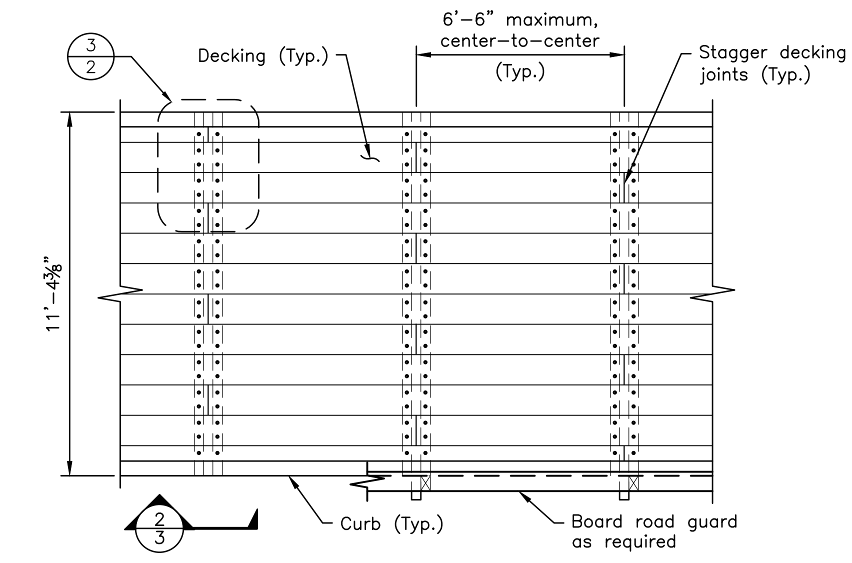
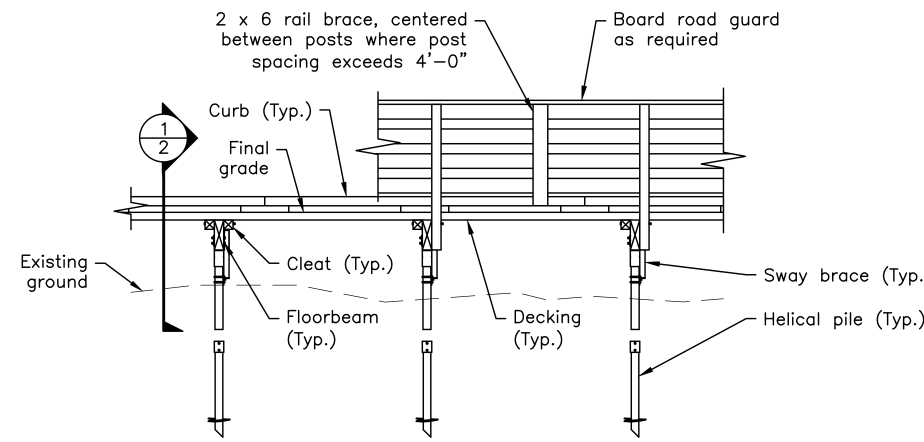
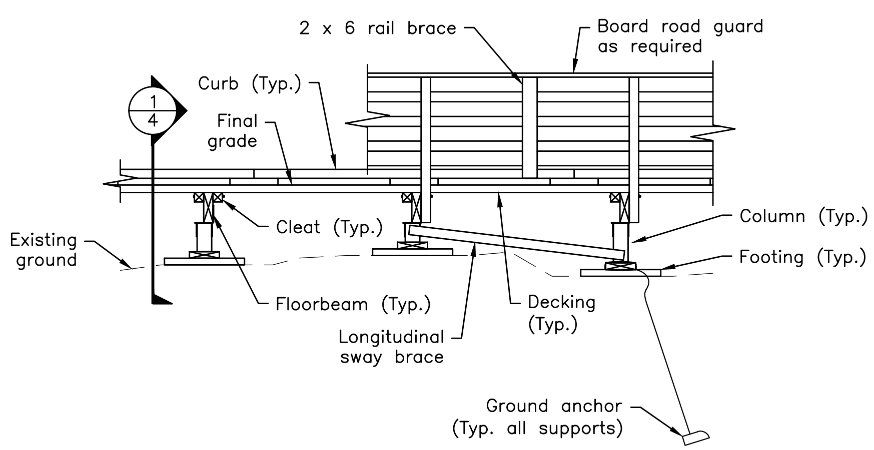


PLAN



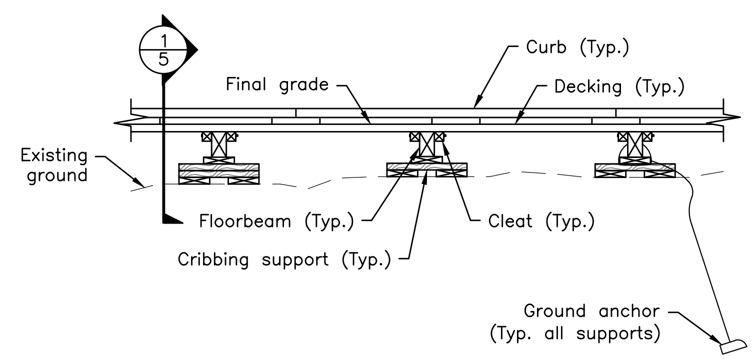
ELEVATION

HELICAL PILE OPTION
(SEE SHT. 2)



ELEVATION

TIMBER COLUMN OPTION
(SEE SHT. 4)



ELEVATION

CRIBBING OPTION
(SEE SHT. 5)

1/1 TYPICAL BOARD ROAD

Scale: NTS

GENERAL NOTES

1. Furnish all material and workmanship in accordance with the State of Alaska Standard Specifications for Highway Construction unless otherwise noted.

2. Coordinate with all affected utility service companies to maintain services during construction.

3. Hot-dip galvanize all metal products in accordance with Section 716, unless noted otherwise. For wood fasteners, see notes 5(C-G).

4. Structural steel:

   A. Furnish ASTM A36 steel above-ground bracing and brackets.

   B. Repair hot-dipped galvanized members that are field cut or drilled in accordance with Section 716.

5. Timber:

   A. Furnish Douglas Fir-Larch No. 1 or Better for all timber unless noted otherwise. Provide S4S finished timber unless noted otherwise.

   B. Furnish Douglas Fir-Larch No. 2 for timber footings and cribbing. Footing and cribbing boards may be rough-sawn.

   C. Pressure treat all timber per Section 714 with use category UC4B, unless noted otherwise.

   D. Furnish framing screws or lag screws with either square, star, or hex head. Furnish stainless steel, hot-dip galvanized, treated for permanent exterior use and contact with pressure treated wood, or approved equal. Where used with a washer or in contact with other metal, the products shall be compatible and not corrode due to contact.

   E. Furnish ASTM A307 bolts and lag screws.

   F. Install fasteners a minimum of 2" from the end of a member to prevent splitting wood. Replace split wood. Pre-drill where required.

   G. Provide lag screw fasteners with a pull out capacity of 200 pounds per inch of thread, minimum, unless otherwise noted. Provide #10 or larger wood screws.

   H. Furnish a nut and two washers for each bolt. Place washer on each side of timber members.

6. Helical piles:

   A. Install to a minimum depth of 15 feet and 1,500 foot-pounds of torque. Depth is measured to the top of the upper helix.

   B. Do not exceed the installation torque rating from the manufacturer. Pre-drill as necessary through hard, frozen ground at the surface or to achieve minimum embedment if helical piles cannot be advanced without exceeding install torque rating.

   C. Furnish ASTM A53, Grade B for central shaft pipes.

   D. Furnish ASTM A36 helix bearing plates.

   E. Furnish ASTM A307 bolts in splice connections.

7. Ground anchors: provide ground anchor with 2 kip minimum working capacity in medium dense sandy gravel.

8. Joist hangers: Furnish hot-rolled steel joist hangers, zinc coat per ASTM A653, G90.

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BOARD ROAD
PLAN AND ELEVATION

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Chief Engineer

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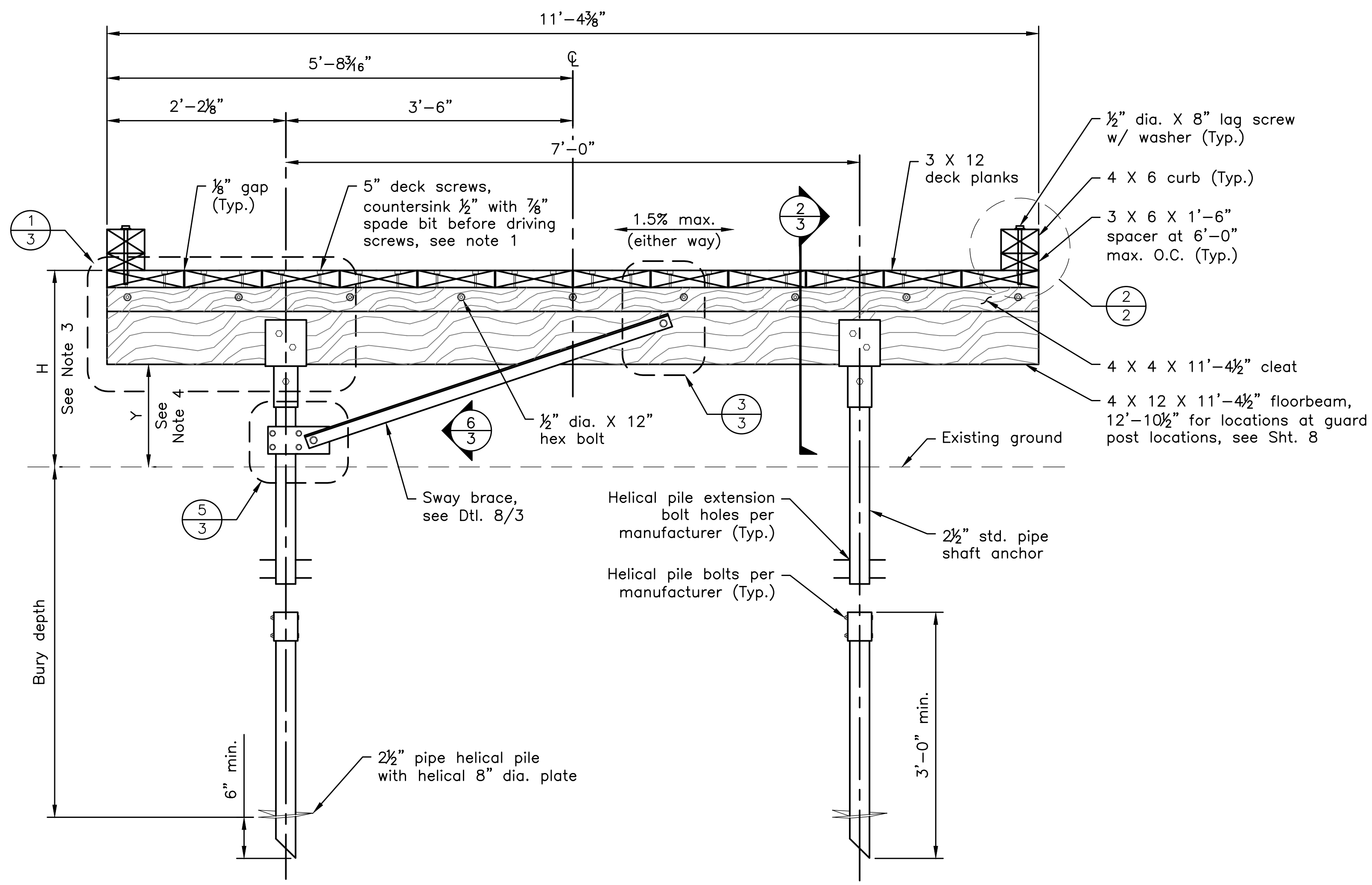


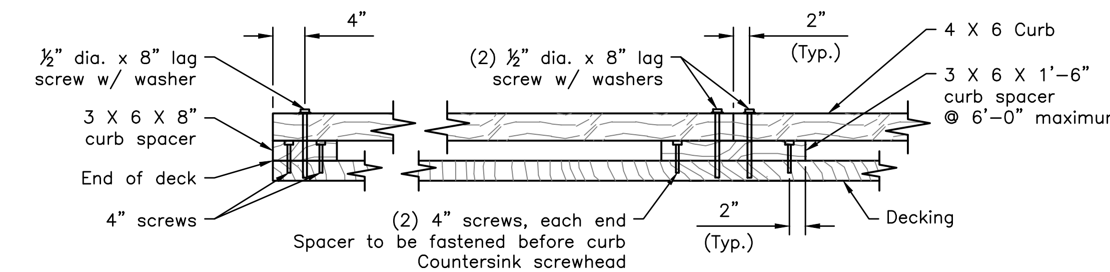
NOTES:

1. Position deck screws to avoid conflict with 12" through-bolts.
2. Sway brace: Alternate sides of board road every set of helical piles.
3. Provide guard where required by Detail 1/8.
4. Omit sway brace where Y is less than 12".

1/2 TYPICAL SECTION

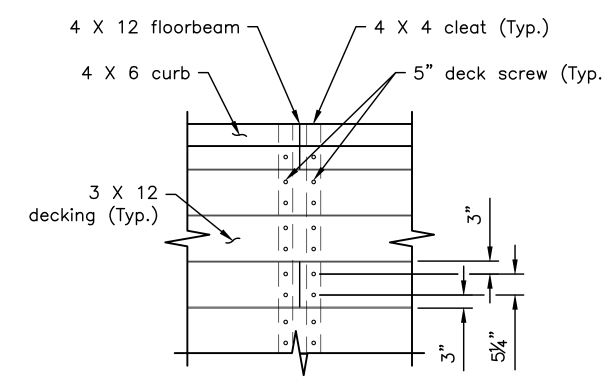
Scale: NTS

NOTE:
Locate curb splices 1'-0" minimum from board road guards where used.



2/2 CURB CONNECTION DETAIL

Scale: NTS



3/2 BOARD ROAD CONNECTION PLAN

Scale: NTS

State of Alaska DOT&PF
ALASKA STANDARD PLAN
BOARD ROAD TYPICAL SECTION AND DETAILS

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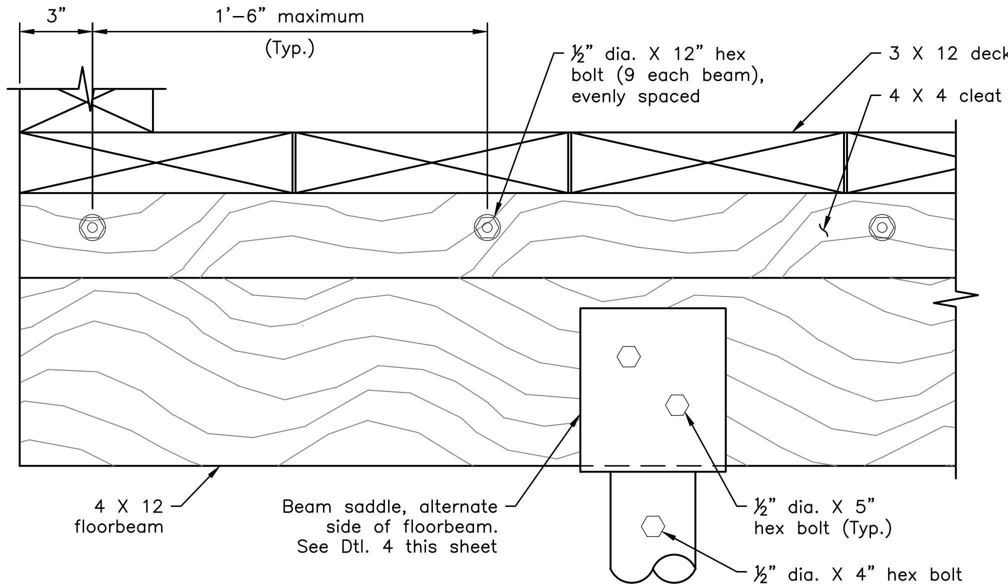
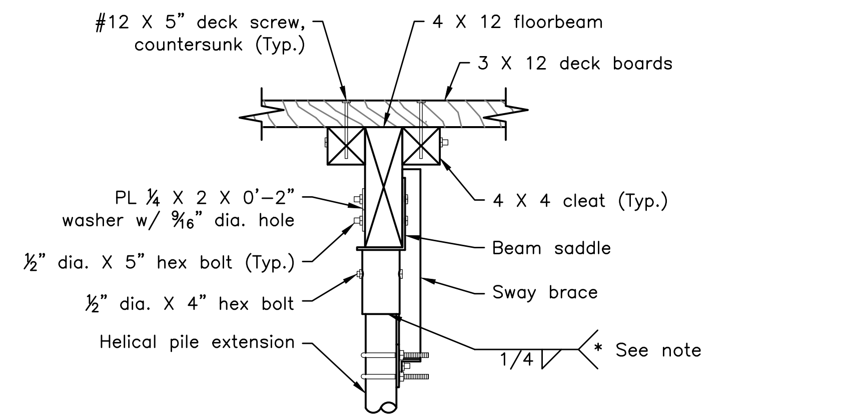
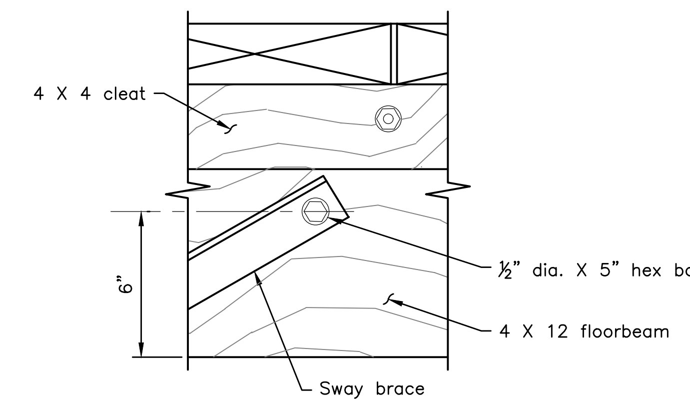


1/3 BEAM SADDLE – ELEVATION
Scale: NTS

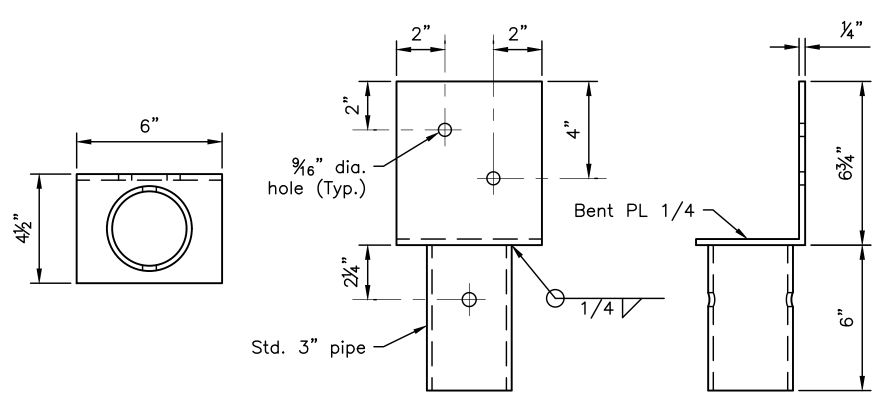


*NOTE:
If the bolted connection between the beam saddle and helical pile cannot be achieved, this weld may be used with the approval of the Engineer.

