad 0) \cdot kip$$

$$V_{u.DC_i} := W_{STRI.DC_i} \cdot \left(\frac{L_{span_i}}{2} - h_{g_i} \right) + V_{u.DC.ref_i} \qquad V_{u.DC}{}^T = (3.0 \quad 1558.6 \quad 4.3) \cdot lbf$$

$$V_{u.LL_i} := W_{STRI.LL_i} \cdot \left(\frac{L_{span_i}}{2} - h_{g_i} \right) + V_{u.LL.ref_i} \qquad V_{u.LL}{}^T = (0.06 \quad 2.75 \quad 0.09) \cdot kip$$

$$V_{u_i} := V_{u.DC_i} + V_{u.LL_i} \qquad V_u{}^T = (0.06 \quad 4.31 \quad 0.09) \cdot kip$$

$$Shear_{Ratio_i} := \frac{\phi_s \cdot V_{n_i}}{V_{u_i}} \qquad Shear_{Ratio}{}^T = (57.49 \quad 0.86 \quad 85.09)$$

$$RF_{V.u_i} := \frac{\Phi_V \cdot \left(V_{n_i}\right) - V_{u.DC_i}}{V_{u.LL_i}} \qquad RF_{V.u}{}^T = (57.28 \quad 0.71 \quad 84.80)$$

Note: 4x6 cap at bent 2 would experience partial distributed loading at mid span (i.e. between loads P.1 and P.2). Conservatively analyzed here with full span (R.1 to R.2) distributed loading for simplicity (Distributed live load accounts for ~1% of total moment compared to joist loading).

6x8 cap would not experience distributed loading. Conservatively analyzed here.

Rating factors for all members other than 4x6 cap at bent 2 > 1.0. Say OK for shear
Rating factors for 4x6 bent 2 cap < 1.0. No good for shear
Bent 7 appears similar to bent 2. Assume no good. for shear

C/D ratios for all members other than 4x6 cap at bent 2 > 1.0. Say OK for shear
C/D ratios for 4x6 bent 2 cap < 1.0. No good. for shear
Bent 7 appears similar to bent 2. Assume no good for shear

Bearing

$Wt_{Adj.DC} := V_{u.DC.ref}$ $\qquad Wt_{Adj.DC}^{T} = (0.00 \quad 1.55 \quad 0.00)\cdot kip$

$Wt_{Adj.LL} := V_{u.LL.ref}$ $\qquad Wt_{Adj.LL}^{T} = (0.00 \quad 2.60 \quad 0.00)\cdot kip$

$Wt_{adj.6x8.DC} := 1.59\cdot kip$ STR DC Bearing loading on 21.75" glulams, per Stringer calcs.

$Wt_{adj.6x8.LL} := 6.84\cdot kip$ STR LL Bearing loading on 21.75" glulams, per Stringer calcs.

$Wt_{Adj.DC_6} := Wt_{adj.6x8.DC}$ $\qquad Wt_{Adj.DC}^{T} = (0.00 \quad 1.55 \quad 0.00 \quad 0.00 \quad 0.00 \quad 1.59)\cdot kip$

$Wt_{Adj.LL_6} := Wt_{adj.6x8.LL}$ $\qquad Wt_{Adj.LL}^{T} = (0.00 \quad 2.60 \quad 0.00 \quad 0.00 \quad 0.00 \quad 6.84)\cdot kip$

$R_{u.DC_i} := W_{STRI.DC_i}\cdot\frac{L_{span_i}}{2} + Wt_{Adj.DC_i}$ $\qquad R_{u.DC}^{T} = (4.08 \quad 1559.64 \quad 6.42)\cdot lbf$

$R_{u.LL_i} := W_{STRI.LL_i}\cdot\frac{L_{span_i}}{2} + Wt_{Adj.LL_i}$ $\qquad R_{u.LL}^{T} = (0.08 \quad 2.77 \quad 0.13)\cdot kip$

$R_{u_i} := R_{u.DC_i} + R_{u.LL_i}$ $\qquad R_{u}^{T} = (0.09 \quad 4.33 \quad 0.14)\cdot kip$

$Bearing_{Ratio_i} := \frac{\phi_{cp}\cdot R_{n_i}}{R_{u_i}}$ $\qquad Bearing_{Ratio}^{T} = (119.11 \quad 2.38 \quad 187.18)$

$RF_{R.u_i} := \frac{\Phi_b\cdot\left(R_{n_i}\right) - R_{u.DC_i}}{R_{u.LL_i}} = ...$ $\qquad RF_{R.u}^{T} = (118.72 \quad 2.97 \quad 186.59)$

Note: Rating factors for all members > 1.0. Say OK for bearing
C/D ratios for all members > 1.0. Say OK for bearing

Bridge Type: Timber and Glulam pedestrian bidge.simple supports for dead load and live load.

RAILING ANALYSIS:

References: AASHTO LRFD BDS 9th Ed. 2020 (AASHTO)
AASHTO MBE 3rd E. 2018 with 2019 Interims (MBE)
AASTHO LRFD Guide Spec for the Design of Pedestrian Bridges 2009 w/ 2015 Interims (PED)
ODOT LRFR Manual June 2018 (ODOT)
ODOT Bridge Design Manual (BDM)

LRFR Strength Limit State:

$$RF = \frac{\phi_c \cdot \phi_s \cdot \phi \cdot (R_n) - (\gamma_{DC}) \cdot (DC) - (\gamma_{DW}) \cdot (DW)}{(\gamma_L) \cdot (LL + IM)}$$ (MBE 6A.4.2.1-1)

Resistance Factors:

$\phi_s := 0.75$	LRFD resistance factor for shear	(AASHTO 8.5.2.2)
$\phi_f := 0.85$	LRFD resistance factor for flexure	(AASHTO 8.5.2.2)
$\phi_{cp} := 0.90$	LRFD resistance factor for compression perpendicular to grain.	(AASHTO 8.5.2.2)
$\phi_{cl} := 0.90$	LRFD resistance factor for compression parallel to grain.	(AASHTO 8.5.2.2)
$\phi_c := 0.95$	Condition factor for railing/post condition rating = 5 (Fair)	(MBE T. 6A.4.2.3-1)
$\phi_{sf} := 1.00$	System Factor for Flexure, structure type: "Timber Stringers"	(ODOT 1.4.1.4)
$\phi_{sv} := 1.00$	System Factor for Shear, structure type: "Timber Stringers"	(ODOT 1.4.1.4)
$\phi_{sa} := 1.00$	System Factor for Axial, All other girder bridges and slab bridges	(MBE 6A.4.2.4-1)