2/3 BOARD ROAD SUPPORT SECTION
Scale: NTS

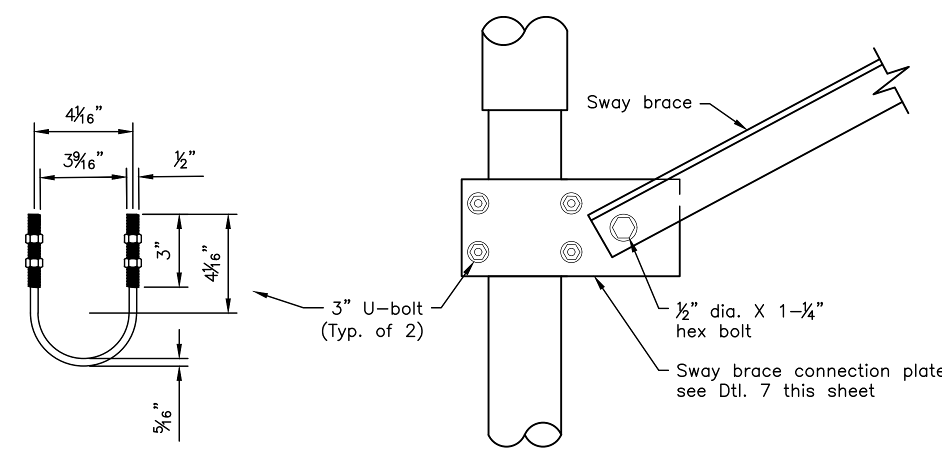


3/3 SWAY BRACE CONNECTION – ELEVATION
Scale: NTS

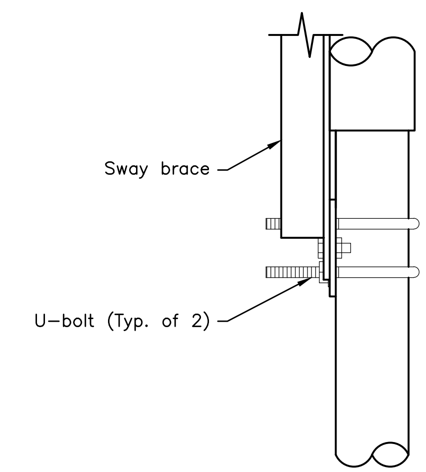


NOTE:
An L 6 X 6 X 5/16 may be substituted for the bent PL ¼. Maintain the vertical location of the bolt holes into the floorbeam.

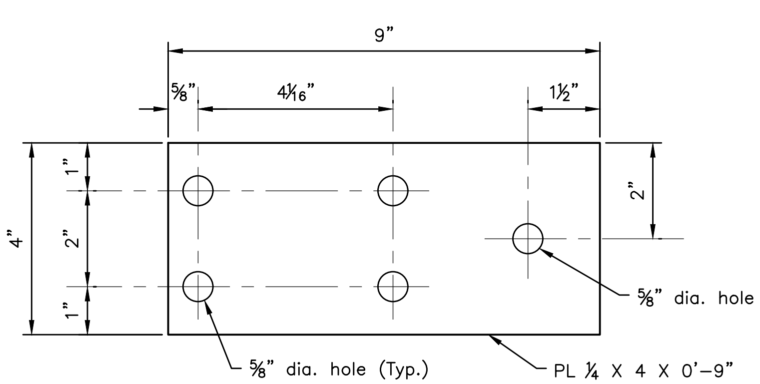
4/3 SADDLE DETAIL
Scale: NTS



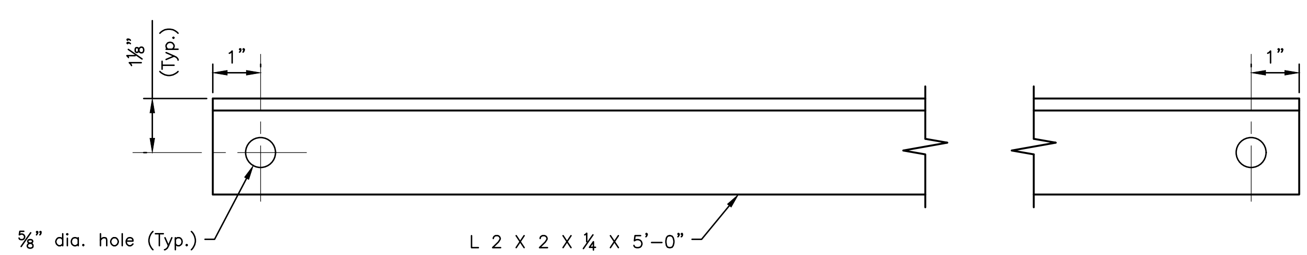
5/3 SWAY BRACKET
Scale: NTS



6/3 U-BOLT CONNECTION
Scale: NTS



7/3 SWAY BRACE CONNECTION PLATE
Scale: NTS



8/3 SWAY BRACE
Scale: NTS

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ALASKA STANDARD PLAN
BOARD ROAD SUBSTRUCTURE CONNECTION DETAILS

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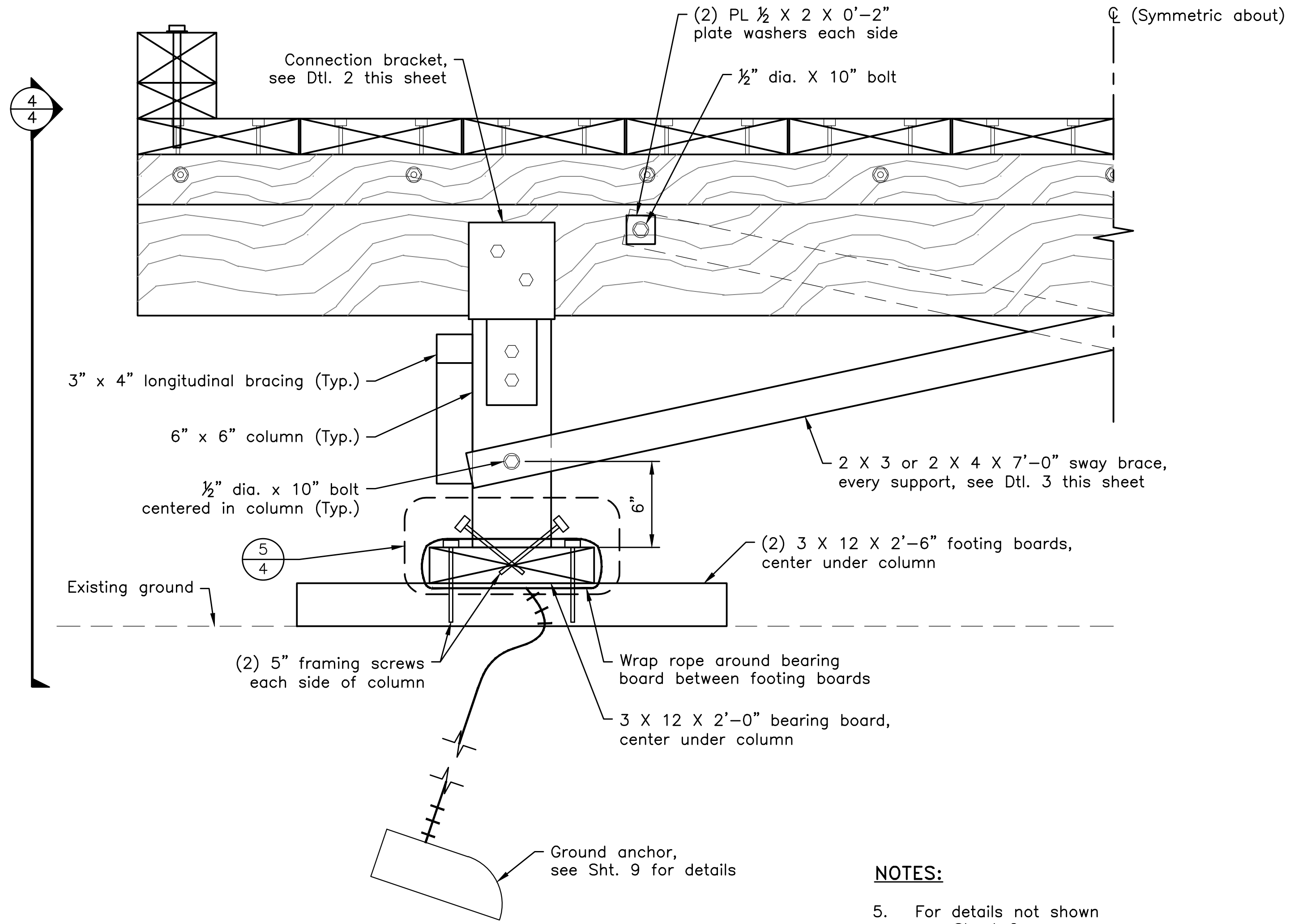


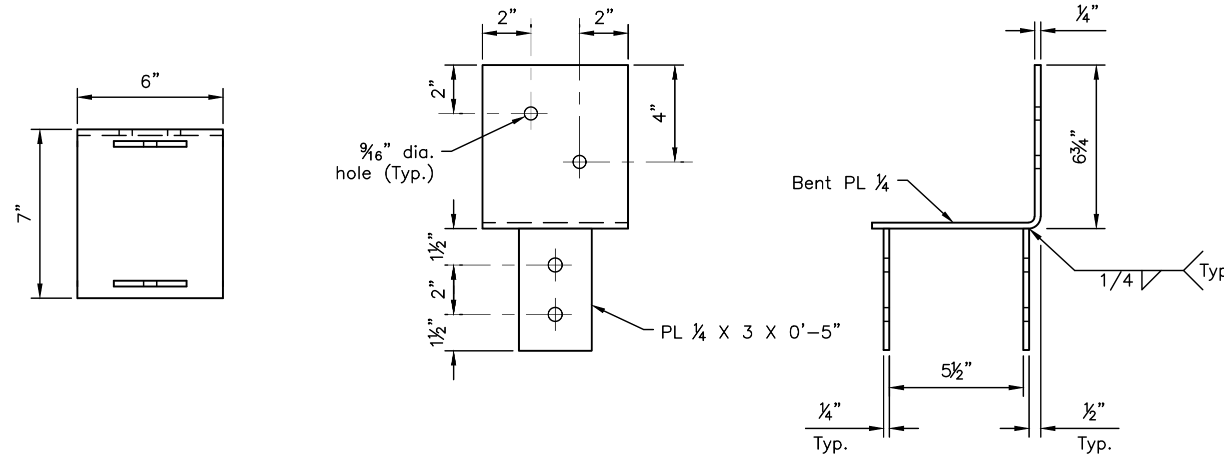
1/4 TYPICAL SECTION
Scale: NTS

NOTES:

5. For details not shown see Sheet 2.
6. Provide guard where required by Detail 1/8.

NOTES:

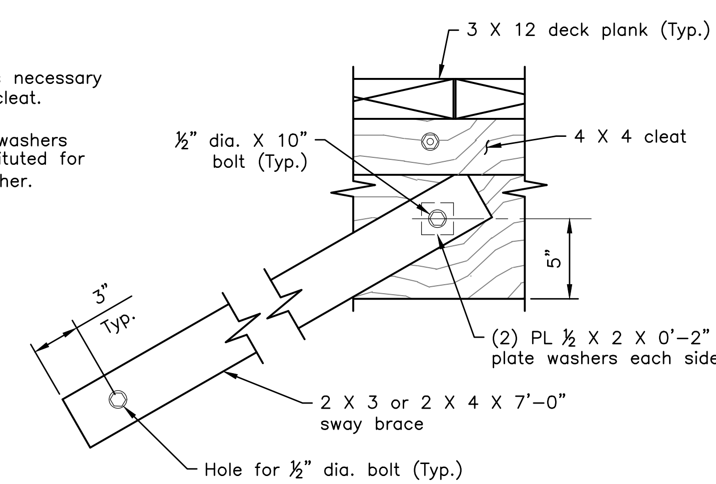
1. Prefabricated post caps/joist holders may be substituted with Engineer approval.
2. A rolled L 8 X 8 member may be substituted for the bent plate. Trim corner of floorbeam minimum required to clear L 8 X 8 inside radius.



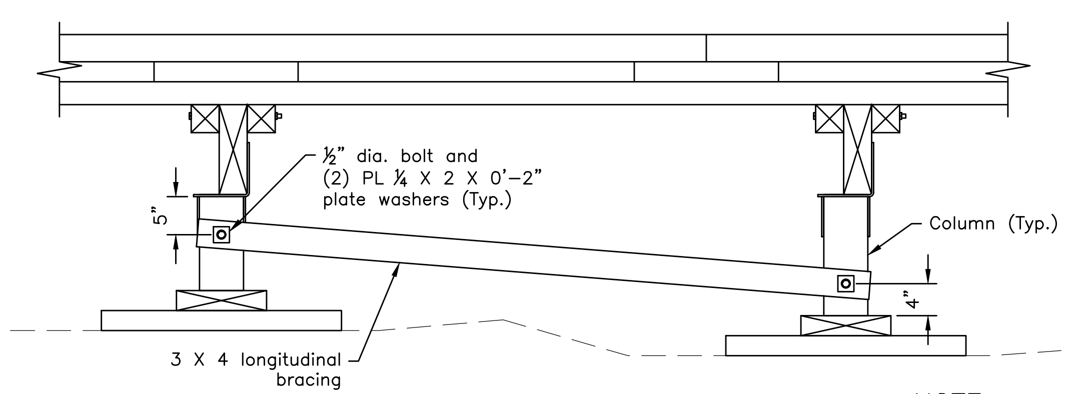
2/4 TIMBER COLUMN SADDLE DETAILS
Scale: NTS

NOTES:

3. Trim brace as necessary to clear the cleat.
4. Double PL ¼ washers may be substituted for the PL ½ washer.

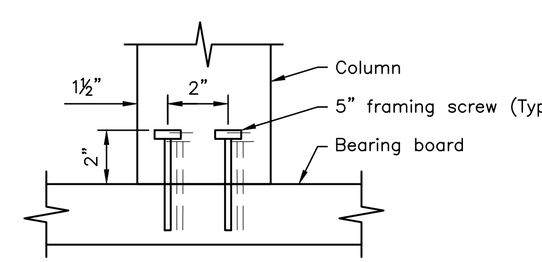


3/4 TIMBER COLUMN SWAY BRACE DETAILS
Scale: NTS



4/4 ELEVATION
Scale: NTS

NOTE:
Ground anchors not shown.



5/4 TOE CONNECTION DETAIL
Scale: NTS

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ALASKA STANDARD PLAN
BOARD ROAD
TIMBER COLUMN
SUPPORT DETAILS

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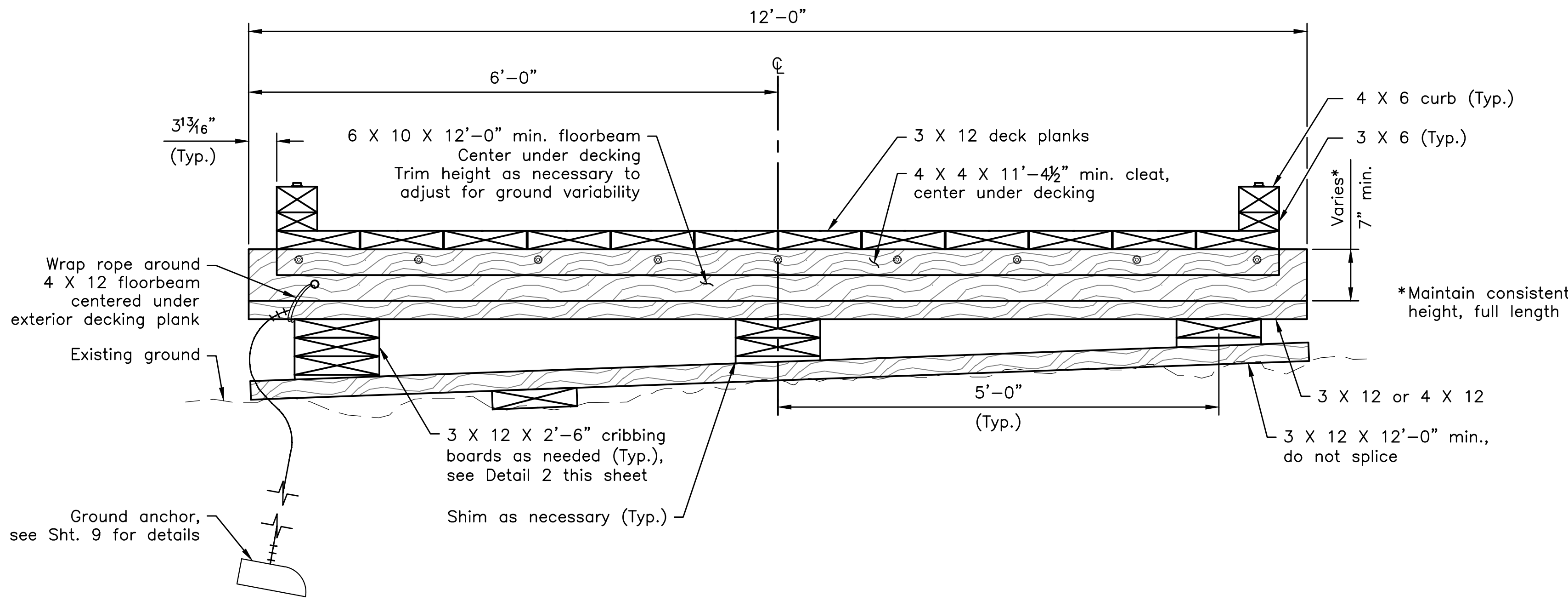
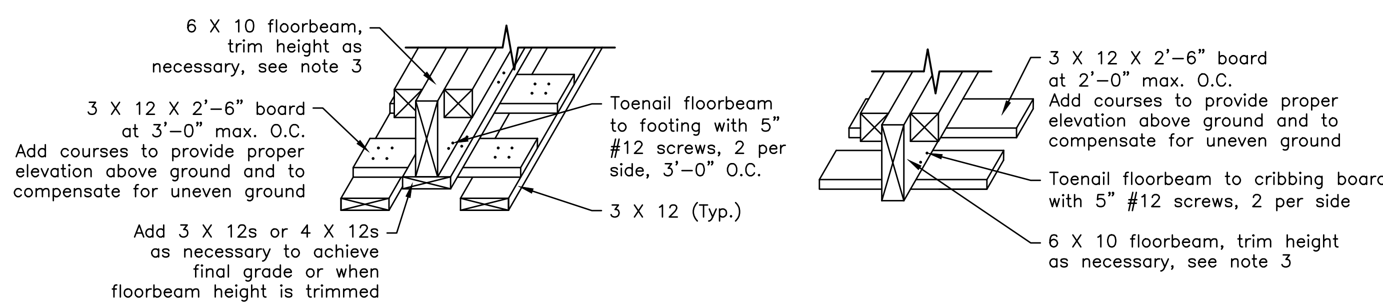


See Sheet 2 for additional notes and details.

1/5 TYPICAL SECTION – ON CRIBBING SUPPORT

Scale: NTS



NOTES:

1. Use (4) 5" #12 screws where each cribbing board contacts another cribbing board.
2. Stack 3 X 12 X 2'–6" as required to provide a level board road (<2% cross slope).
3. For the floorbeams, 6–by timbers may be substituted for the 4 X 12s. Omit cleats where 6–by timbers are used.

2/5 CRIBBING DETAIL

Scale: NTS

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ALASKA STANDARD PLAN

BOARD ROAD
CIRBBING SUPPORT DETAILS

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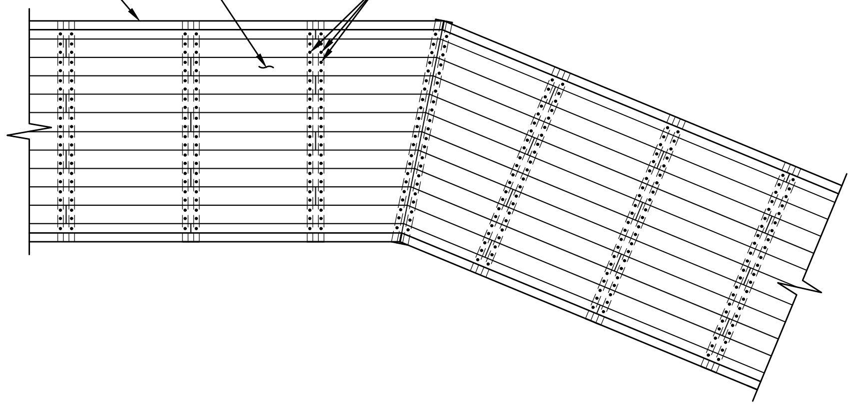
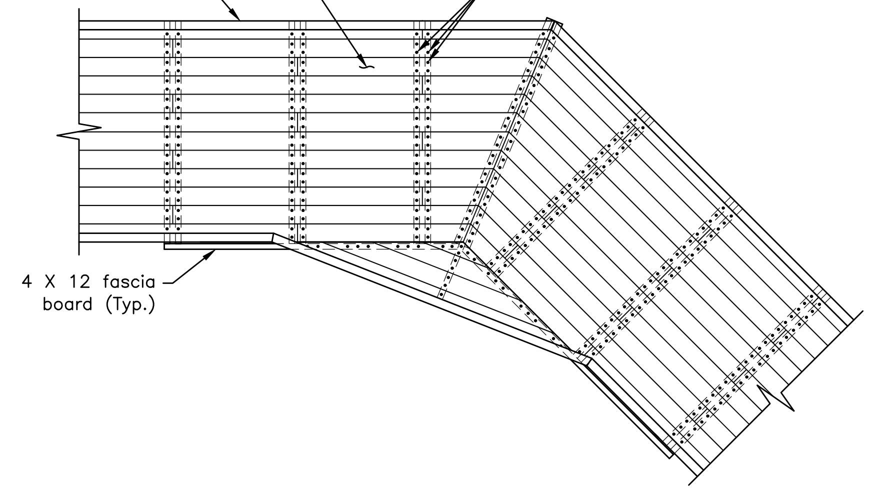
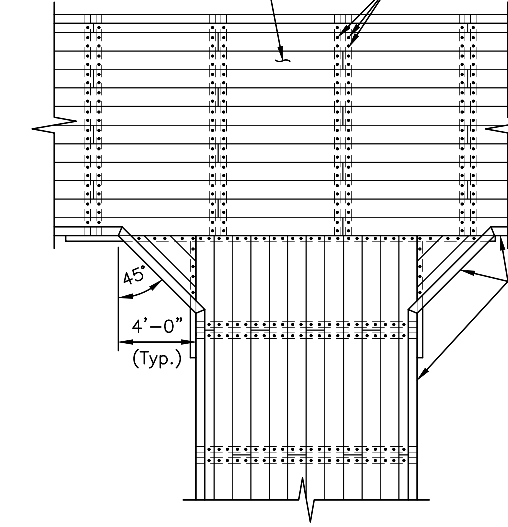


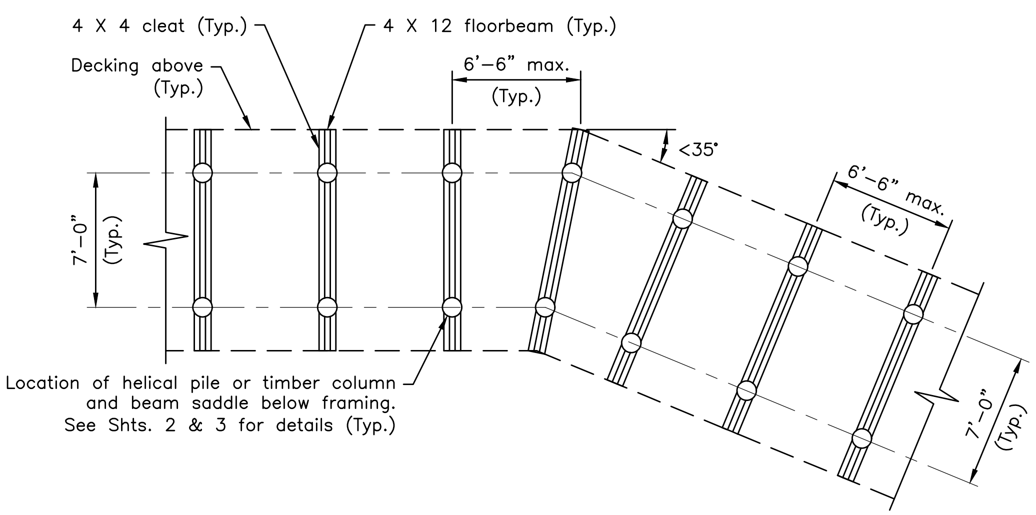
(1/6) ANGLED DECKING PLAN – LESS THAN 35 DEGREES
Scale: NTS



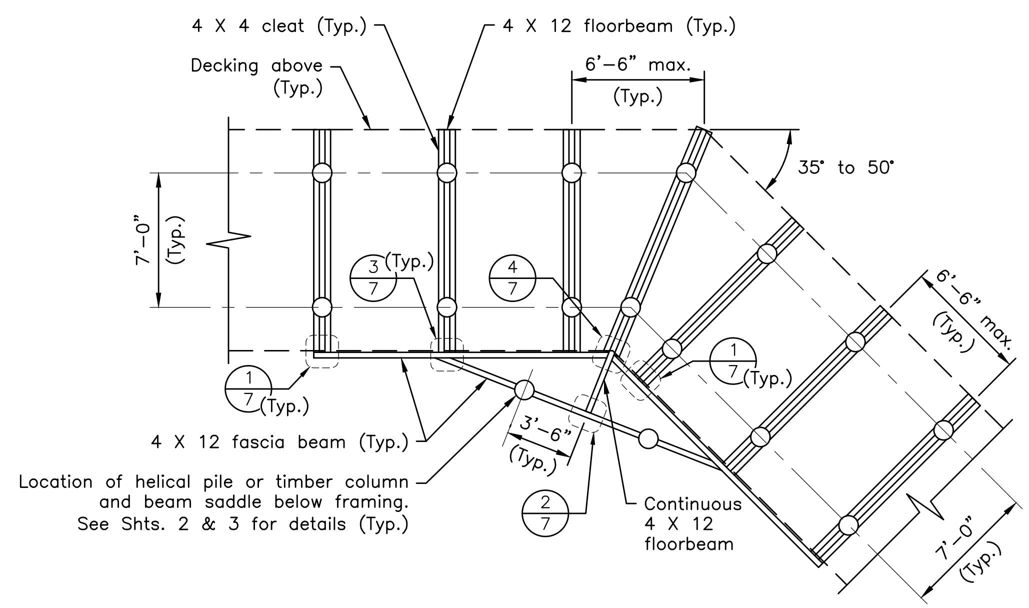
(3/6) ANGLED DECKING PLAN – BETWEEN 35 AND 50 DEGREES
Scale: NTS



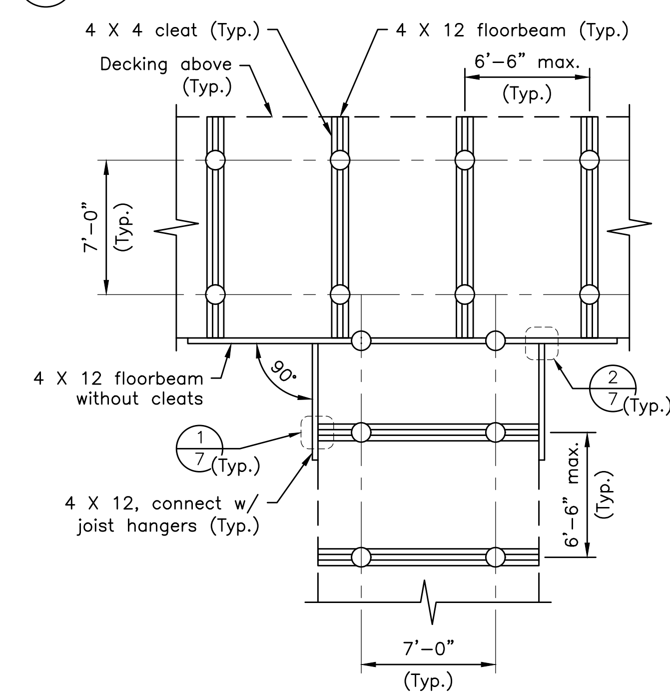
(5/6) DECKING PLAN – 90 DEGREE INTERSECTION
Scale: NTS



(2/6) ANGLED FRAMING PLAN – LESS THAN 35 DEGREES
Scale: NTS



(4/6) ANGLED FRAMING PLAN – BETWEEN 35 AND 50 DEGREES
Scale: NTS



(6/6) FRAMING PLAN – 90 DEGREE INTERSECTION
Scale: NTS

NOTE:
For framing angles between 50 and 90 degrees, use combinations of angle plans for angles between 0 and 50 degrees or develop project–specific framing plans for Engineer approval.

State of Alaska DOT&PF
ALASKA STANDARD PLAN
BOARD ROAD
ANGLED DECKING
AND FRAMING PLANS

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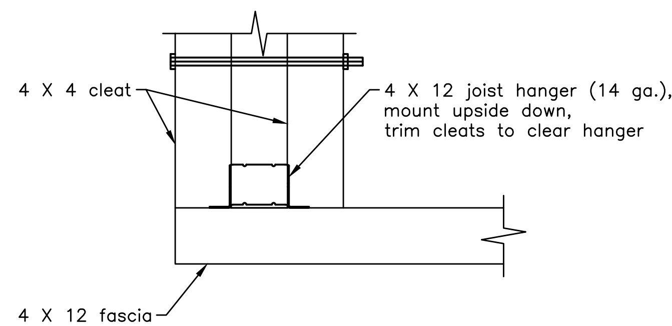
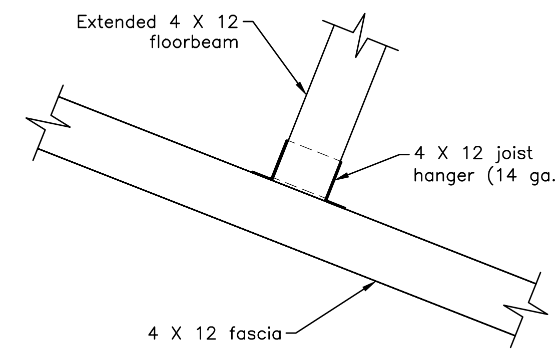
NOTES:

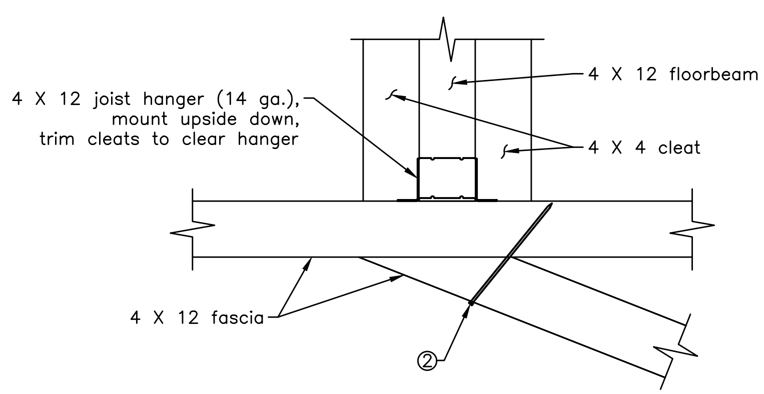
1. Angle framing screws to avoid the tip protruding beyond framing.
2. 8" screws. 3 screws centered vertically, equally spaced. 3" minimum to 4" maximum between screws.