Combined Resistance Factors:

For Flexure: $\Phi_f := \phi_f \cdot (\max(\phi_c \cdot \phi_{sf}, 0.85))$ $\Phi_f = 0.807$ (Note: $\phi_c \phi_s \geq 0.85$

For Shear: $\Phi_v := \phi_s \cdot (\max(\phi_c \cdot \phi_{sv}, 0.85))$ $\Phi_v = 0.712$ per MBE 6A.4.2.1-3)

Load Factors:

Dead Load Factors γ_{DC}:

$\gamma_{DC.max} := 1.25$ max. MBE T. 6A.4.2.2-1 for structural components and attachments STR I

Live Load Factors γ_L:

$\gamma_{LL} := 1.750$ MBE T. 6A.4.2.2-1, assume pedestrian loading as Inventory

Dynamic load allowance, IM, is not required with pedestrian loading, PED 3.1

Bridge Members Analysis below shall consist of railing members:

1) 4x4 rail posts
2) Long. rail members (2x4)
    2a) X-axis (Vert, along narrow face)
    2b) Y-axis (Horiz, along wide face)

Bridge Geometry:

Member Height: $h_g := (3.5 \;\; 3.5 \;\; 1.5)^T \text{ in}$

Member Width: $b_g := (3.5 \;\; 1.5 \;\; 3.5)^T \text{ in}$

Deck Thickness: $t_f := 1.5\text{in}$ 2"x12" / 2"x6" / 2"x4" timber decking

Span: $L_{span} := \left[(40 \cdot \text{in} + t_f + 0.5 \cdot 5.5 \cdot \text{in}) \;\; 8 \cdot \text{ft} \;\; 8 \cdot \text{ft}\right]^T$ $L_{span}{}^T = (3.69 \;\; 8.00 \;\; 8.00)\text{ ft}$

$i := 1 .. \text{length}(L_{span})$ Set counter

Material Properties:

$w_t := 0.050\text{kcf}$ (AASHTO Table 3.5.1-1) (Assumed timber species of douglas fir is softwood)

Dimensional Lumber (L) (Douglas Fir, Dimension ≥ 2 in. wide, Select Structural), per ODOT 8.2.4

"Select Structural" grade assumed here as values provided in ODOT LRFR match AASHTO Select Structural

$F_{bo} := 1.5\text{ksi}$ (ODOT 8.2.4) Bending stress

$F_{vo} := 0.180\text{ksi}$ (ODOT 8.2.4) Shear parallel to grain

$F_{cpo} := 0.625\text{ksi}$ (ODOT 8.2.4) Compression perpendicular to grain

$F_{co} := 1.700\text{ksi}$ (AASHTO Table 8.4.1.1.4-1) Compression parallel to grain

$E_o := 1900\text{ksi}$ (ODOT 8.2.4) Mod. of Elasticity

Dimensional Lumber (L) (Douglas Fir, Posts and Timber, Select Structural),

"Select Structural" grade assumed here as values provided in ODOT LRFR match AASHTO Select Structural

$F_{bo.p} := F_{bo}$ (AASHTO T. 8.4.1.1.4-1) Bending stress

$F_{vo.p} := 0.170\text{ksi}$ (AASHTO T. 8.4.1.1.4-1) Shear parallel to grain

$F_{cpo.p} := F_{cpo}$ (AASHTO T. 8.4.1.1.4-1) Compression perpendicular to grain

$F_{co.p} := 1.150\text{ksi}$ (AASHTO Table 8.4.1.1.4-1) Compression parallel to grain

$E_{o.p} := 1600\text{ksi}$ (AASHTO T. 8.4.1.1.4-1) Mod. of Elasticity

$$F_{vo.ref} := (F_{vo.p} \;\; F_{vo} \;\; F_{vo})^T$$

$$F_{co.ref} := (F_{co.p} \;\; F_{co} \;\; F_{co})^T$$

$$E_{o.ref} := (E_{o.p} \;\; E_o \;\; E_o)^T$$

Adjustment Factors (AASHTO 8.4.4)

$C_M := 1.0$ (Wet service factor for Glu-Lam less than 16%, and sawn lumber less than 19%, ODOT 8.2.4.3). Unless submerged, timber is considered dry (BDM 1.8.2)

$C_F := 1.0$ Size factor, (ODOT 8.2.4.4)

$C_{F.Fbo} := 0.86$ Size factor, loads applied to wide face, (AASHTO T. 8.4.4.4-2)

$C_{F.Eo} := C_F$

$C_{F.o} := C_F$

$C_{fu} := (1.0 \quad 1.0 \quad 1.10)^T$ (Flat-use factor, ODOT 8.2.4.6 & AASHTO T. 8.4.4.6-1)

$C_{i.E} := 0.95$ (Incising factor, for E_o, ODOT 8.2.4.6)

$C_i := 0.80$ Incising factor, for F.bo and F.von (ODOT 8.2.4.7), & F.to and F.co (AASHTO T. 8.4.4.7-1)

$C_{i.cpo} := 1.0$ Incising factor, for F_{cpo} (ODOT 8.2.4.7)

$C_d := 1.0$ Deck factor (ODOT 8.2.4.8)

$C_{\lambda.1} := 0.8$ Time effect factor, Strength Limit State 1 (ODOT 8.2.4.9)

$E := (E_o) \cdot (C_M) \cdot (C_{i.E}) = 1805 \cdot ksi$ (ODOT 8.2.4.1)

$E_p := (E_{o.p}) \cdot (C_M) \cdot (C_{i.E}) = 1520 \cdot ksi$ (AASHTO 8.4.4.1-6)

$C_{KF.f} := \frac{2.5}{\phi_f} = 2.94$ Format conversion factor, F.b (ODOT 8.2.4.2)

$C_{KF.s} := \frac{2.5}{\phi_s} = 3.33$ Format conversion factor, F.v (ODOT 8.2.4.2)

$C_{F.Fb.ref} := (C_F \quad C_F \quad C_{F.Fbo})^T$ (ODOT 8.2.4.1)

$F_{b_i} := (F_{bo}) \cdot (C_{KF.f}) \cdot (C_M) \cdot (C_{F.Fb.ref_i}) \cdot (C_{fu_i}) \cdot (C_i) \cdot (C_d) \cdot C_{\lambda.1}$ $\quad F_b^T = (2.82 \quad 2.82 \quad 2.67) \cdot ksi$

$F_v := (F_{vo.ref}) \cdot (C_{KF.s}) \cdot (C_M) \cdot (C_i) \cdot (C_{\lambda.1})$ $\quad F_v^T = (0.36 \quad 0.38 \quad 0.38) \cdot ksi$ (ODOT 8.2.4.1)

Beam stability factor, C_L calculated below. F_b will be adjusted to account for C_L. Calculations below are based on AASHTO 8.6.2 / ODOT 8.2.4.10.