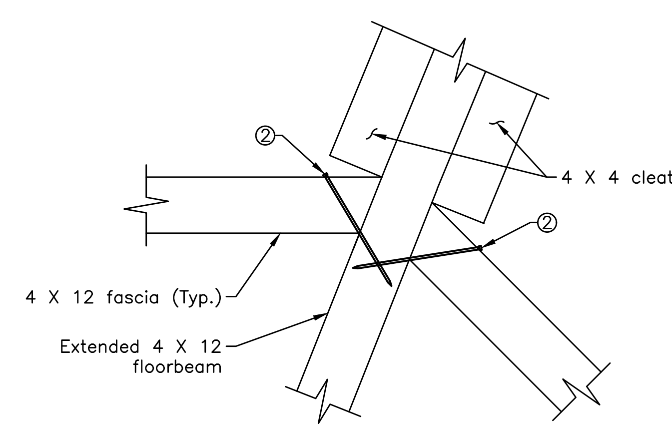
1/7 FRAMING DETAIL – PLAN
Scale: NTS



2/7 FRAMING DETAIL – PLAN
Scale: NTS



3/7 FRAMING DETAIL – PLAN
Scale: NTS



4/7 FRAMING DETAIL – PLAN
Scale: NTS

State of Alaska DOT&PF
ALASKA STANDARD PLAN

BOARD ROAD
FRAMING DETAILS

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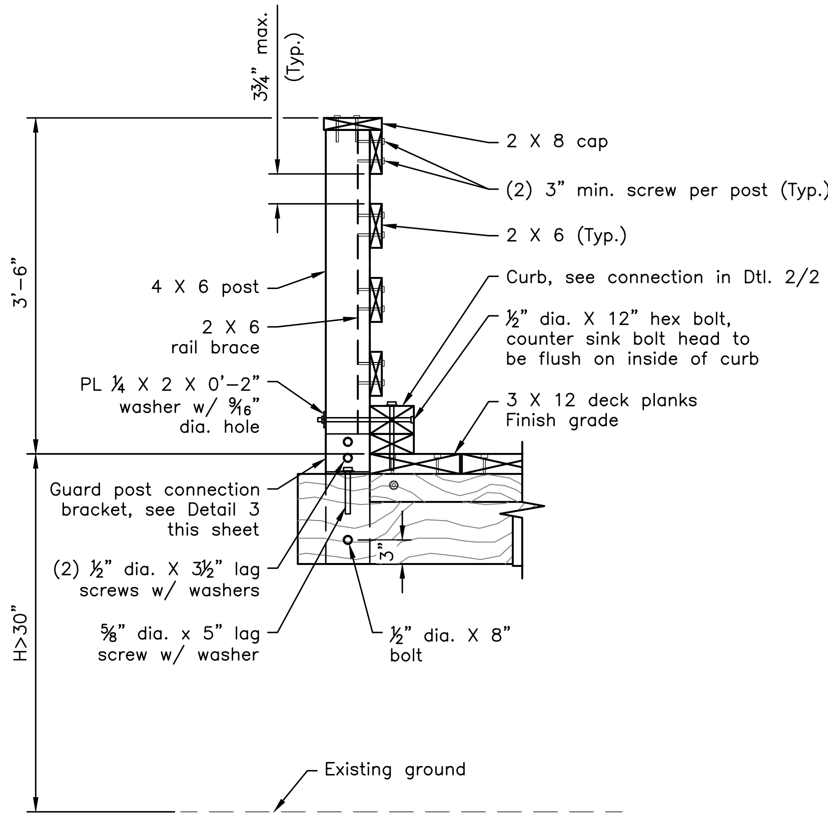


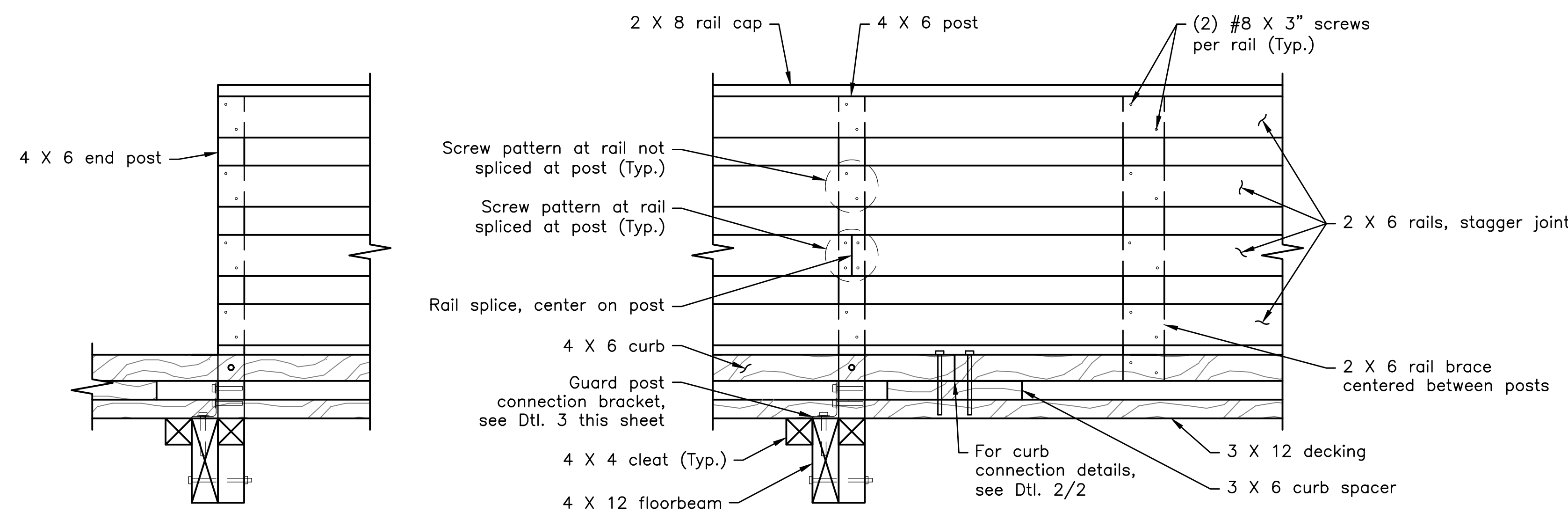
NOTES:

1. Board road guard required where deck finish grade exceeds 30" above existing grade or where handrails are required. Determine "H" independently on each side of board road.

2. Mount handrails to guards. For "H" < 30", the 2 X 6 rails not used for handrail connections, 2 X 8 cap, and rail braces may be removed from the guard.

3. See Detail 1/2 for other board road details.

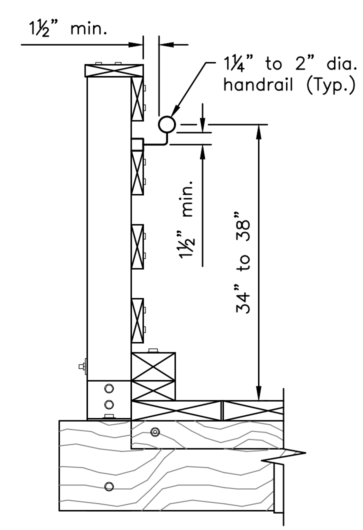
1/8 TYPICAL BOARD ROAD GUARD SECTION
Scale: NTS



AT END OF RAIL

TYPICAL RAIL

2/8 TYPICAL BOARD ROAD GUARD ELEVATION
Scale: NTS



4/8 HANDRAIL DETAILS
Scale: NTS

HANDRAIL NOTES:

4. Provide continuous, graspable, and circular handrail on any mainline board road exceeding 5% grade.

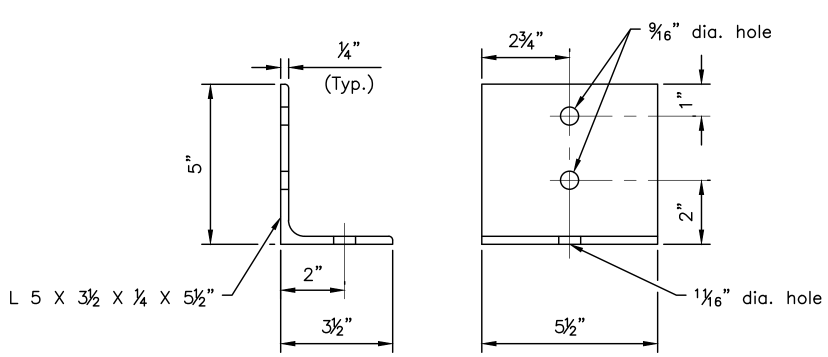
5. Provide handrails on both sides of the board road where handrails are required.

6. Extend handrail horizontally in the direction of the ramp a minimum of 12 inches beyond the top and bottom of the ramp runs.

7. Provide handrail returns to the guards at terminations.

8. Furnish handrail assemblies constructed of galvanized steel, stainless steel or aluminum.

9. Furnish handrail assemblies meeting the loading requirements of the 2024 International Building Code 1607.9.



3/8 GUARD POST CONNECTION BRACKET
Scale: NTS

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ALASKA STANDARD PLAN

BOARD ROAD GUARD DETAILS

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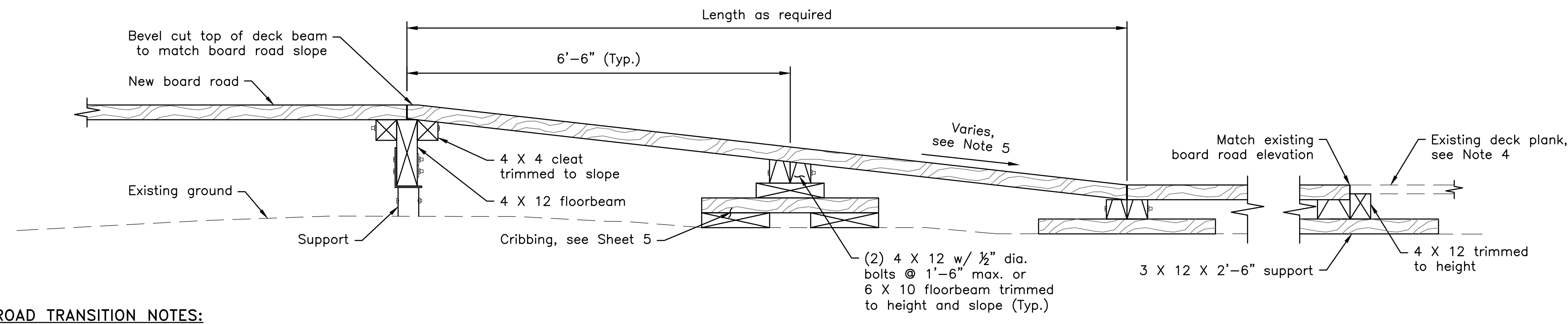


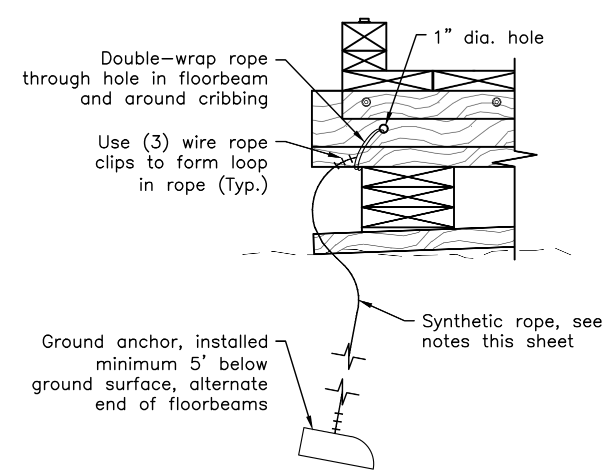
BOARD ROAD TRANSITION NOTES:

1. If existing board road is narrower than new board road, fillet between existing and new board roads at 45 degrees.
2. Existing and new board road surfaces shall be flush, so that no wood protrudes above the wear surface more than ⅛".

BOARD ROAD GRADE NOTES:

3. Splice all deck boards at supports with vertical alignment angle points.
4. Existing board road configurations and construction varies. Existing board road heights vary. Use helical pile, timber column, or cribbing foundations as required to match existing grade. See foundation details on sheets 2, 4, and 5. Develop project-specific connection details to existing board roads for Engineer approval.
5. If the grade exceeds 5%, provide ADA compliant landings at the top and bottom of ramp.

1/9 BOARD ROAD PROFILE
Scale: NTS



GROUND ANCHOR NOTES:

6. Provide polyester rope, ⅜" minimum diameter with 3000 pound minimum breaking strength.
7. Connect the rope directly to the ground anchor, remove any rope provided with the anchor as necessary.
8. Provide 1'-0" of slack in rope in final position.

2/9 GROUND ANCHOR DETAIL
Scale: NTS

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ALASKA STANDARD PLAN

BOARD ROAD MISCELLANEOUS DETAILS

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Next Code and Standards Review Date: 02/20/2036

STANDARD PLAN DEVELOPMENT REPORT (SPDR)

Standard Plan No.: M-26.00

Title: Board Roads

Prepared by: Eric E. Bonn, P.E.

Date: February 20, 2026

Use: This Alaska Standard Plan (ASP) has details and guidance on constructing timber board roads.

This ASP is a stand-alone document with no references to other ASPs.

Design and Application Considerations:

- Board Road
  - Board roads are structurally designed for all-terrain vehicles (ATV) as described in the Design Backup section, below.
  - The clear width allows for two passing ATVs and is similar to other recent projects.
  - Longitudinal deck planks match typical previous designs. Some ADA standards suggest deck plans should be oriented perpendicular to the direction of travel. However, orienting the decking longitudinally allows for more cost-effective maintenance (a primary mode of plank failure is wear from snow machines), is a superior driving surface, and is selected by the Department as the preferred orientation for this application.
  - The board roads are also used as public pedestrian routes. This ASP complies with ADA requirements for cross slope and handrails. It is also designed with guards for fall protection at vertical drops as required by the International Building Code. The designer for projects should consider the following for ADA compliance:
    - An alignment and profile should be provided whenever feasible to properly incorporate ADA requirements (as noted in the following), making use of this ASP for the board road itself.
    - As much of a board road system as possible should be less than 5% grade (use 4.5% nominally); where approaches exist or are anticipated, the grade should be less than 2% to accommodate approach landings.
    - The maximum board road grade is nominally 8.0%.
    - Any board road exceeding 5% grade is a ramp.
    - All ramps require handrails.

        - A ramp cannot exceed a vertical change of more than 30 inches. Landings are required at the top and bottom of all ramps. Ramps may be consecutive after a landing is provided.
        - A landing is a 5-foot segment of board road where all slopes are less than 2%. Use cross slope and grades that are nominally 1.5%.
    - Board road heights should be as low to the ground as foundations allow and seek to be less than 30" above existing ground to avoid needing to install guards.
    - The connections of the horizontal guard rails to the posts do not meet the strength requirements for vertical rail loads. Based on performance history of similar rails, this is considered acceptable to the Department.
    - Polyester rope was selected for its inherent UV resistance. Other synthetic rope materials would be acceptable if UV protection was provided.
    - Transitions between new and existing board roads will require site-specific solutions.
    - Due to the variable site configurations and layouts likely to be encountered at the board road installation locations, various project-specific details may need to be developed in coordination with this ASP.
- Foundations:
    - Options for helical piles, timber posts, and cribbing are provided.
    - A geotechnical investigation is recommended to assist in selecting the preferred or primary foundation type.
    - Selection of the foundation type should consider frozen ground, thaw, frost heaving, and flooding (which can float the board road).
    - Where flooding is expected, the preferred foundation is the helical pile when subsurface soil conditions allow.
    - Multiple foundation types should be expected due to constructability constraints.
- Material Properties: as specified on the ASP drawing.

<u>Applicable Design Standards, Codes, and Specifications</u>:

Design standards and guidelines that apply to this ASP are contained in the following publications:

Standards:

- AASHTO LRFD Bridge Design Specification (LRFD, 2020)
- AASHTO LRFD Guide Specifications for the Design of Pedestrian Bridges (2^{nd} Ed. w/2015 Interims)
- AISC Steel Construction Manual (15^{th} Edition)

- National Design Specifications for Wood Construction (NDS, 2018)
- International Building Code (2024)
- ASCE 7: Minimum Design Loads and Associated Criteria for Buildings and Other Structures (2022)
- 2020 Alaska Standard Specification for Highway Construction (SSHC)
  - 713 Structural Timber, Lumber, and Piling
  - 714: Preservatives for Timber
  - 716: Structural Steel
- Approved Modifications to the Standard Specifications for Highway Construction, Standard Modifications for 2020 SSHC
- American Society for Testing and Materials (ASTM)
  - ASTM A36: Standard Specification for Structural Steel
  - ASTM A53: Standard Specification for Pipe, Steel, Black and Hot-Dipped, Zinc-Coated, Welded and Seamless
  - ASTM A307: Standard Specification for Carbon Steel Bolts, Studs, and Threaded Rod 60,000 PSI Tensile Strength

Guidelines:

- AKDOT&PF Alaska Bridges and Structures Design Manual (ABSM, 2025)
- AKDOT&PF Alaska Preconstruction Manual (APM, 2025)

History:

Alaska Standard Plan M-26.00 (February 20, 2026) – issued as a new ASP.