Braced := "No" — "Yes" if compression side of beam is continuously braced and beam is braced laterally at supports, Else "No".

$K_{bE} := 0.76$ — (Euler buckling coefficient for visually graded lumber)

$L_{u.ref} := (2 \;\; 1 \;\; 1)^T$

$L_{u_i} := L_{u.ref_i} L_{span_i}$ — Assume posts laterally braced base, and horz. members at ends only

(ODOT 8.2.4.10)

$$L_{e_i} := \begin{cases} (2.06)\cdot(L_{u_i}) & \text{if } \dfrac{L_{u_i}}{h_{g_i}} < 7 \\ (1.63)\cdot(L_{u_i}) + (3)\cdot(h_{g_i}) & \text{if } 7 \le \dfrac{(L_{u_i})}{h_{g_i}} \le 14.3 \\ (1.84)\cdot(L_{u_i}) & \text{otherwise} \end{cases}$$

$L_e^T = (13.57 \;\; 14.72 \;\; 14.72)\,ft$

$$R_{b_i} := \min\left[\sqrt{\frac{(L_{e_i})\cdot(h_{g_i})}{(b_{g_i})^2}}, 50\right]$$

$R_b^T = (6.82 \;\; 16.58 \;\; 4.65)$ (ODOT 8.2.4.10)

$$F_{bE_i} := \frac{(K_{bE})\cdot(E)}{(R_{b_i})^2} \qquad F_{bE_1} := \frac{(K_{bE})\cdot(E_p)}{(R_{b_1})^2}$$

$F_{bE}^T = (24.8 \;\; 5.0 \;\; 63.4)\cdot ksi$ (ODOT 8.2.4.10)

$$A_i := \frac{F_{bE_i}}{F_{b_i}}$$

$A^T = (8.79 \;\; 1.77 \;\; 23.74)$ (ODOT 8.2.4.10)

(Beam Stability factor, ODOT 8.2.4.10)

$$C_{L_i} := \begin{cases} 1 & \text{if Braced = "Yes"} \\ \dfrac{1 + A_i}{1.9} - \sqrt{\dfrac{(1 + A_i)^2}{3.61} - \dfrac{A_i}{0.95}} & \text{otherwise} \end{cases}$$

$C_L^T = (0.99 \;\; 0.95 \;\; 1.00)$

$$F_{b_i} := (F_{b_i})\cdot(C_{L_i})$$

$F_b^T = (2.806 \;\; 2.670 \;\; 2.665)\cdot ksi$

Capacities:

$S_i := \frac{(b_{g_i}) \cdot (h_{g_i})^2}{6}$ $S^T = (7.1 \quad 3.1 \quad 1.3) \cdot in^3$ Section modulus

$M_{n_i} := F_{b_i} \cdot S_i$ $M_n^T = (1.67 \quad 0.68 \quad 0.29) \cdot kip \cdot ft$ Nominal Moment Capacity, AASHTO 8.6.3-1

$V_{n_i} := \frac{F_{v_i} \cdot b_{g_i} \cdot h_{g_i}}{1.5}$ $V_n^T = (2.96 \quad 1.34 \quad 1.34) \cdot kip$ Nominal Shear Capacity, AASHTO 8.7-2

Component Dead Loads (DC):

Deadload of components

$W_{DC_i} := (w_t) \cdot (h_{g_i}) \cdot (b_{g_i})$ $W_{DC}^T = (4.25 \quad 1.82 \quad 1.82) \cdot plf$

Wearing Surface Dead Loads (DW):

N/A

Live Loads (LL):

Pedestrian Loading

$W_{LL} := 0.050 \cdot klf$ Both transversely and vertically, acting simultaneously, AASHTO 13.8.2

$P_{LL} := 0.20 \cdot kip$ Concentrated load acting at any point in any direction, simultaneously with the distributed loads above, AASHTO 13.8.2

$P_{LL.Post} := P_{LL} + W_{LL} \cdot L_{span_3} = 0.60 \cdot kip$ Horiz. point load on post at height of top railing, AASHTO 13.8.2

Analysis Sections:

Railing spans are simply supported. Max bending moment assumed at mid span. Max shear assumed at face of support (difference in shear at location d.v from support, per AASHTO 8.7, is negligible).

Posts are assumed as fixed. Max bending moment and shear assumed at face of support.

Distributed loads

$W_{STRI.DC_i} := \gamma_{DC.max} \cdot (W_{DC_i})$ $W_{STRI.DC}^T = (5.32 \quad 2.28 \quad 2.28) \cdot plf$

$W_{STRI.LL} := \gamma_{LL} \cdot (W_{LL}) = 0.09 \cdot klf$

$P_{STRI.LL} := \gamma_{LL} \cdot (P_{LL}) = 0.35 \cdot kip$

$P_{STRI.LL.Post} := \gamma_{LL} \cdot (P_{LL.Post}) = 1.05 \cdot kip$

Flexure

$M_{LL.Post} := P_{LL.Post} \cdot L_{span_1} = 2.21 \cdot kip \cdot ft$ LL on post. No DC loading/negligible

$M_{DC.Rail.vert} := \dfrac{W_{DC_2} \cdot \left(L_{span_2}\right)^2}{8} = 0.015 \cdot kip \cdot ft$ DC vertical load on railing

$M_{LL.Rail.vert} := \dfrac{W_{LL} \cdot \left(L_{span_2}\right)^2}{8} = 0.40 \cdot kip \cdot ft$ Distributed LL horiz. on railing

$M_{LL.Rail.horiz} := M_{LL.Rail.vert}$ Distributed LL vert. on railing

$M_{LL.Rail.P.LL} := \dfrac{P_{LL} \cdot L_{span_2}}{4} = 0.40 \cdot kip \cdot ft$ Concentrated LL

$$Bending_{Ratio.Post} := \frac{\phi_f \cdot M_{n_1}}{M_{LL.Post}} = 0.64$$

$$Bending_{Ratio.Rail.vert} := \frac{\phi_f \cdot M_{n_2}}{M_{DC.Rail.vert} + M_{LL.Rail.vert} + M_{LL.Rail.P.LL}} = 0.71$$