Tests or Backup Data: None.

Design Backup:

Loadings:

- Vertical loadings included the following:
  - Four-wheel all-terrain vehicle (ATV) towing a trailer with 700-pound wheel loads per HPCM. The ATV used for design was the Polaris Sportsman Touring XP-1000 totaling 1500 pounds for vehicle and payload.
  - Pedestrian loading of 90 pounds per square foot per LRFD.

    - Pedestrian rail distributed load of 50 pounds per linear foot or concentrated load of 200 pounds per IBC, not in combination with horizontal pedestrian loading.
    - Snow accumulation loading of 54 pounds per square foot per ASCE 7
- Horizontal loadings include the following:
    - Pedestrian rail distributed load of 50 pounds per linear foot or concentrated load of 200 pound per IBC, not in combination with vertical pedestrian loading.
    - Wind load, due to a 140 miles per hour 3-second-gust, of 65 pounds per square foot per LRFD.
- Load Combinations
    - The LRFD does not specifically address load combinations applicable to board roads for the load and resistance factor design. Three vertical load combinations were considered for component dead loads (DC), live loads (pedestrian or vehicular, LL) and snow accumulation (IC):
        - Case 1 = 1.25*DC + 1.75*LL + 1.0*IC
        - Case 2 = 1.25*DC + 1.75*LL
        - Case 3 = 1.25*DC + 1.35*LL + 1.0*IC

      Case 3 was selected as the most appropriate for the ASP.
    - LRFD limit state horizontal load combinations considered include the following:
        - Strength I (1.25*DC + 1.75*LL) for pedestrian rail loads
        - Strength III (1.25*DC + 1.00*WS) for wind load on structure (WS)
        - Strength V (1.25*DC + 1.35*LL + 0.40*WS) for pedestrian rail loads with wind on structure. The LRFD combination was modified per ABSM 12.1.3.5).
- Design Calculations are attached

<u>Construction Considerations:</u>

- Soil conditions can vary widely from site to site and due to time of year of construction. The contractor needs to be prepared to adjust their construction types and methods.
- Installation of the helical anchors may require predrilling to install to the minimum depth required.
- Substitution of other wood grades would require verification of the load carrying capacities and may require reduced span lengths or larger members.
- Existing ground is not to be disturbed to the extent possible during construction. No vegetation removal is allowed.

M&O Considerations:

- Adjustments to the board road grade may need to be made due to changes in ground level over time due to frost heave, water table variations, or settlement.
- Extreme flooding and wind events could displace the structure, requiring realignment.
- All elements of the board roads should be regularly inspected and any damaged components repaired or replaced.

TA for Standard Service

Alaska Department of Transportation and Public Facilities

M26.00 CALCULATIONS

Prepared For: AKDOT&PF
Project Number: 1571.50315.01
Date: 20 February 2026

Project Abstract:
The Western Alaska Board Roads ASP Project is for development of Alaska Standard Plan M26.00 and corresponding calculations for timber board roads typically used in small villages in western Alaska.

These calculations accompany drawings for the above project dated: 20 February 2026

TABLE OF CONTENTS

Structural Calculations
TA for Standard Service
Alaska Department of Transportation and Public Facilities

Prepared For: AKDOT&PF
Project Number: 1571.50315.01
Date: 20 February 2026

STATE OF ALASKA
49 TH
ERIC E. BONN
No. 13727
REGISTERED PROFESSIONAL ENGINEER
02/20/2026

Subject	Page

General

WESTERN ALASKA BOARDWALK ASP
LOADING CALCULATIONS

Narrative:

This file calculates and summarizes the loads placed on or developed in the walkway.

Table of Contents:

1. References:

AASHTO LRFD Bridge Design Specifications, 9th Ed. (LRFD)
AASHTO LRFD Guide Specifications for the Design of Pedestrian Bridges (AASHTO Ped)
Alaska Highway Preconstruction Manual (AHPM)
Alaska Bridges and Structures Manual (ABSM)
ASCE 7-22 Minimum Design Loads for Buildings and Other Structures (ASCE)

2. Constants/Input Information:

General --

$kcf := 1000\ pcf$ (unit definition)

Span --

$S_{fb} := 5\ ft + 4\ in = 5.33\ ft$ (spacing of floorbeams, maximize for allowable design)

$L_{span} := S_{fb} = 5.33\ ft$ (span length)

3. Loads:

Component Dead Load (DC, deck planks) --

Timber --

$\gamma_w := 0.050\ kcf$ (unit weight, LRFD Tbl. 3.5.1-1)

Pedestrian Live Load (PL, LRFD Ped 3.1) --

$p_{ped} := 0.090\ ksf$ (pressure)

Vehicle Live Load (LL) --

Use four-wheeler ATV live load from AHPM 1170 as shown below. The ATV and trailer load will be considered a Strength I limit state load.

https://www.polaris.com/en-us/off-road/sportsman/models/sportsman-touring-xp-1000-trail/sportsman-touring-xp-1000-trail-heavy-metal-specs/#

2. Use as the minimum design vehicle a four-wheel, all-terrain vehicle pulling a two-wheel trailer with an 80-gallon tank, with a design load of up to 700 pounds per wheel.

DIMENSIONS

Estimated Dry Weight	929 lbs. (421 kg)
Front/Rear Rack Capacity	120 lbs. (55 kg) / 240 lbs. (110 kg)
Fuel Capacity	5.25 gal (19.9 L)
Ground Clearance	11.25" (28.5 cm)
Hitch Type	Standard 1.25" (3.2 cm) receiver
Overall Vehicle Size (L x W x H)	84.3 x 48.2 x 48.8" (214 x 123 x 124 cm)
Payload Capacity	575 lbs. (261 kg)
Person Capacity	2
Seat Height	35.75" (90.8 cm)
Towing Capacity	1,500 lbs. (680 kg)
Wheelbase	57" (144.8 cm)

$W_{atv} := 929\ lbf + 575\ lbf = 1504\ lbf$ (total load of ATV + payload)

$P_{wheel.atv} := \frac{W_{atv}}{4} = 376\ lbf$ (wheel load, ATV + payload)

$P_{wheel.tr} := 700\ lbf$ (wheel load, trailer)

$s_{axle.atv} := 50.5\ in = 4.21\ ft$ (axle spacing, ATV)

$s_{axle.tr} := 48\ in = 4.00\ ft$ (axle spacing, ATV to trailer, assumed)

$s_{wl} := 48\ in = 4.00\ ft$ (wheel line spacing, ATV and assumed for trailer)

Snow Loads (IC) --

Snow loads are not in the LRFD. Use ASCE 7-16 to determine the snow load.
The City of Bethel will be taken as representative.
Treat the loading as if on a flat roof (ASCE 7.3).

$Risk := \text{“II”}$ (Risk Category, not low or high risk to human life, ASCE Tbl. 1.5-1)

$p_g := 64\ psf$ (ground snow loading from ASCE Tbl. 7.2-1)

$Roughness := \text{“C”}$ (Surface Roughness Category, open terrain with scattered obstructions, ASCE 26.7.2)

$Exposure := \text{“Partially Exposed”}$ (exposure of "roof", assumed)

$C_e := 1.0$ (exposure factor, ASCE Tbl. 7.3-1)

$C_t := 1.2$ (thermal factor, open-air structure, ASCE Tbl. 7.3-2)

$p_f := (0.7) \cdot (C_e) \cdot (C_t) \cdot (p_g) = 54\ psf$ (flat roof snow load, ASCE Eq. 7.3-1)

Wind Load --

$V_3 := 140\ mph$ (3-second gust wind speed, LRFD Fig. 3.8.1.1.2-1)

$GSR := \text{“C”}$ (ground surface roughness category, open terrain with scattered obstructions, LRFD 3.8.1.1.4)

$WEC := \text{“C”}$ (wind exposure category, GSR <> "B" or "D", LRFD 3.8.1.1.5)

$Z := 6\ ft$ (structure height, approximate)

$K_z := 1.00$ (elevation coefficient, Z < 33', WEC = "C", LRFD Tbl. C3.8.1.2.1-1)

$G := 1.00$ (gust effect factor, all other structures, LRFD Tbl. 3.8.1.2.1-1)

$C_D := 1.3$ (drag coefficient, I-girder superstructure is similar, LRFD Tbl. 3.8.1.2.1-2)

$P_Z := \left(2.56 \cdot 10^{-6} \cdot ksf\right) \cdot \left(\frac{V_3}{mph}\right)^2 \cdot \left(K_z\right) \cdot \left(G\right) \cdot \left(C_D\right) = 65.2\ psf$ (wind pressure, LRFD Eq. 3.8.1.2.1-1)

Live Load on Pedestrian Rail (ABSM 16.5.2 & LRFD 13.8.2) --

$w_{LLr} := 0.050\ klf$ (distributed load acting transversely and vertically simultaneously)

$P_{LLr} := 0.20\ kip$ (concentrated load)

Seismic --

Seismic loads are not considered due to low risk to human life and ease of repair.

```
STRUDL 'WESTERN ALASKA BOARDWALK ASP'

$ 2-SPAN CONTINUOUS LOADING MODELS

$ ENGINEER:  ERIC E. BONN, P.E.
$ COMPANY:  DOWL, LLC


$ ***** JOINT DEFINITIONS *****


UNITS INCHES

JOINT COORDINATES

$ SPAN LENGTHS = 5'-6"
101   0.00 0.00 $ BENT
123 132.00 0.00 $ BENT
$ SPAN LENGTHS = 6'-0"
147 276.00 0.00

GENERATE BETWEEN 101 123 FREE GLOBAL
 XDIRECTION 22 PARTS EQUAL ID INC 1
GENERATE BETWEEN 123 147 FREE GLOBAL
 XDIRECTION 24 PARTS EQUAL ID INC 1

STATUS SUPPORT -
 101 112 123 135 147

JOINT RELEASES
 101 147 FORCE X MOMENT Z
 112 123 MOMENT Z
 135 FORCE X MOMENT Z

$ ***** MEMBER DEFINITIONS *****


TYPE PLANE FRAME

GENERATE 22 MEMBERS ID 101 INC 1 FROM 101 INC 1 TO 102 INC 1
GENERATE 24 MEMBERS ID 123 INC 1 FROM 123 INC 1 TO 124 INC 1

DEFINE GROUP 'g56Span' '5.50-FOOT SPANS' MEMBERS 101 TO 122
DEFINE GROUP 'g60Span' '6.00-FOOT SPANS' MEMBERS 123 TO 146
DEFINE GROUP 'gPLANK' 'ALL PLANK MEMBERS' GROUPS 'g56Span' 'g60Span'

MEMBER RELEASES
 122 END   MOMENT Z
$ 123 START FORCE X MOMENT Z

UNITS INCHES

MEMBER PROPERTIES PRISMATIC
 GROUP 'g56Span' AX 28.125 IZ 14.65
 GROUP 'g60Span' AX 28.125 IZ 14.65

MATERIAL STEEL

UNITS KIP INCHES

CONSTANTS
 E 1539 GROUP 'g56Span'
 E 1539 GROUP 'g60Span'


$ ***** LOADINGS *****


UNITS LBS FEET
```

```
LOADING 'LD_UNIF' 'UNIFORM 1 KLF INFLUENCE LOAD'
 MEMBER LOADS
  GROUP 'gPLANK' FORCE Y GLOBAL UNIFORM FRACTIONAL W -1.00 LA 0.0 LB 1.0

$ LIVE LOAD - ATV W/TRAILER WHEEL LINE
MOVING LOAD GENERATOR
 SUPERSTRUCTURE N 2 MEMBER EXISTING
 TRUCK LOAD DIRECTION BOTH -
 VEHICLE GENERAL TRUCK 376 4.75 376.0 4.00 700.0
 GENERATE LOAD Y SCALE -1.00 INITIAL 'atv1' CREATE GROUP 'ATV'
END LOAD GENERATOR


$ ***** RUN ANALYSIS AND OUTPUT RESULTS *****


STIFFNESS ANALYSIS

UNITS INCHES KIPS

LOAD LIST 'LD_UNIF'
LIST REACTIONS JOINTS EXISTING  112 135

LOAD LIST GROUP 'ATV'
LIST FORCE ENVELOPE MEMBERS EXISTING 101 TO 111 SECTION FR DS 0.0 1.0
LIST FORCE ENVELOPE MEMBERS EXISTING 123 TO 134 SECTION FR DS 0.0 1.0
LIST MAXIMUM REACTION ENVELOPE FOR LOADS ACTIVE
```

Superstructure

WESTERN ALASKA BOARD ROAD ASP
LONGITUDINAL DECK PLANK CALCULATIONS
Case 3: 1.25*DC + 1.35*LL + 1.00*IC

Narrative:

This file checks the structural capacity of the longitudinal deck planks.

The Case 3 load combination is not in the LRFD. This combination was developed based on the Strength II limit state combination with the addition of snow accumulation, which is not normally included. The probability of having a design snow load in combination with a heavily overloaded design vehicle was deemed low, justifying a lower live load factor than the 1.75 used for Strength I limit state combinations.

Table of Contents:

1. References:

AASHTO LRFD Bridge Design Specifications, 9th Ed. (LRFD)
AASHTO LRFD Guide Specifications for the Design of Pedestrian Bridges, 2nd Ed. w/2015 Interims (AASHTO Ped)
AISC Steel Construction Manual, 15th Ed. (AISC)
Alaska Highway Preconstruction Manual, January 2025 Ed. (AHPM)
Alaska Bridges and Structures Manual, June 2025 Ed. (ABSM)
National Design Specifications for Wood Construction, 2018 Ed. (NDS)

2. Constants/Input Information:

General --

$kcf := 1000\ pcf$ (unit definition)

Timber Members (S4S) --

Curb (4 x 6 curb member and 3 x 6 spacers) --

$b_k := 5.5\ in$ (width of curbs)

Deck Planks (3 x 12, flat use) --

$b_{plank} := 11.25\ in$ (width)

$t_{plank} := 2.5\ in$ (thickness)

$S_{plank} := \frac{(b_{plank}) \cdot (t_{plank}{}^2)}{6} = 11.72\ in^3$ (section modulus)

$I_{plank} := \frac{(b_{plank}) \cdot (t_{plank}{}^3)}{12} = 14.65\ in^4$ (moment of inertia)

Floorbeam (4 x 12 w/4 x 4 cleats) --

$b_{fb} := 3.5\ in$ (width, floorbeam)

$N_{cleat} := 2$ (number of cleats)

$b_{cleat} := 3.5\ in$ (width, cleat)

Span --

$S_{fb} := 6\ ft + 6\ in = 6.50\ ft$ (spacing of floorbeams, maximize for allowable design)

$L_{span} := S_{fb} - (2) \cdot \left(\frac{b_{fb} + (N_{cleat}) \cdot (b_{cleat})}{4} \right) = 6.06\ ft$ (effective span length)

Deck and Walkway --

$b_{dk} := (7\ ft + 0\ in) + (2) \cdot (1\ ft + 9\ in) = 10.50\ ft$ (total width of decking)

$b_{wlk} := b_{dk} - (2) \cdot (b_k) = 9.58\ ft$ (width of walkway)

Materials --

Timber Reference Design Values (Douglas Fir-Larch, No. 1 or better, LRFD Tbl. 8.4.1.1.4-1) --

Decking is "dimension lumber"

$F_{bo} := 1.200\ ksi$ (bending)

$F_{vo} := 0.180\ ksi$ (shear parallel to grain)

$F_{cpo} := 0.625\ ksi$ (compression perpendicular to grain)

$E_o := 1800\ ksi$ (modulus of elasticity)

$\gamma_w := 0.050\ kcf$ (unit weight, LRFD Tbl. 3.5.1-1)

Load Modifiers (LRFD 1.3.1.2) --

$\eta_D := 1.00$ (ductility factor, conventional design, LRFD 1.3.3)

$\eta_R := 1.05$ (redundancy factor, deck planks are nonredundant, LRFD 1.3.4)

$\eta_I := 1.00$ (operational importance factor, typical bridge, LRFD 1.3.5)

$\eta := (\eta_D) \cdot (\eta_R) \cdot (\eta_I) = 1.05$ (load modifier, max. γi appropriate, LRFD Eq. 1.3.2.1-2)

Load Factors (LRFD Tbls. 3.4.1-1 & 2) --

$\gamma_{DC} := 1.25$ (component dead load)

$\gamma_{LL} := 1.35$ (live load, ATV and trailer per Strength II limit state used for Case 3 combination)

$\gamma_{IC} := 1.00$ (ice and snow accumulation, per Extreme Event II limit state used for Case 3 combination)

Resistance Factors (LRFD 8.5.2.2) --

$\phi_f := 0.85$ (flexure)

$\phi_v := 0.75$ (shear)

$\phi_{cp} := 0.90$ (compression perpendicular to grain)

<u>3. Loads on Deck:</u>

See Loads.mcdx for loading calculations.