Bending about X axis (vertical direction)

$$Bending_{Ratio.Rail.horiz} := \frac{\phi_f \cdot M_{n_3}}{M_{LL.Rail.horiz} + M_{LL.Rail.P.LL}} = 0.31$$

Bending about Y axis (horizontal direction)

$$Bending_{Combined} := \left(\frac{M_{DC.Rail.vert} + M_{LL.Rail.vert}}{\phi_f \cdot M_{n_2}} + \frac{M_{LL.Rail.horiz} + M_{LL.Rail.P.LL}}{\phi_f \cdot M_{n_3}}\right)^{-1} = 0.25$$

Combined loading ratio of X and Y axis bending, should be greater than 1.0
Note: Point load added to ratio of M_{n_3} controls by inspection.

$$RF_{M.Post} := \frac{\Phi_f \cdot M_{n_1}}{M_{LL.Post}} = 0.61$$

Rating factor of post

$$RF_{M.Rail.vert} := \frac{\Phi_f \cdot M_{n_2}}{M_{DC.Rail.vert} + M_{LL.Rail.vert} + M_{LL.Rail.P.LL}} = 0.68$$

Rating factor of railing about X axis (vertical direction)

$$RF_{M.Rail.horiz} := \frac{\Phi_f \cdot M_{n_3}}{M_{LL.Rail.horiz} + M_{LL.Rail.P.LL}} = 0.29$$

Rating factor of railing about Y axis (horizontal direction)

$$RF_{M.Rail.combined} := \left(\frac{M_{DC.Rail.vert} + M_{LL.Rail.vert}}{\Phi_f \cdot M_{n_2}} + \frac{M_{LL.Rail.horiz} + M_{LL.Rail.P.LL}}{\Phi_f \cdot M_{n_3}}\right)^{-1} = 0.24$$

Note: C/D ratios and Rating Factors for all members < 1.0. Say No Good for flexure

Shear

$V_{LL.Post} := P_{LL.Post} = 0.60 \cdot kip$ LL on post. No DC loading/negligible

$$V_{DC.Rail.vert} := \frac{W_{DC_2} \cdot \left(L_{span_2}\right)}{2} = 0.007 \cdot kip$$ DC vertical load on railing

$$V_{LL.Rail.vert} := \frac{W_{LL} \cdot \left(L_{span_2}\right)}{2} = 0.20 \cdot kip$$ Distributed LL horiz. on railing

$V_{LL.Rail.horiz} := V_{LL.Rail.vert}$ Distributed LL vert. on railing

$V_{LL.Rail.P.LL} := P_{LL} = 0.20 \cdot kip$ Concentrated LL

$$Shear_{Ratio.Post} := \frac{\phi_s \cdot V_{n_1}}{V_{LL.Post}} = 3.70$$

$$Shear_{Ratio.Rail.vert} := \frac{\phi_s \cdot V_{n_2}}{V_{DC.Rail.vert} + V_{LL.Rail.vert} + V_{LL.Rail.P.LL}} = 2.47$$ Bending about X axis (vertical direction)

$$Shear_{Ratio.Rail.horiz} := \frac{\phi_s \cdot V_{n_3}}{V_{LL.Rail.horiz} + V_{LL.Rail.P.LL}} = 2.52$$ Bending about Y axis (horizontal direction

Combined shear shall use the SRSS combination of shear capacity vs the shear demands of the longitudinal railing member. Point load added to ratio of V_{n_3} controls by inspection.

$$Shear_{Combined} := \sqrt{\left(\frac{\phi_s \cdot V_{n_2}}{V_{DC.Rail.vert} + V_{LL.Rail.vert}}\right)^2 + \left(\frac{\phi_s \cdot V_{n_3}}{V_{LL.Rail.horiz} + V_{LL.Rail.P.LL}}\right)^2} = 5.48$$

$$RF_{V.Post} := \frac{\Phi_v \cdot V_{n_1}}{V_{LL.Post}} = 3.52$$ Rating factor of post

$$RF_{V.Rail.vert} := \frac{\Phi_v \cdot V_{n_2} - V_{DC.Rail.vert}}{V_{LL.Rail.vert} + V_{LL.Rail.P.LL}} = 2.38$$ Rating factor of railing about X axis (vertical direction)

$$RF_{V.Rail.horiz} := \frac{\Phi_v \cdot V_{n_3}}{V_{LL.Rail.horiz} + V_{LL.Rail.P.LL}} = 2.39$$ Rating factor of railing about Y axis (horizontal direction)

$$RF_{VRail.Combined} := \sqrt{\left(\frac{\phi_s \cdot V_{n_2} - V_{DC.Rail.vert}}{V_{LL.Rail.vert}}\right)^2 + \left(\frac{\phi_s \cdot V_{n_3}}{V_{LL.Rail.horiz} + V_{LL.Rail.P.LL}}\right)^2} = 5.60$$

Note: C/D ratios and Rating Factors for all members > 1.0. Say OK for shear

Bridge Type: Timber and Glulam pedestrian bidge.simple supports for dead load and live load.

PIER POST ANALYSIS:

References:
AASHTO LRFD BDS 9th Ed. 2020 (AASHTO)
AASHTO MBE 3rd E. 2018 with 2019 Interims (MBE)
AASTHO LRFD Guide Spec for the Design of Pedestrian Bridges 2009 w/ 2015 Interims (PED)
ODOT LRFR Manual June 2018 (ODOT)
ODOT Bridge Design Manual (BDM)

LRFR Strength Limit State:

$$RF = \frac{\phi_c \cdot \phi_s \cdot \phi \cdot (R_n) - (\gamma_{DC}) \cdot (DC) - (\gamma_{DW}) \cdot (DW)}{(\gamma_L) \cdot (LL + IM)}$$ (MBE 6A.4.2.1-1)

Resistance Factors:

$\phi_s := 0.75$	LRFD resistance factor for shear	(AASHTO 8.5.2.2)
$\phi_f := 0.85$	LRFD resistance factor for flexure	(AASHTO 8.5.2.2)
$\phi_{cp} := 0.90$	LRFD resistance factor for compression perpendicular to grain.	(AASHTO 8.5.2.2)
$\phi_{cl} := 0.90$	LRFD resistance factor for compression parallel to grain.	(AASHTO 8.5.2.2)
$\phi_c := 1.0$	Condition factor for substructure (Column, Posts) rating = 6 (Good)	(MBE T. 6A.4.2.3-1)
$\phi_{sf} := 1.00$	System Factor for Flexure, structure type: "Timber Stringers"	(ODOT 1.4.1.4)
$\phi_{sv} := 1.00$	System Factor for Shear, structure type: "Timber Stringers"	(ODOT 1.4.1.4)
$\phi_{sa} := 1.00$	System Factor for Axial, All other girder bridges and slab bridges	(MBE 6A.4.2.4-1)

Combined Resistance Factors:

For Flexure: $\Phi_f := \phi_f \cdot (\max(\phi_c \cdot \phi_{sf}, 0.85))$ $\Phi_f = 0.850$

For Shear: $\Phi_v := \phi_s \cdot (\max(\phi_c \cdot \phi_{sv}, 0.85))$ $\Phi_v = 0.750$

For Axial: $\Phi_a := \phi_{cl} \cdot (\max(\phi_c \cdot \phi_{sa}, 0.85))$ $\Phi_a = 0.900$

(Note: $\phi_c \phi_s \geq 0.85$ per MBE 6A.4.2.1-3)

Load Factors:

Dead Load Factors γ_{DC}:

$\gamma_{DC.max} := 1.25$ max. MBE T. 6A.4.2.2-1 for structural components and attachments STR I

Live Load Factors γ_L:

$\gamma_{LL} := 1.750$ MBE T. 6A.4.2.2-1, assume pedestrian loading as Inventory

Dynamic load allowance, IM, is not required with pedestrian loading, PED 3.1

Bridge Members

Analysis below shall consist of timber post members:

1) 4x6 posts, Bent 2
2) 4x4 posts, Bent 1A

Bridge Geometry:

Member Height: $h_g := (5.5 \;\; 3.5)^T \, in$

$i := 1 .. length(h_g)$ Set counter

Member Width: $b_g := 3.5in$

Span: $L_{span} := 2.25 \cdot ft$ Conservatively assume 2'-3" column/post height (controlling on north side of bent 7). Bent 1A south side appears roughly 1'-10.5", north side looks similar; assume same length as at Bent 2, conservative.

Material Properties:

$w_t := 0.050kcf$ (AASHTO Table 3.5.1-1) (Assumed timber species of douglas fir is softwood)

Dimensional Lumber (L) (Douglas Fir, Posts and Timber, Select Structural),

"Select Structural" grade assumed here as values provided in ODOT LRFR match AASHTO Select Structural

$F_{bo} := 1.5ksi$ (AASHTO T. 8.4.1.1.4-1) Bending stress

$F_{vo} := 0.170ksi$ (AASHTO T. 8.4.1.1.4-1) Shear parallel to grain

$F_{cpo} := 0.625ksi$ (AASHTO T. 8.4.1.1.4-1) Compression perpendicular to grain

$F_{co} := 1.150ksi$ (AASHTO Table 8.4.1.1.4-1) Compression parallel to grain

$E_o := 1600ksi$ (AASHTO T. 8.4.1.1.4-1) Mod. of Elasticity

Adjustment Factors (AASHTO 8.4.4)

$C_M := 1.0$ (Wet service factor for Glu-Lam less than 16%, and sawn lumber less than 19%, ODOT 8.2.4.3). Unless submerged, timber is considered dry (BDM 1.8.2)

$C_{F.Fbo} := 1.4$ Size factors (AASHTO T. 8.4.4.4-1, "Select Structural" row)

$C_{F.Fco} := 1.1$

$C_{F.o} := 1.0$ All other properties

$C_{fu} := 1.0$ (Flat-use factor, ODOT 8.2.4.6)

$C_{i.E} := 0.95$ (Incising factor, for E_o, ODOT 8.2.4.6)

$C_i := 0.80$ Incising factor, for F.bo and F.von (ODOT 8.2.4.7), & F.to and F.co (AASHTO T. 8.4.4.7-1)

$C_{i.cpo} := 1.0$ Incising factor, for F_{cpo} (ODOT 8.2.4.7)

$C_d := 1.0$ Deck factor (ODOT 8.2.4.8)

$C_{\lambda.1} := 0.8$ Time effect factor, Strength Limit State 1 (ODOT 8.2.4.9)

$E := (E_o) \cdot (C_M) \cdot (C_{i.E}) = 1520 \cdot ksi$ (ODOT 8.2.4.1)

$C_{KF.f} := \frac{2.5}{\phi_f} = 2.94$ Format conversion factor, F.b (ODOT 8.2.4.2)

$C_{KF.s} := \frac{2.5}{\phi_s} = 3.33$ Format conversion factor, F.v (ODOT 8.2.4.2)

$C_{KF.cp} := \frac{2.1}{\phi_{cp}} = 2.33$ Format conversion factor, F.cp (compression perpendicular to grain.) (ODOT 8.2.4.2)

$F_b := (F_{bo}) \cdot (C_{KF.f}) \cdot (C_M) \cdot (C_{F.Fbo}) \cdot (C_{fu}) \cdot (C_i) \cdot (C_d) \cdot C_{\lambda.1} = 3.95 \cdot ksi$ (ODOT 8.2.4.1)

$F_v := (F_{vo}) \cdot (C_{KF.s}) \cdot (C_M) \cdot (C_i) \cdot (C_{\lambda.1}) = 0.36 \cdot ksi$ (ODOT 8.2.4.1)

$F_c := (F_{co}) \cdot (C_{KF.s}) \cdot (C_M) \cdot (C_{F.Fco}) \cdot (C_i) \cdot (C_{\lambda.1}) = 2.70 \cdot ksi$ (AASHTO 8.4.4.1-4)

$F_{cp} := (F_{cpo}) \cdot (C_{KF.cp}) \cdot (C_M)(C_i)(C_{\lambda.1}) = 0.93 \cdot ksi$ (ODOT 8.2.4.1)

$L_u := L_{span}$ Assume posts laterally braced base, and horz. members at ends only

$K_{KL} := 2.1$ Effective Length Factor, "K" (AASHTO T. C4.6.2.5-1e)

$L_e := K_{KL} \cdot L_{span} = 4.73\,ft$

Column Stability Factor, C.p

$K_{cE} := 0.52$ Euler buckling coeff. for visually graded lumber columns

$$F_{cE} := \frac{K_{cE} \cdot E_o \cdot h_g^{\,2}}{L_e^{\,2}} = \begin{pmatrix} 7.83 \\ 3.17 \end{pmatrix} \cdot ksi$$ Euler buckling stress, AASHTO 8.8.2-4

$$B_i := \mathrm{Min}\left(\frac{F_{cE_i}}{F_c}, 1.0\right) \qquad B = \begin{pmatrix} 1.00 \\ 1.00 \end{pmatrix}$$ AASHTO 8.8.2-3