Component Dead Load (DC) --

$w_{plank} := (b_{plank}) \cdot (t_{plank}) \cdot (\gamma_w) = 9.77\ plf$ (unit weight of plank)

Live Loads --

Pedestrian (PL) --

$p_{ped} := 0.090\ ksf$ (pressure)

$w_{ped} := (p_{ped}) \cdot (b_{plank}) = 84.38\ plf$ (distributed load)

Vehicular (LL) --

Wheel Loads --

$P_{wheel.atv} := 376\ lbf$ (ATV)

$P_{wheel.tr} := 700\ lbf$ (trailer)

Axle Spacings --

$s_{axle.atv} := 57\ in = 4.75\ ft$ (ATV)

$s_{axle.tr} := 48\ in = 4.00\ ft$ (ATV to trailer, assumed)

Distribution Factor for Wheel Load --

$DF := 1.0$ (tire narrower than plank width)

Dynamic Load Allowance --

Not applied (wood structure, LRFD 3.6.2.3)

Snow (IC) --

$p_{snow} := 54\ psf$ (pressure)

$w_{snow} := (p_{snow}) \cdot (b_{plank}) = 50.6\ plf$ (distributed load on plank)

4. Flexural Demand and Resistance:

Flexural Demand --

General Mid-Span Moment Formulas --

$$M_w(w) := \frac{(w) \cdot (L_{span}{}^2)}{8}$$

(due to uniform distributed load, assume simply-supported, AISC Tbl. 3-23.1)

$$M_{p.ctr}(P) := \frac{(P) \cdot (L_{span})}{4}$$

(concentrated load at center of span, AISC Tbl. 3-23.7)

$$M_{p.a}(P, a) := \frac{(P) \cdot (L_{span} - a) \cdot (0.5 \cdot L_{span})}{L_{span}}$$

(concentrated load at point "a" > Lspan/2, AISC Tbl. 3-23.8)

Unfactored Mid-Span Moments --

$M_{DC} := M_w(w_{plank}) = 0.54\ in \cdot kip$ (component dead load)

$$M_{VL} := (DF) \cdot M_{p.ctr}(P_{wheel.tr}) + \left\| \begin{array}{l} \text{if } 0.5 \cdot L_{span} > s_{axle.tr} \\ \quad \left\| \begin{array}{l} a_{atv} \leftarrow 0.5 \cdot L_{span} + s_{axle.tr} \\ M_{p.a}(P_{wheel.atv}, a_{atv}) \end{array} \right. \\ \text{else} \\ \quad \| \ 0 \cdot in \cdot kip \end{array} \right.$$

(live load, vehicle, trailer axle place at mid-span controls by inspection)

$M_{VL} = 12.73\ in \cdot kip$

$M_{PL} := M_w(w_{ped}) = 4.65\ in \cdot kip$ (pedestrian live load)

$M_{LL} := \max(M_{VL}, M_{PL}) = 12.73\ in \cdot kip$ (controlling live VL or PL load)

$M_{IC} := M_w(w_{snow}) = 2.79\ in \cdot kip$ (snow load)

Factored and Combined Mid-Span Moments --

$M_{uDC} := (M_{DC}) \cdot (\gamma_{DC}) = 0.67\ in \cdot kip$ (component dead load)

$M_{uLL} := (M_{LL}) \cdot (\gamma_{LL}) = 17.19\ in \cdot kip$ (live load)

$M_{uIC} := (M_{IC}) \cdot (\gamma_{IC}) = 2.79\ in \cdot kip$ (snow load)

$M_u := (\eta) \cdot (M_{uDC} + M_{uLL} + M_{uIC}) = 21.68\ in \cdot kip$ (combined, Case 3 combination)

Flexural Resistance (LRFD 8.6) --

Adjustment Factors for Reference Design Values and Adjusted Design Value (LRFD 8.4.4) --

$C_{KF.b} := \dfrac{2.5}{\phi_f} = 2.94$ (format conversion factor, LRFD 8.4.4.2)

$C_{F.b} := 1.0$ (size factor, 3 x 12, LRFD Tbl. 8.4.4.4-1)

$C_{M.b} := \begin{cases} 1.00 & \text{if } (F_{bo}) \cdot (C_{F.b}) \le 1.15\ ksi \\ 0.85 & \text{else} \end{cases} = 0.85$ (wet service factor, t <= 4", LRFD Tbl. 8.4.4.3-1)

$C_{fu} := 1.20$ (flat-use factor, width > 10", thickness = 3", LRFD Tbl. 8.4.4.6-1)

$C_{i.b} := 0.80$ (incising factor, LRFD Tbl. 8.4.4.7-1)

$C_d := 1.0$ (non-stress or laminated deck, LRFD 8.4.4.8)

$C_\lambda := 0.8$ (time effect factor, Strength-I used, LRFD Tbl. 8.4.4.9-1)

$F_b := (F_{bo}) \cdot (C_{KF.b}) \cdot (C_{M.b}) \cdot (C_{F.b}) \cdot (C_{fu}) \cdot (C_{i.b}) \cdot (C_d) \cdot (C_\lambda)$ (adjusted design value, LRFD Eq. 8.4.4.1-1)

$F_b = 2.30\ ksi$

Components in Flexure, Rectangular Section (LRFD 8.6.2) --

$C_L := 1.0$ (beam stability factor, depth < width, LRFD 8.6.2)

$M_n := (F_b) \cdot (S_{plank}) \cdot (C_L) = 27.00\ in \cdot kip$ (nominal resistance, LRFD Eq. 8.6.2-1)

$M_r := (\phi_f) \cdot (M_n) = 22.95\ in \cdot kip$ (factored resistance, LRFD Eq. 8.6.1-1)

$CDR_f := \frac{M_r}{M_u} = 1.06$ >= 1.0, OKAY (capacity-to-demand ratio)

5. Shear Demand and Resistance:

Shear Demand --

General Girder End Formulas --

$V_w(w) := \frac{(w) \cdot (L_{span})}{2}$ (due to uniform distributed load, assume simply-supported, AISC Tbl. 3-23.1)

$V_{p.end}(P) := P$ (concentrated load at end of span, with "a" = 0, AISC Tbl. 3-23.8)

$V_{p.a}(P, a) := \frac{(P) \cdot (L_{span} - a)}{L_{span}}$ (concentrated load at point "a", AISC Tbl. 3-23.8)

Unfactored Girder End Shears --

Conservatively taking the shear at the extreme end of the span.

$V_{DC} := V_w(w_{plank}) = 0.030\ kip$ (component dead load)

$V_{VL} := V_{p.end}(P_{wheel.tr}) + \max(V_{p.a}(P_{wheel.atv}, s_{axle.tr}), 0\ kip) = 0.828\ kip$ (vehicular live load)

$V_{PL} := V_w(w_{ped}) = 0.256\ kip$ (pedestrian live load)

$V_{LL} := \max(V_{VL}, V_{PL}) = 0.828\ kip$ (controlling live load)

$V_{IC} := V_w(w_{snow}) = 0.153\ kip$ (snow load)

Factored and Combined Girder End Shears --

$V_{uDC} := (V_{DC}) \cdot (\gamma_{DC}) = 0.037\ kip$ (component dead load)

$V_{uLL} := (V_{LL}) \cdot (\gamma_{LL}) = 1.118\ kip$ (live load)

$V_{uIC} := (V_{IC}) \cdot (\gamma_{IC}) = 0.153\ kip$ (snow load)

$V_u := (\eta) \cdot (V_{uDC} + V_{uLL} + V_{uIC}) = 1.37\ kip$ (combined, Case 3 combination)

Shear Resistance (LRFD 8.7) --

Adjustment Factors for Reference Design Values and Adjusted Design Value (LRFD 8.4.4) --

$C_{KF.v} := \frac{2.5}{\phi_v} = 3.33$ (format conversion factor, LRFD 8.4.4.2)

$C_{M.v} := 0.97$ (wet service factor, t <= 4", LRFD Tbl. 8.4.4.3-1)

$C_{i.v} := 0.80$ (incising factor, LRFD Tbl. 8.4.4.7-1)

$C_{\lambda} = 0.80$ (time effect factor, Strength-I used, recalled)

$F_v := (F_{vo}) \cdot (C_{KF.v}) \cdot (C_{M.v}) \cdot (C_{i.b}) \cdot (C_{\lambda}) = 0.37\ ksi$ (adjusted design value, LRFD Eq. 8.4.4.1-2)

Components Under Shear (LRFD 8.7) --

$V_n := \frac{(F_v) \cdot (b_{plank}) \cdot (t_{plank})}{1.5} = 6.98\ kip$ (nominal resistance, LRFD Eq. 8.7-2)

$V_r := (\phi_v) \cdot (V_n) = 5.24\ kip$ (factored resistance, LRFD Eq. 8.7-1)

$CDR_v := \frac{V_r}{V_u} = 3.81$ >= 1.0, OKAY (capacity-to-demand ratio)

6. Bearing Perpendicular to Grain Demand and Resistance:

Compression Demand --

Factored and Combined Girder End Compression --

$P_u := V_u = 1.37\ kip$ (factored compression)

Compression Resistance (LRFD 8.8) --

Adjustment Factors for Reference Design Values and Adjusted Design Value (LRFD 8.4.4) --

$C_{KF.cp} := \frac{2.1}{\phi_{cp}} = 2.33$ (format conversion factor, LRFD 8.4.4.2)

$C_{M.cp} := 0.67$ (wet service factor, LRFD Tbl. 8.4.4.3-1)

$C_{i.cp} := 1.00$ (incising factor, LRFD Tbl. 8.4.4.7-1)

$C_{\lambda} = 0.80$ (time effect factor, Strength-I, recalled)

$F_{cp} := (F_{cpo}) \cdot (C_{KF.cp}) \cdot (C_{M.cp}) \cdot (C_{i.cp}) \cdot (C_{\lambda}) = 0.78\ ksi$ (adjusted design value, LRFD Eq. 8.4.4.1-5)

Compression Perpendicular to Grain (LRFD 8.8.3) --

$L_b := \frac{b_{fb}}{2} = 1.75\ in$ (length of bearing along grain, assuming decking spliced at floorbeam)

$A_b := (b_{plank})\ (L_b) = 19.69\ in^2$ (bearing area)

$C_b := 1.0$ (adjustment factor for bearing, w/ in 3" of end, LRFD Tbl. 8.8.3-1)

$P_n := (F_{cp}) \cdot (A_b) \cdot (C_b) = 15.39\ kip$ (nominal resistance, LRFD Eq. 8.8.3-1)

$P_r := (\phi_{cp})\ (P_n) = 13.85\ kip$ (factored resistance, LRFD Eq. 8.8.3-1)

$CDR_{cp} := \frac{P_r}{P_u} = 10.08$ >= 1.0, OKAY (capacity-to-demand ratio)

WESTERN ALASKA BOARDWALK ASP
HANDRAIL and SUBSTRUCTURE CONNECTION CALCULATIONS - IBC LOADING

Narrative:

This file is for the load demands and resistances for the rails, rail posts, and sway bracing. Pedestrian live loadings on the rails are per the International Building Code.

Table of Contents:

1. References:

AASHTO LRFD Bridge Design Specifications, 9th Ed. (LRFD)
AASHTO LRFD Guide Specifications for the Design of Pedestrian Bridges (AASHTO Ped)
International Building Code, XX Ed. (IBC)
National Design Specification, 2018 Ed. (NDS)
Alaska Bridges and Structures Manual (ABSM)

2. Constants/Input Information:

General --

$kcf := 1000\ pcf$ (unit definition)

Geometry --

Deck (3 x 12, S4S) --

$t_{dk} := 2.5\ in$ (thickness)

$b_{wlk} := 10\ ft + 0\ in = 10.00\ ft$

Floorbeam (4 x 12, S4S) --

$s_{fb} := 6\ ft + 6\ in = 6.50\ ft$ (spacing)

$h_{fb} := 11.25\ in$ (height)

$b_{fb} := 3.5\ in$ (width)

$L_{fb} := 12\ ft + 0\ in = 12.00\ ft$ (length)

Posts (4 x 6, S4S) --

$s_{post} := s_{fb} = 6.50\ ft$ (spacing)

$h_{post.dk} := 42\ in = 3.50\ ft$ (height of post + rail cap, above top of deck)

$b_{post} := 3.5\ in$ (width)

$d_{post} := 5.5\ in$ (depth)

$A_{post} := (b_{post}) \cdot (d_{post}) = 19.25\ in^2$ (cross-sectional area)

$S_{x.post} := \dfrac{(b_{post}) \cdot (d_{post}^{\ 2})}{6} = 17.65\ in^3$ (section modulus, about axis parallel to boardwalk)

$S_{y.post} := \dfrac{(d_{post}) \cdot (b_{post}^{\ 2})}{6} = 11.23\ in^3$ (section modulus, about axis perpendicular to boardwalk)

$I_{post} := \dfrac{(b_{post}) \cdot (d_{post}^{\ 3})}{12} = 48.53\ in^4$ (moment of inertia)

$d_{h.post} := 0.5\ in$ (bolt hole diameter in connection)

$S_{x.post.mod} := S_{x.post} - \dfrac{(d_{h.post}) \cdot (d_{post}^{\ 2})}{6} = 15.13\ in^3$ (modified section modulus to account for bolt hole)

Rail Cap (2 x 8, S4S) --

$t_{rc} := 1.5\ in$ (thickness)

$b_{rc} := 7.25\ in$ (width)

$A_{rc} := (b_{rc}) \cdot (t_{rc}) = 10.88\ in^2$ (cross-sectional area)

$S_{rc} := \frac{(b_{rc}) \cdot (t_{rc}^{\,2})}{6} = 2.72\ in^3$ (section modulus)

$I_{rc} := \frac{(b_{rc}) \cdot (t_{rc}^{\,3})}{12} = 2.04\ in^4$ (moment of inertia)

$y_{rc} := h_{post.dk} - \frac{t_{rc}}{2} = 3.44\ ft$ (centroid of rail cap above top of deck)

Rails (2 x 6, S4S) --

$N_r := 4$ (number)

$h_r := 5.5\ in$ (height)

$b_r := 1.5\ in$ (width)

$gap_r := 3.75\ in$ < 4" IBC requirement
< 6" LRFD requirement
(open gap between rails)

$y_r := y_{rc} - \frac{t_{rc} + h_r}{2} - (h_r + gap_r) \cdot \begin{bmatrix} 0 \\ 1 \\ 2 \\ 3 \end{bmatrix} = \begin{bmatrix} 3.15 \\ 2.38 \\ 1.60 \\ 0.83 \end{bmatrix} ft$ (centroid of rails above top of deck)

$A_r := (b_r) \cdot (h_r) = 8.25\ in^2$ (cross-sectional area)

$S_{x.r} := \frac{(b_r) \cdot (h_r^{\,2})}{6} = 7.56\ in^3$ (section modulus, about horizontal axis)

$S_{y.r} := \frac{(h_r) \cdot (b_r^{\,2})}{6} = 2.06\ in^3$ (section modulus, about vertical axis)

$I_{x.r} := \frac{(b_r) \cdot (h_r^{\,3})}{12} = 20.80\ in^4$ (moment of inertia, about horizontal axis)

$I_{y.r} := \frac{(h_r) \cdot (b_r^{\,3})}{12} = 1.55\ in^4$ (moment of inertia, about vertical axis)

Curb Spacers and Curbs --

Curb Spacers (3 x 6) --

$h_{ks} := 2.5\ in$ (height of curb spacer)

$y_{ks} := \frac{h_{ks}}{2} = 0.10\ ft$ (centroid of curb spacer above top of deck)

Curbs (4 x 6) --

$h_k := 3.5\ in$ (height of curb)

$b_k := 5.5\ in$ (width of curb)

$S_{x.k} := \frac{(b_k) \cdot (h_k^{\ 2})}{6} = 11.23\ in^3$ (section modulus, about horizontal axis)

$S_{y.k} := \frac{(h_k) \cdot (b_k^{\ 2})}{6} = 17.65\ in^3$ (section modulus, about vertical axis)

$I_{x.k} := \frac{(b_k) \cdot (h_k^{\ 3})}{12} = 19.65\ in^4$ (moment of inertia, about horizontal axis)

$I_{y.k} := \frac{(h_k) \cdot (b_k^{\ 3})}{12} = 48.53\ in^4$ (moment of inertia, about vertical axis)

$y_k := h_{ks} + \frac{h_k}{2} = 0.35\ ft$ (centroid of curb above top of deck)

Post Connection --

See Rail Post Connect calculations.

Material (Wood) --

Wood --

Timber Reference Design Values (Douglas Fir-Larch, No. 1 or better, LRFD Tbl. 8.4.1.1.4-1) --

The posts, rail caps, and rails are classified as "dimension lumber"

$F_{bo} := 1.200\ ksi$ (bending)

$F_{vo} := 0.180\ ksi$ (shear parallel to grain)

$F_{cpo} := 0.625\ ksi$ (compression perpendicular to grain)

$E_o := 1800\ ksi$ (modulus of elasticity)

$\gamma_w := 0.050\ kcf$ (unit weight, LRFD Tbl. 3.5.1-1)

$G_w := 0.50$ (assigned specific gravity, NDS Tbl 12.3.3A)

Adjusted Design Value for Modulus of Elasticity --

$C_{M.E} := 0.90$ (wet service factor, LRFD Tbl. 8.4.4.3-1)

$C_{i.E} := 0.95$ (incising factor, LRFD Tbl. 8.4.4.7-1)

$E := \left(E_o\right) \cdot \left(C_{M.E}\right) \cdot \left(C_{i.E}\right) = 1539\ ksi$ (modulus of elasticity, LRFD Eq. 8.4.4.1-6)

Bolts (ASTM A307 Gr. A) --

$F_{yb} := 36\ ksi$ (yield stress, not required but generally taken as this per Gr. C)

$F_{ub} := 60\ ksi$ (tensile strength, LRFD 6.4.3.2)

Load Modifiers (LRFD 1.3.1.2) --

$\eta_D := 1.00$ (ductility factor, conventional design, LRFD 1.3.3)

$\eta_R := 1.05$ (redundancy factor, rail components are nonredundant, LRFD 1.3.4)

$\eta_I := 1.00$ (operational importance factor, typical bridge, LRFD 1.3.5)

$\eta := \left(\eta_D\right) \cdot \left(\eta_R\right) \cdot \left(\eta_I\right) = 1.05$ (load modifier, max. γi appropriate, LRFD Eq. 1.3.2.1-1)

Load Factors (LRFD Tbls. 3.4.1-1 & 2) --

$\gamma_{DC} := 1.25$ (component dead load)

$\gamma_{LL1} := 1.75$ (live load, forces on rail, in Strength I limit state)

$\gamma_{LL5} := 1.35$ (live load, forces on rail, in Strength V limit state, ABSM 12.1.3.5)

$\gamma_{WS3} := 1.00$ (wind load on structure, in Strength III limit state)

$\gamma_{WS5} := 0.40$ (wind load on structure, in Strength V limit state, ABSM 12.1.3.5)

Resistance Factors --

Wood (LRFD 8.5.2.2) --

$\phi_f := 0.85$ (flexure)

$\phi_v := 0.75$ (shear)

$\phi_{cp} := 0.90$ (compression perpendicular to grain)

$\phi_{conn} := 0.65$ (connections)

Steel (LRFD 6.5.4.2) --

$\phi_u := 0.80$ (tension, facture in net section)

$\phi_y := 0.95$ (tension, yield in gross section)

$\phi_s := 0.75$ (bolts in shear, A307)

$\phi_{bb} := 0.80$ (bolts bearing on material)

3. Loads (from Loads.mcdx):

Live Loads on Rail (LLr) --

Use maximum effects due to distributed or concentrated loads.

$w_{LLr} := 0.050\ klf$ (distributed load, transversely or vertically)

$P_{LLr} := 0.20\ kip$ (concentrated load)

Wind Load on Structure (WS) --

$P_Z := 65.2\ psf$ (pressure)

$w_{WSpost} := (b_{post}) \cdot (P_Z) = 19.0\ plf$ (distributed load, on post)

$w_{WSrc} := (t_{rc}) \cdot (P_Z) = 8.2\ plf$ (distributed load, on rail cap)

$w_{WSr} := (h_r) \cdot (P_Z) = 29.9\ plf$ (distributed load, on rail)

$w_{WSk} := (h_k) \cdot (P_Z) = 19.0\ plf$ (distributed load, on curb)

4. Rail Analyses:

Flexural Demand and Resistance --

Flexural Demand --

At mid-span between posts, assumed simply-supported.

Due to Self-Weight Dead Load on Rail --

$w_r := A_r \cdot \gamma_w = 2.86\ plf$ (distributed load)

$M_{DCr} := \frac{(w_r) \cdot (s_{post}^{\ 2})}{8} = 0.18\ in \cdot kip$ (unfactored moment, acting vertically only)

$M_{uDCr} := (M_{DCr}) \cdot (\gamma_{DC}) = 0.23\ in \cdot kip$ (unfactored moment, acting vertically only)

Due to Live Loads on Rail (horizontal load controls by inspection) --

Unfactored --

$M_{LLr.w} := \frac{(w_{LLr}) \cdot (s_{post}{}^2)}{8} = 3.17\ in \cdot kip$ (due to distributed load)

$M_{LLr.P} := \frac{(P_{LLr}) \cdot (s_{post})}{4} = 3.90\ in \cdot kip$ (due to concentrated load)

$M_{LLr.wP} := \max(M_{LLr.w}, M_{LLr.P}) = 3.90\ in \cdot kip$ (controlling load)

Revised Moment From Finite Element Model (Rail Model) --

$M_{LLr.fea} := 0.163 \cdot ft \cdot kip = 1.96\ in \cdot kip$

Factored --

$M_{uLL1r.wP} := (M_{LLr.fea}) \cdot (\gamma_{LL1}) = 3.42\ in \cdot kip$ (STR-I)

$M_{uLL5r.wP} := (M_{LLr.fea}) \cdot (\gamma_{LL5}) = 2.64\ in \cdot kip$ (STR-V)

Due to Wind Loads on Rail --

Unfactored --

$M_{WSr} := \frac{(w_{WSr}) \cdot (s_{post}{}^2)}{8} = 1.89\ in \cdot kip$ (acting transversely only)

Factored --

Strength III Limit State --

$M_{uWS3r} := (M_{WSr}) \cdot (\gamma_{WS3}) = 1.89\ in \cdot kip$ (acting transversely only)

Strength V Limit State --

$M_{uWS5r} := (M_{WSr}) \cdot (\gamma_{WS5}) = 0.76\ in \cdot kip$ (acting transversely only)

Due to Load Combinations --

Strength I Limit State (1.25*DC + 1.75*LL + 0.00*WS) --

$M_{u1.h} := (\eta) \cdot (M_{uLL1r.wP}) = 3.59\ in \cdot kip$ (transverse load)

$M_{u1.v} := (\eta) \cdot (M_{uDCr}) = 0.24\ in \cdot kip$ (vertical load)

Strength III Limit State (1.25*DC + 1.00*WS) --

Maximum horizontal controls by inspection.

$M_{u3.h} := (\eta) \cdot (M_{uWS3r}) = 1.99\ in \cdot kip$ (transverse load)

$M_{u3.v} := (\eta) \cdot (M_{uDCr}) = 0.24\ in \cdot kip$ (vertical load)

Strength V Limit State (1.25*DC + 1.35*LL + 0.40*WS) --

Maximum horizontal controls by inspection.

$M_{u5.h} := (\eta) \cdot (M_{uLL5r.wP} + M_{uWS5r}) = 3.57\ in \cdot kip$ (transverse load)

$M_{u5.v} := (\eta) \cdot (M_{uDCr}) = 0.24\ in \cdot kip$ (vertical load)

Flexural Resistances (LRFD 8.6) --

Adjustment Factors for Reference Design Values (LRFD 8.4.4) --

$C_{KF.b} := \frac{2.5}{\phi_f} = 2.94$ (format conversion factor, LRFD 8.4.4.2)

$C_{Fr.b} := 1.3$ (size factor, 2 x 6, LRFD Tbl. 8.4.4.4-1)

$C_{Mr.b} := \begin{array}{l} \text{if } (F_{bo}) \cdot (C_{Fr.b}) \le 1.15\ ksi \\ \quad 1.00 \\ \text{else} \\ \quad 0.85 \end{array} = 0.85$ (wet service factor, t < 4", LRFD Tbl. 8.4.4.3-1)

$C_{fu.rh} := 1.15$ horizontal loading
$C_{fu.rv} := 1.00$ vertical loading
(flat-use factor, width = 6", thickness = 2", LRFD Tbl. 8.4.4.6-1)

$C_{i.b} := 0.80$ (incising factor, LRFD Tbl. 8.4.4.7-1)

$C_d := 1.0$ (non-stress or laminated deck, LRFD 8.4.4.8)

$C_{\lambda 1} := 0.8$ STR-I
$C_{\lambda 35} := 1.0$ STR-III & V (time effect factor, LRFD Tbl. 8.4.4.9-1)

Adjusted Design Values (LRFD Eq. 8.4.4.1-1)

Strength I Limit State --

$$F_{br1.h} := \left(F_{bo}\right) \cdot \left(C_{KF.b}\right) \cdot \left(C_{Mr.b}\right) \cdot \left(C_{Fr.b}\right) \cdot \left(C_{fu.rh}\right) \cdot \left(C_{i.b}\right) \cdot \left(C_d\right) \cdot \left(C_{\lambda 1}\right)$$ (horizontal)

$$F_{br1.h} = 2.87 \; ksi$$

$$F_{br1.v} := \left(F_{bo}\right) \cdot \left(C_{KF.b}\right) \cdot \left(C_{Mr.b}\right) \cdot \left(C_{Fr.b}\right) \cdot \left(C_{fu.rv}\right) \cdot \left(C_{i.b}\right) \cdot \left(C_d\right) \cdot \left(C_{\lambda 1}\right)$$ (vertical)

$$F_{br1.v} = 2.50 \; ksi$$

Strength III & V Limit States --

$$F_{br35.h} := \left(F_{bo}\right) \cdot \left(C_{KF.b}\right) \cdot \left(C_{Mr.b}\right) \cdot \left(C_{Fr.b}\right) \cdot \left(C_{fu.rh}\right) \cdot \left(C_{i.b}\right) \cdot \left(C_d\right) \cdot \left(C_{\lambda 35}\right)$$ (horizontal)

$$F_{br35.h} = 3.59 \; ksi$$

$$F_{br35.v} := \left(F_{bo}\right) \cdot \left(C_{KF.b}\right) \cdot \left(C_{Mr.b}\right) \cdot \left(C_{Fr.b}\right) \cdot \left(C_{fu.rv}\right) \cdot \left(C_{i.b}\right) \cdot \left(C_d\right) \cdot \left(C_{\lambda 35}\right)$$ (vertical)

$$F_{br35.v} = 3.12 \; ksi$$

Components in Flexure, Rectangular Section (LRFD 8.6.2) --

About Horizontal Axis (Vertical Loading) --

Elastic Buckling Stress --

$$L_{e.r} := \begin{array}{l} L_u \leftarrow s_{post} \\ \lambda_r \leftarrow \dfrac{L_u}{h_r} \\ \text{if } \lambda_r < 7 \\ \quad 2.06 \cdot L_u \\ \text{else if } 7 \le \lambda_r \le 14.3 \\ \quad 1.63 \cdot L_u + 3 \cdot h_r \\ \text{else} \\ \quad 1.84 \cdot L_u \end{array} = 11.97 \; ft$$

$$R_{br.v} := min\left(\sqrt{\frac{L_{e.r} \cdot h_r}{{b_r}^2}}, 50\right) = 18.74$$ (LRFD Eq. 8.6.2-5)

$K_{bE} := 0.76$ (Euler buckling coefficient, visually graded lumber, LRFD 8.6.2)

$$F_{bEr.v} := \frac{K_{bE} \cdot E}{{R_{br.v}}^2} = 3.33 \; ksi$$ (elastic buckling stress, LRFD Eq. 8.6.2-4)

Parameters for Beam Stability (LRFD Eq. 8.6.2-3) --

$$A_{r1.v} := \frac{F_{bEr.v}}{F_{br1.v}} = 1.33$$ (STR-I)

$$A_{r35.v} := \frac{F_{bEr.v}}{F_{br35.v}} = 1.07$$ (STR-III & V)

Beam Stability Factors (LRFD Eq. 8.6.2-2) --

$$C_{Lr1.v} := \frac{1 + A_{r1.v}}{1.9} - \sqrt{\frac{\left(1 + A_{r1.v}\right)^2}{3.61} - \frac{A_{r1.v}}{0.95}} = 0.90$$ (STR-I)

$$C_{Lr35.v} := \frac{1 + A_{r35.v}}{1.9} - \sqrt{\frac{\left(1 + A_{r35.v}\right)^2}{3.61} - \frac{A_{r35.v}}{0.95}} = 0.84$$ (STR-III & V)

Nominal Resistances (LRFD Eq. 8.6.2-1) --

$M_{n.r1.v} := (F_{br1.v}) \cdot (S_{x.r}) \cdot (C_{Lr1.v}) = 17.08\ in \cdot kip$ (STR-I)

$M_{n.r35.v} := (F_{br35.v}) \cdot (S_{x.r}) \cdot (C_{Lr35.v}) = 19.88\ in \cdot kip$ (STR-III & V)

Factored Resistances (LRFD Eq. 8.6.1-1) --

$M_{r1.v} := (\phi_f) \cdot (M_{n.r1.v}) = 14.52\ in \cdot kip$ (STR-I)

$M_{r35.v} := (\phi_f) \cdot (M_{n.r35.v}) = 16.90\ in \cdot kip$ (STR-III & V)

Capacity-to-Demand Ratios --

Strength I Limit State --

$CDR_{fr1.v} := \frac{M_{r1.v}}{M_{u1.v}} = 60.92$ >= 1.00 OKAY

Strength III Limit State --

$CDR_{fr3.v} := \frac{M_{r35.v}}{M_{u3.v}} = 70.91$ >= 1.00 OKAY

Strength V Limit State --

$CDR_{fr5.v} := \frac{M_{r35.v}}{M_{u5.v}} = 70.91$ >= 1.00 OKAY

About Vertical Axis (Horizontal Loading) --

Beam Stability Factors (LRFD Eq. 8.6.2-2) --

Loading on wide face.

$C_{Lr1.h} := 1.00$ (STR-I)

$C_{Lr35.h} := 1.00$ (STR-III & V)

Nominal Resistances (LRFD Eq. 8.6.2-1) --

$M_{n.r1.h} := \left(F_{br1.h}\right) \cdot \left(S_{y.r}\right) \cdot \left(C_{Lr1.h}\right) = 5.92\ in \cdot kip$ (STR-I)

$M_{n.r35.h} := \left(F_{br35.h}\right) \cdot \left(S_{y.r}\right) \cdot \left(C_{Lr35.h}\right) = 7.40\ in \cdot kip$ (STR-III & V)

Factored Resistances (LRFD Eq. 8.6.1-1) --

$M_{r1.h} := \left(\phi_f\right) \cdot \left(M_{n.r1.h}\right) = 5.03\ in \cdot kip$ (STR-I)

$M_{r35.h} := \left(\phi_f\right) \cdot \left(M_{n.r35.h}\right) = 6.29\ in \cdot kip$ (STR-III & V)

Capacity-to-Demand Ratios --

Strength I Limit State --

$CDR_{fr1.h1} := \dfrac{M_{r1.h}}{M_{u1.h}} = 1.40$ >= 1.00 OKAY (Case 1)

Strength III Limit State --

$CDR_{fr3.h} := \dfrac{M_{r35.h}}{M_{u3.h}} = 3.16$ >= 1.00 OKAY

Strength V Limit State --

$CDR_{fr5.h} := \dfrac{M_{r35.h}}{M_{u5.h}} = 1.76$ >= 1.00 OKAY

Shear Demand and Resistance --

Shear Demand --

Unfactored --

At posts, assumed simply-supported.