$c_c := 0.8$ Coefficient for sawn lumber

$$C_{p_i} := \mathrm{Min}\left[\frac{1 + B_i}{2 \cdot c_c} - \sqrt{\left(\frac{1 + B_i}{2 \cdot c_c}\right)^2 - \frac{B_i}{c_c}}, 1.0\right] \qquad C_p = \begin{pmatrix} 0.69 \\ 0.69 \end{pmatrix}$$ AASHTO 8.8.2-2

$$P_{n_i} := F_c \cdot \left(h_{g_i} \cdot b_g\right) \cdot C_{p_i} \qquad P_n = \begin{pmatrix} 35.90 \\ 22.84 \end{pmatrix} \cdot kip$$

Component Dead Loads (DC):

Deadload of components

$W_{DC} := (w_t) \cdot (h_g) \cdot (b_g) = \begin{pmatrix} 6.68 \\ 4.25 \end{pmatrix} \cdot plf$ Both 4x6 post and 4x6 cap beams.

$L_{cap} := 7.4ft$ Bent 2 cap beam, per decking calcs

Wearing Surface Dead Loads (DW):

N/A

Live Loads (LL):

See referenced loading below

Analysis Sections:

4x6 posts located at Bents 2 and 7. Outside post at bent 2 controls by larger tributary area, by inspection.
4x4 posts located at Bents 1A, 7A, and 8. Bent 1A assumed to control by larger triputary area, and being center support of continuous span 1 span.

Distributed loads

$$W_{STRI.DC} := \gamma_{DC.max} \cdot (W_{DC}) = \begin{pmatrix} 8.36 \\ 5.32 \end{pmatrix} \cdot plf$$

$W_{STRI.DC.Spans} := 49.53\ plf$ Superstructure DC loading on 4x6 spans, per Stringer, per Stringer calcs.

$W_{STRI.LL.Spans} := 315\ plf$ Superstructure LL loading on 4x6 spans, per Stringer, per Stringer calcs.

Axial

$$P_{u.DC.Post_i} := W_{STRI.DC_i} \cdot (L_{span} + 0.5 \cdot L_{cap}) + W_{STRI.DC.Spans} \cdot 0.5(13 + 12.2) \cdot ft$$

$P_{u.DC.Post} = \begin{pmatrix} 0.67 \\ 0.66 \end{pmatrix} \cdot kip$ DC vertical load on post

$P_{u.LL.Post} := W_{STRI.LL.Spans} \cdot 0.5(13 + 12.2) \cdot ft = 3.969 \cdot kip$ LL on post.

$P_{u.Post_i} := P_{u.DC.Post_i} + P_{u.LL.Post}$ $\quad P_{u.Post} = \begin{pmatrix} 4.64 \\ 4.62 \end{pmatrix} \cdot kip$

$Axial_{Ratio.Post_i} := \dfrac{\phi_{cl} \cdot P_{n_i}}{P_{u.Post_i}}$ $\quad Axial_{Ratio.Post} = \begin{pmatrix} 6.96 \\ 4.45 \end{pmatrix}$

$RF_{P.u_i} := \dfrac{\Phi_a \cdot (P_{n_i}) - P_{u.DC.Post_i}}{P_{u.LL.Post}}$ $\quad RF_{P.u} = \begin{pmatrix} 7.97 \\ 5.01 \end{pmatrix}$

Note: Rating factor > 1.0. Say OK for axial
C/D ratio > 1.0. Say OK for axial

Bridge Type: Timber and Glulam pedestrian bidge.simple supports for dead load and live load.

CONCRETE PIER ANALYSIS:

References: AASHTO LRFD BDS 9th Ed. 2020 (AASHTO)
AASHTO MBE 3rd E. 2018 with 2019 Interims (MBE)
AASTHO LRFD Guide Spec for the Design of Pedestrian Bridges 2009 w/ 2015 Interims (PED)
ODOT LRFR Manual June 2018 (ODOT)

LRFR Strength Limit State:

$$RF = \frac{\phi_c \cdot \phi_s \cdot \phi \cdot (R_n) - (\gamma_{DC}) \cdot (DC) - (\gamma_{DW}) \cdot (DW)}{(\gamma_L) \cdot (LL + IM)}$$ (MBE 6A.4.2.1-1)

Resistance Factors:

$\phi_s := 0.90$	LRFD resistance factor for shear	(AASHTO 5.5.4.2)
$\phi_f := 0.90$	LRFD resistance factor for tension controlled sections	(AASHTO 5.5.4.2)
$\phi_{cp} := 0.70$	LRFD resistance factor for compression (assumed similar to Strut & Tie model without confinement reinforcing)	(AASHTO 5.5.4.2)
$\phi_c := 0.85$	Condition factor for substructure condition rating = 3 (Poor)	(MBE T. 6A.4.2.3-1)
$\phi_{sf} := 1.00$	System Factor for Flexure, structure type: "Timber Stringers"	(ODOT 1.4.1.4)
$\phi_{sv} := 1.00$	System Factor for Shear, structure type: "Timber Stringers"	(ODOT 1.4.1.4)
$\phi_{sa} := 1.00$	System Factor for Axial, All other girder bridges and slab bridges	(MBE 6A.4.2.4-1)

Combined Resistance Factors:

For Axial: $\Phi_a := \phi_s \cdot (\max(\phi_c \cdot \phi_{sa}, 0.85))$ $\Phi_a = 0.765$ (Note: $\phi_c \phi_s \geq 0.85$ per MBE 6A.4.2.1-3)

Load Factors:

Dead Load Factors γ_{DC}:

$\gamma_{DC.max} := 1.25$ max. MBE T. 6A.4.2.2-1 for structural components and attachments STR I

Live Load Factors γ_L:

$\gamma_{LL} := 1.750$ MBE T. 6A.4.2.2-1, assume pedestrian loading as Inventory

Dynamic load allowance, IM, is not required with pedestrian loading, PED 3.1

Bridge Members

Analysis below shall consist of the concrete piers:
1) 12" round concrete columns at bents 3-6

Bridge Geometry:

Member Height: $L_{col} := (3.25 \;\; 2)^{T} \text{ft}$

Assume 3'-3" column free height of bent 4 south pier, 2'-0" of bent 4 north pier, scaled from site photos. Piers at bents 5 assumed similar. Piers at bents 3 and 6 shorter than 2'-0".

Member Diam: $d_c := 12\text{in}$

Member area: $a_g := \pi \cdot \frac{d_c^{\,2}}{4} = 113.10 \cdot \text{in}^2$

Material Properties:

$\gamma_c := 0.145\text{kcf}$ (AASHTO Table 3.5.1-1)

$f_c := 3.0\text{ksi}$ Assumed concrete strength, MBE T. 6A.5.2.1-1

Conservatively assume no reinforcing

$K_{KL} := 2.1$ Effective Length Factor, "K" (AASHTO T. C4.6.2.5-1e)

$r := 0.25 \cdot d_c = 0.25\,\text{ft}$ Radius of gyration (AASHTO C5.6.4.3)

$S_R := \frac{K_{KL} \cdot L_{col}}{r} = \begin{pmatrix} 27.30 \\ 16.80 \end{pmatrix}$ (AASHTO 5.6.4.3)

For members not braced against sidesway, the effects of slenderness may be neglected where the slenderness ratio, $K\ell_u/r$, is less than 22.

$E_c := 2.500 \cdot \left(\frac{f_c}{\text{ksi}}\right)^{0.33} \cdot \text{ksi} = 3.59 \cdot \text{ksi}$ (AASHTO C5.4.2.4-1)

$I_g := \frac{\pi \cdot d_c^{\,4}}{64} = 1017.88 \cdot \text{in}^4$

$\beta_d := 0$ No moment

$E_s := 29000\text{ksi}$ Steel modulus of elasticity

$I_s := 0\text{in}^4$ Steel moment of inertia (assumed no steel)

$E_{c.u} := \frac{E_c}{\text{ksi}} = 3.59$ $\quad I_{g.u} := \frac{I_g}{\text{in}^4} = 1017.88$ $\quad E_{s.u} := \frac{E_s}{\text{ksi}} = 29000.00$ $\quad I_{s.u} := \frac{I_s}{\text{in}^4} = 0.00$ Unitless values

$EI := \text{Max}\left(\frac{E_{c.u} \cdot I_{g.u}}{5} + E_{s.u} \cdot I_{s.u}, \frac{\frac{E_{c.u} \cdot I_{g.u}}{2.5}}{1 + \beta_d}\right) \cdot \text{ksi} \cdot \text{in}^4 = 1462.67 \cdot \text{kip} \cdot \text{in}^2$ Flexural stiffness (AASHTO 5.6.4.3)

Capacities:

$$P_e := \frac{\pi^2 \cdot EI}{\left(K_{KL} \cdot L_{col_1}\right)^2} = 2.15 \cdot kip$$ Euler buckling load (AASHTO 4.5.3.2.2b-5)

$k_c := 0.85$ Ratio max conc compressive stress to design compressive stength

$P_{n.1} := 0.80 \cdot \left(k_c \cdot f_c \cdot a_g\right) = 230.72 \cdot kip$ AASHTO 5.6.4.4-3

$P_{n_1} := \mathrm{Min}\left(P_e, P_{n.1}\right) = 2.15 \cdot kip$ $P_{n_2} := P_{n.1} = 230.72 \cdot kip$ $P_n = \begin{pmatrix} 2.15 \\ 230.72 \end{pmatrix} \cdot kip$

Component Dead Loads (DC):

Deadload of components

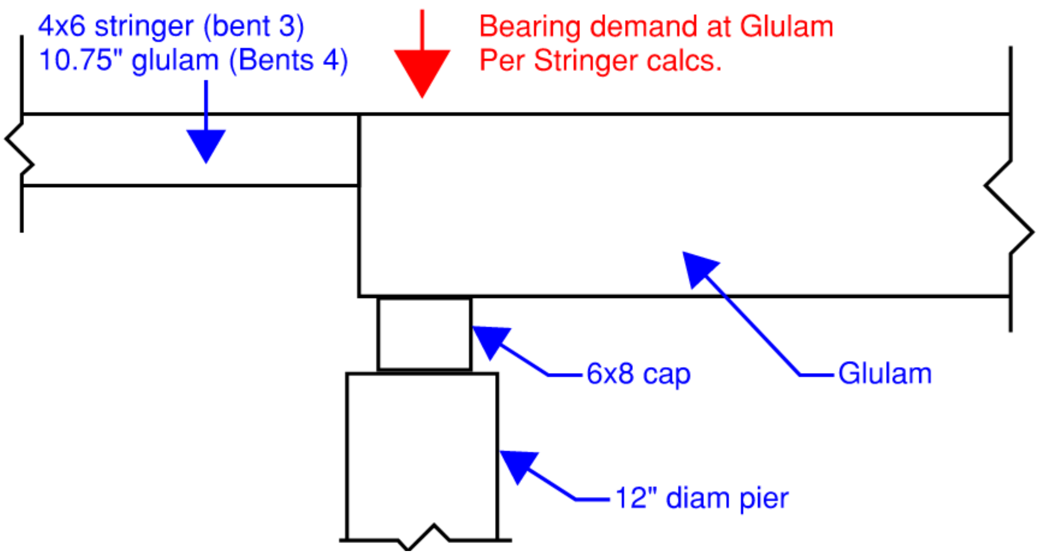
$W_{DC} := \left(\gamma_c\right) \cdot \left(a_g\right) = 113.88 \cdot plf$ 12" diam conc column.

Wearing Surface Dead Loads (DW):

N/A

Live Loads (LL):

See referenced loading below. Sketch also provided for clarity.



Analysis Sections:

Concrete columns support each end of both 21.75" glulam spans.

loads

$R_{21.75.Glulam.DC} := 1.59 \cdot kip$ STR DC Bearing loading on 21.75" glulams, per Stringer calcs.

$R_{21.75.Glulam.LL.3} := 6.84 \cdot kip$ STR LL Bearing loading on 21.75" glulams at bent 3, per Stringer calcs.

$R_{21.75.Glulam.LL.4} := 6.33 \cdot kip$ STR LL Bearing loading on 21.75" glulams at bent 4, per Stringer calcs.

$$R_{21.75.Glulam.LL} := \left(R_{21.75.Glulam.LL.4} \quad R_{21.75.Glulam.LL.3}\right)^T$$

$W_{6x8} := 3.44 plf$ Weight of 6x8 timber cap, per Decking calcs.

$L_{6x8} := 4 \cdot ft$ Length of 6x8 timber cap.

$$P_{6x8} := \gamma_{DC.max} \cdot 0.5 \cdot \left(W_{6x8} \cdot L_{6x8}\right) = 8.60\,lbf$$

$$P_{col} := \gamma_{DC.max} \cdot W_{DC} \cdot L_{col} = \begin{pmatrix} 0.46 \\ 0.28 \end{pmatrix} \cdot kip$$

Axial

$$P_{u.DC} := R_{21.75.Glulam.DC} + P_{6x8} + P_{col} = \begin{pmatrix} 2.06 \\ 1.88 \end{pmatrix} \cdot kip$$

$$P_{u.LL} := R_{21.75.Glulam.LL} = \begin{pmatrix} 6.33 \\ 6.84 \end{pmatrix} \cdot kip$$

$$P_u := P_{u.DC} + P_{u.LL} \qquad P_u = \begin{pmatrix} 8.39 \\ 8.72 \end{pmatrix} \cdot kip$$

$$Axial_{Ratio.Post} := \frac{\phi_{cp} \cdot P_n}{P_u} = \begin{pmatrix} 0.18 \\ 18.51 \end{pmatrix}$$

$$RF_{P.u} := \frac{\Phi_a \cdot \left(P_n\right) - P_{u.DC}}{P_{u.LL}} \qquad RF_{P.u} = \begin{pmatrix} -0.07 \\ 25.53 \end{pmatrix}$$

Note: Rating factor for piers taller than 2'-7" (South side bents 4 and 5) < 1.0, No Good.
Rating factor for piers shorter than 2'-7" (all others) > 1.0. Say OK

C/D ratio for piers taller than 2'-7" (South side bents 4 and 5) < 1.0, No Good.
C/D ratio for piers shorter than 2'-7" (all others) > 1.0. Say OK

Attachment C
Firerock Bridge East Stair Evaluation Site Visit Memo

Technical Memorandum

Firerock Bridge East Stair Evaluation Site Visit Memo

Date: March 22, 2023
Project name: City of Bend Rimrock Pump Station Improvements
Project no: D3380200
Attention: Jason Suhr/City of Bend
Prepared by: Lori Elkins
Reviewed by: Brady Fuller
Copies to: File

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Introduction and Summary

The purpose of this memorandum is to evaluate an existing wooden stairway, to determine compliance with current building codes, and to report findings on the general condition.

A site visit was conducted on March 17, 2023 to the existing wooden, pedestrian stairway adjacent to the Deschutes River west of Firerock Road. The stairway accesses publicly owned space along the east bank of the Deschutes River. The original constructor of the bridge is not clearly understood, but it seems apparent that the stair was meant to access a now derelict wooden pedestrian bridge over the river, which was constructed to support a now-abandoned City of Bend water main which served properties in the vicinity. An unlocked gate from Firerock Road allows access to a pathway along a platted public use easement leading to stairs, with nothing preventing public access to the stairway.

The 2022 Oregon Structural Specialty Code (OSSC) is the current code that governs publicly accessible stairways. No clear differentiation in the code exists between interior and exterior stairs and, as such, this memo applies the OSSC code requirements to the stairs for the current evaluation.

Key outcomes from the site visit include:

- The existing stairway has several components that do not comply with the OSSC, Section 1011 Stairways.
- The existing handrail and guards for the stairs do not meet OSSC, Sections 1014 and 1015, Handrails and Guards, respectively.
- The existing stairs are in general disrepair and structurally would very likely not meet the requirements of OSSC, Chapter 16, Structural Design, however a detailed structural analysis is beyond the scope of this memo.

Components on the stairs that don’t comply with stairway, handrail and guard requirements per Chapter 10 of the OSSC include:

1. Stair width is less than 44 inches required (1011.2). There is an exception for occupancy load served, less than 50 people, in which case the stairway could be considered to meet the code 36 inch minimum width.
2. Stair risers and treads are not uniform size and shape. (1011.5.4). The top riser is shorter than the others and treads are not uniform in length. Numerous treads are various widths.
3. Nosings are not beveled and exceed the maximum overhang. Stair risers are not solid (1011.5.5).
4. There is no stair landing at the bottom of the stairway (1011.6).

5. There are slopes greater than permissible at landing and stair treads (1011.7.1).
6. Vertical rise is greater than 12 feet without an intermediate landing (1011.8).
7. Handrail is not provided on both sides of the stairs (1011.11) Handrails are not provided at all, however guards are provided but on one side only.
8. The 2x4 wood guards do not meet graspability requirements for a handrail (1014.3).
9. Guards do not meet the 42 inch height requirement at the stairway (1015.3).
10. Guards do not meet the requirement for not allowing a maximum 4 inch sphere to pass through. (1015.4).

In addition to the above, structurally the stairs had some obvious deficiencies. It would be difficult to evaluate the capacity and compare it to the code because of the deteriorated condition and various construction details, and the years of evident repair and additional “patches” that have been applied. However, it is believed the capacity of the stairs is lacking for several reasons:

1. Numerous stair treads, risers, and supports were bowed, split and/or checked.
2. Additionally, several posts were not fully bearing on the foundation and when they were bearing, there are no signs of a positive connection to the foundation.

The OSSC code requires stair treads to be designed for 100 pounds per square foot as well as a 300 pound point load applied over a 2 inch square area (Table 1607.1). Handrails and guards must be designed to withstand 50 pound per linear foot (1607.9.1) or a 200 pound point load (1607.9.1.1). For comparison, standard aluminum rail posts (commonly used in the building industry) can typically be calculated to span anywhere from 4 feet to 6 feet maximum, depending on the connections. The posts at this stairway are spaced over 7 feet and are not well connected in some instances to the stringers. In addition, the bottom post connection is loose and can be deflected easily several inches outward and it was found that one of the bolts that connected the post was missing and replaced with a nail.

It is likely that these stairs would not meet the calculated required loads and repairing the deficiencies would be challenging. Based on judgment, it would likely be much more cost-effective to replace the stairs than to repair them if the City’s intent is for the stairs to meet the current code requirements. Alternatively, the stairs could be demolished to remove the hazard and not replaced. The following photos show existing conditions as observed and captions denote key deficiencies noted. All photos were taken March 17, 2023.

The City may wish to assert ownership or responsibility for the stair, place warning signs, and otherwise close it to public access until such time that demolition, repair or replacement is confirmed.

Photograph 1: Overall view of upper stair landing.

Photograph 2: Overall view from top of stairs looking down.

Photograph 3: Top riser not the same as height as others.

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Photograph 4: Nosings not beveled and excessive overhang and gaps.

Photograph 5: Checks and splits in treads.

Photograph 6: Supports not positively connection to foundation.

Photograph 7: Loose post connections.

Photograph 8: Stair treads varying widths.