Due to Self-Weight Dead Load on Rail (vertical only) --

$V_{DC} := \dfrac{\left(w_r\right) \cdot \left(s_{post}\right)}{2} = 9.3\ lbf$

Due to Live Load on Rail (either vertical or horizontal) --

$V_{LL} := \max\left(P_{LLr}, \left(w_{LLr}\right) \cdot \left(\dfrac{s_{post}}{2}\right)\right) = 200.0\ lbf$

Due to Wind Load on Rail (horizontal only) --

$$V_{WS} := \frac{(w_{WSr}) \cdot (s_{post})}{2} = 97.1\ lbf$$

Factored --

Strength I Limit State (1.25*DC + 1.75*LL + 0.00*WS) --

$$V_{u1.r} := (V_{DC}) \cdot (\gamma_{DC}) + (V_{LL}) \cdot (\gamma_{LL1}) = 361.6\ lbf$$

(vertical loading controls by inspection)

Strength III Limit State (1.25*DC + 1.00*WS) --

$$V_{u3.r.vert} := (V_{DC}) \cdot (\gamma_{DC}) = 11.6\ lbf$$

(acting vertically)

$$V_{u3.r.horz} := (V_{WS}) \cdot (\gamma_{WS3}) = 97.1\ lbf$$

(acting horizontally)

$$V_{u3.r} := \sqrt{V_{u3.r.vert}^{\ 2} + V_{u3.r.horz}^{\ 2}} = 97.8\ lbf$$

(resultant)

$$\theta_{Vu3.r} := \operatorname{atan}\left(\frac{V_{u3.r.vert}}{V_{u3.r.horz}}\right) = 6.83\ deg$$

(angle of resultant)

Strength V Limit State (1.25*DC + 1.35*LL + 0.40*WS) --

$$V_{u5.r.vert} := (V_{DC}) \cdot (\gamma_{DC}) + (V_{LL}) \cdot (\gamma_{LL5}) = 281.6\ lbf$$

(acting vertically)

$$V_{u5.r.horz} := (V_{WS}) \cdot (\gamma_{WS5}) = 38.8\ lbf$$

(acting horizontally)

$$V_{u5.r} := \sqrt{V_{u5.r.vert}^{\ 2} + V_{u5.r.horz}^{\ 2}} = 284.3\ lbf$$

(resultant)

$$\theta_{Vu5.r} := \operatorname{atan}\left(\frac{V_{u5.r.vert}}{V_{u5.r.horz}}\right) = 82.15\ deg$$

(angle of resultant)

Components Under Shear (LRFD 8.7) --

Adjustment Factors for Reference Design Values and Adjusted Design Value (LRFD 8.4.4) --

$$C_{KF.v} := \frac{2.5}{\phi_v} = 3.33$$

(format conversion factor, LRFD 8.4.4.2)

$$C_{M.v.r} := 0.97$$

(wet service factor, t < 4", LRFD Tbl. 8.4.4.3-1)

$C_{i.v} := 0.80$ (incising factor, LRFD Tbl. 8.4.4.7-1)

$C_{\lambda 1} = 0.80$ (time effect factor, Strength-I, recalled)

$C_{\lambda 35} = 1.00$ (time effect factor, Strength-III & V, recalled)

$F_{v.r1} := (F_{vo}) \cdot (C_{KF.v}) \cdot (C_{M.v.r}) \cdot (C_{i.v}) \cdot (C_{\lambda 1}) = 0.37\ ksi$ (adjusted design value, Strength-I, LRFD Eq. 8.4.4.1-2)

$F_{v.r35} := (F_{vo}) \cdot (C_{KF.v}) \cdot (C_{M.v.r}) \cdot (C_{i.v}) \cdot (C_{\lambda 35}) = 0.47\ ksi$ (adjusted design value, Strength-III & V, LRFD Eq. 8.4.4.1-2)

Components Under Shear (LRFD 8.7) --

Nominal Resistance (LRFD Eq. 8.7-2) --

Strength I Limit State (vertical only) --

$V_{n1.r} := \dfrac{(F_{v.r1}) \cdot (b_r) \cdot (h_r)}{1.5} = 2049\ lbf$ (nominal resistance, LRFD Eq. 8.7-2)

Strength III Limit State (combined vertical and horizontal) --

$I_{r3} := 8.738 \cdot 10^{-5} \cdot ft^4 = 1.81\ in^4$ (rotated moment of inertia)

$y_{r3} := 0.406\ in$ (vertical distance from centroid of area above rail centroid to rail centroid)

$Q_{r3} := (0.5) \cdot (A_r) \cdot (y_{r3}) = 1.67\ in^3$ (moment area, from CAD)

$b_{r3} := 5.54\ in$ (width of rotated section though centroid)

$V_{n3.r} := \dfrac{(F_{v.r35}) \cdot (I_{r3}) \cdot (b_{r3})}{Q_{r3}} = 2791\ lbf$ (nominal resistance)

Strength V Limit State (combined vertical and horizontal) --

$I_{r5} := 9.861 \cdot 10^{-4} \cdot ft^4 = 20.45\ in^4$ (rotated moment of inertia)

$y_{r5} := 1.375\ in$ (vertical distance from centroid of area above rail centroid to rail centroid)

$Q_{r5} := (0.5) \cdot (A_r) \cdot (y_{r5}) = 5.67\ in^3$ (moment area, from CAD)

$b_{r5} := 1.51\ in$ (width of rotated section though centroid)

$V_{n5.r} := \dfrac{(F_{v.r35}) \cdot (I_{r5}) \cdot (b_{r5})}{Q_{r5}} = 2535\ lbf$ (nominal resistance)

Factored Resistance (LRFD Eq. 8.7-1) --

Strength I Limit State --

$V_{r1.r} := (\phi_v) \cdot (V_{n1.r}) = 1536\ lbf$

$\dfrac{V_{r1.r}}{V_{u1.r}} = 4.25$ > 1.00 OK

Strength III Limit State --

$V_{r3.r} := (\phi_v) \cdot (V_{n3.r}) = 2093\ lbf$

$\dfrac{V_{r3.r}}{V_{u3.r}} = 21.40$ > 1.00 OK

Strength V Limit State --

$V_{r5.r} := (\phi_v) \cdot (V_{n5.r}) = 1901\ lbf$

$\dfrac{V_{r5.r}}{V_{u5.r}} = 6.69$ > 1.00 OK

5. Post Analyses:

Flexural Demand and Resistance --

Flexural Demand --

Due to Dead Loads --

Dead load moments negligible by inspection.

$M_{uDCpost} := 0\ in \cdot kip$ (moment)

Due to Live Loads on Rail (@ centroid of upper longitudinal element, LRFD 13.8.2) --

Unfactored --

$w_{LLr} = 50\ plf$ (distributed load, recalled)

$P_{LLr.w} := (w_{LLr}) \cdot (s_{post}) = 325\ lbf$ (due to distributed load)

$P_{LLr} = 200\ lbf$ (due to concentrated load, recalled)

$P_{LL.post} := \max(P_{LLr.w}, P_{LLr}) = 325\ lbf$ (controlling horizontal load)

$y_{LL} := h_{post.dk} - y_k - (0.5) \cdot (t_{rc}) = 3.08\ ft$ (distance from centroid of rail cap to top bolt in connection to curb)

$M_{LLpost} := (P_{LL.post}) \cdot (y_{LL}) = 12.03\ in \cdot kip$ (moment)

Factored --

$M_{uLLpost1} := (M_{LLpost}) \cdot (\gamma_{LL1}) = 21.04\ in \cdot kip$ (STR-I)

$M_{uLLpost5} := (M_{LLpost}) \cdot (\gamma_{LL5}) = 16.23\ in \cdot kip$ (STR-V)

Due to Wind Load on Structure --

Centroids of Force Above Top Bolt in Connection to Curb --

$y_{WSrc} := y_{LL} = 3.08\ ft$ (rail cap)

$$y_{WSr} := y_r - y_k = \begin{bmatrix} 2.79 \\ 2.02 \\ 1.25 \\ 0.48 \end{bmatrix} ft$$ (rails)

$$y_{WSpost} := \begin{bmatrix} \text{mean}\left(y_{WSr_1}, y_{WSr_2}\right) \\ \text{mean}\left(y_{WSr_2}, y_{WSr_3}\right) \\ \text{mean}\left(y_{WSr_3}, y_{WSr_4}\right) \\ \text{mean}\left(y_{WSr_4} - \frac{h_r}{2}, \frac{h_k}{2}\right) \end{bmatrix} = \begin{bmatrix} 2.41 \\ 1.64 \\ 0.86 \\ 0.20 \end{bmatrix} ft$$

Unfactored --

Transverse Loads --

$P_{WSpost.rc} := \left(w_{WSrc}\right) \cdot \left(s_{post}\right) = 53\ lbf$ (on rail cap)

$P_{WSpost.r} := \left(w_{WSr}\right) \cdot \left(s_{post}\right) = 194\ lbf$ (per rail)

$$h_{post.exp} := \begin{bmatrix} gap_r \\ gap_r \\ gap_r \\ \left(y_{WSr_4} - \frac{h_r}{2}\right) - \left(\frac{h_k}{2}\right) \end{bmatrix} = \begin{bmatrix} 0.31 \\ 0.31 \\ 0.31 \\ 0.10 \end{bmatrix} ft$$

(exposed height of post between rails and curb)

$$P_{WSpost.post} := \left(w_{WSpost}\right) \cdot \left(h_{post.exp}\right) = \begin{bmatrix} 6 \\ 6 \\ 6 \\ 2 \end{bmatrix} lbf$$

(on post)

Moments --

$M_{WSpost.rc} := \left(y_{WSrc}\right) \cdot \left(P_{WSpost.rc}\right) = 1.96\ in \cdot kip$ (due to rail cap)

$M_{WSpost.r} := \left(\sum y_{WSr}\right) \cdot \left(P_{WSpost.r}\right) = 15.25\ in \cdot kip$ (due to rails)

$M_{WSpost.post} := \left(y_{WSpost}\right) \cdot \left(P_{WSpost.post}\right) = 0.35\ in \cdot kip$ (due to post, dot product)

$M_{WSpost} := M_{WSpost.rc} + M_{WSpost.r} + M_{WSpost.post} = 17.56\ in \cdot kip$ (total)

Factored --

$M_{uWSpost3} := \left(M_{WSpost}\right) \cdot \left(\gamma_{WS3}\right) = 17.56\ in \cdot kip$ (STR-III)

$M_{uWSpost5} := \left(M_{WSpost}\right) \cdot \left(\gamma_{WS5}\right) = 7.03\ in \cdot kip$ (STR-V)

Due to Load Combinations --

Strength I Limit State (1.25*DC + 1.75*LL + 0.00*WS) --

$$M_{u1.post} := (\eta) \cdot (M_{uDCpost} + M_{uLLpost1}) = 22.10\ in \cdot kip$$

Strength III Limit State (1.25*DC + 1.00*WS) --

$$M_{u3.post} := (\eta) \cdot (M_{uDCpost} + M_{uWSpost3}) = 18.44\ in \cdot kip$$

Strength V Limit State (1.25*DC + 1.35*LL + 0.40*WS) --

$$M_{u5.post} := (\eta) \cdot (M_{uDCpost} + M_{uLLpost5} + M_{uWSpost5}) = 24.42\ in \cdot kip$$

Flexural Resistances (LRFD 8.6) --

Adjustment Factors for Reference Design Values (LRFD 8.4.4) --

$C_{KF.b} = 2.94$ (format conversion factor, LRFD 8.4.4.2, recalled)

$C_{Fpost.b} := 1.3$ (size factor, 4 x 6, LRFD Tbl. 8.4.4.4-1)

$$C_{Mpost.b} := \begin{cases} \text{if } (F_{bo}) \cdot (C_{Fpost.b}) \le 1.15\ ksi \\ \quad 1.00 \\ \text{else} \\ \quad 0.85 \end{cases} = 0.85$$

(wet service factor, t = 4", LRFD Tbl. 8.4.4.3-1)

$C_{fu.post} := 1.15$ (flat-use factor, loaded on narrow face, LRFD 8.4.4.6)

$C_{i.b} = 0.80$ (incising factor, LRFD Tbl. 8.4.4.7-1, recalled)

$C_d = 1.0$ (non-stress or laminated deck, LRFD 8.4.4.8, recalled)

$C_{\lambda 1} = 0.8$ STR-I
$C_{\lambda 35} = 1.0$ STR-III & V
(time effect factors, LRFD Tbl. 8.4.4.9-1, recalled)

Adjusted Design Values (LRFD Eq. 8.4.4.1-1)

Strength I Limit State --

$$F_{bpost1} := \left(F_{bo}\right) \cdot \left(C_{KF.b}\right) \cdot \left(C_{Mr.b}\right) \cdot \left(C_{Fr.b}\right) \cdot \left(C_{fu.rh}\right) \cdot \left(C_{i.b}\right) \cdot \left(C_{d}\right) \cdot \left(C_{\lambda 1}\right)$$

$$F_{bpost1} = 2.87\ ksi$$

Strength III & V Limit States --

$$F_{bpost35} := \left(F_{bo}\right) \cdot \left(C_{KF.b}\right) \cdot \left(C_{Mr.b}\right) \cdot \left(C_{Fr.b}\right) \cdot \left(C_{fu.rh}\right) \cdot \left(C_{i.b}\right) \cdot \left(C_{d}\right) \cdot \left(C_{\lambda 35}\right)$$

$$F_{bpost35} = 3.59\ ksi$$

Components in Flexure, Rectangular Section (LRFD 8.6.2) --

About Bridge Longitudinal Axis (Transverse Loading) --

Elastic Buckling Stress --

$$L_{e.post} := (2) \cdot \left(h_{post.dk} - y_k\right) = 6.29\ ft$$ (effective unbraced length, cantilever)

$$R_{bpost} := min\left(\sqrt{\frac{L_{e.post} \cdot d_{post}}{b_{post}^{2}}}, 50\right) = 5.82$$ (LRFD Eq. 8.6.2-5)

$$K_{bE} = 0.76$$ (Euler buckling coefficient, visually graded lumber, LRFD 8.6.2, recalled)

$$F_{bEpost} := \frac{K_{bE} \cdot E}{R_{bpost}^{2}} = 34.50\ ksi$$ (elastic buckling stress, LRFD Eq. 8.6.2-4)

Parameters for Beam Stability (LRFD Eq. 8.6.2-3) --

$$A_{post1} := \frac{F_{bEpost}}{F_{bpost1}} = 12.02$$ (STR-I)

$$A_{post35} := \frac{F_{bEpost}}{F_{bpost35}} = 9.62$$ (STR-III & V)

Beam Stability Factors (LRFD Eq. 8.6.2-2) --

$$C_{Lpost1} := \frac{1 + A_{post1}}{1.9} - \sqrt{\frac{\left(1 + A_{post1}\right)^{2}}{3.61} - \frac{A_{post1}}{0.95}} = 1.00$$ (STR-I)

$$C_{Lpost35} := \frac{1 + A_{post35}}{1.9} - \sqrt{\frac{\left(1 + A_{post35}\right)^2}{3.61} - \frac{A_{post35}}{0.95}} = 0.99$$ (STR-III & V)

Nominal Resistances (LRFD Eq. 8.6.2-1) --

$M_{n.post1} := \left(F_{bpost1}\right) \cdot \left(S_{x.post.mod}\right) \cdot \left(C_{Lpost1}\right) = 43.22\ in \cdot kip$ (STR-I)

$M_{n.post35} := \left(F_{bpost35}\right) \cdot \left(S_{x.post.mod}\right) \cdot \left(C_{Lpost35}\right) = 53.96\ in \cdot kip$ (STR-III & V)

Factored Resistances (LRFD Eq. 8.6.1-1) --

$M_{r.post1} := \left(\phi_f\right) \cdot \left(M_{n.post1}\right) = 36.74\ in \cdot kip$ (STR-I)

$M_{r.post35} := \left(\phi_f\right) \cdot \left(M_{n.post35}\right) = 45.86\ in \cdot kip$ (STR-III & V)

Capacity-to-Demand Ratios --

Strength I Limit State --

$CDR_{fpost1} := \dfrac{M_{r.post1}}{M_{u1.post}} = 1.66$ >= 1.00 OKAY

Strength III Limit State --

$CDR_{fpost3} := \dfrac{M_{r.post35}}{M_{u3.post}} = 2.49$ >= 1.00 OKAY

Strength V Limit State --

$CDR_{fpost5} := \dfrac{M_{r.post35}}{M_{u5.post}} = 1.88$ >= 1.00 OKAY

Shear Demand and Resistance --

Shear Demand --

Due to Dead Loads --

$V_{uDCpost} := 0\ lbf$ Not applicable.

Due to Live Loads on Rail --

Unfactored --

$V_{LLpost} := P_{LL.post} = 0.325\ kip$ (total horizontal load, recalled)

Factored --

$V_{uLLpost1} := (V_{LLpost}) \cdot (\gamma_{LL1}) = 0.57\ kip$ (STR-I)

$V_{uLLpost5} := (V_{LLpost}) \cdot (\gamma_{LL5}) = 0.44\ kip$ (STR-V)

Due to Wind Load on Structure --

Unfactored --

$V_{WSpost} := P_{WSpost.rc} + (N_r) \cdot (P_{WSpost.r}) + \sum P_{WSpost.post} = 850\ lbf$ (total)

Factored --

$V_{uWSpost3} := (V_{WSpost}) \cdot (\gamma_{WS3}) = 0.85\ kip$ (STR-III)

$V_{uWSpost5} := (V_{WSpost}) \cdot (\gamma_{WS5}) = 0.34\ kip$ (STR-V)

Due to Load Combinations --

Strength I Limit State (1.25*DC + 1.75*LL + 0.00*WS) --

$V_{u1.post} := (\eta) \cdot (V_{uDCpost} + V_{uLLpost1}) = 0.60\ kip$

Strength III Limit State (1.25*DC + 1.00*WS) --

$V_{u3.post} := (\eta) \cdot (V_{uDCpost} + V_{uWSpost3}) = 0.89\ kip$

Strength V Limit State (1.25*DC + 1.35*LL + 0.40*WS) --

$V_{u5.post} := (\eta) \cdot (V_{uDCpost} + V_{uLLpost5} + V_{uWSpost5}) = 0.82\ kip$

Components Under Shear (LRFD 8.7) --

Adjustment Factors for Reference Design Values and Adjusted Design Value (LRFD 8.4.4) --

$C_{KF.v} = 3.33$ (format conversion factor, LRFD 8.4.4.2, recalled)

$C_{M.v.post} := 0.97$ (wet service factor, t = 4", LRFD Tbl. 8.4.4.3-1)

$C_{i.v} = 0.80$ (incising factor, LRFD Tbl. 8.4.4.7-1, recalled)

$C_{\lambda 1} = 0.80$ (time effect factor, Strength-I, recalled)

$C_{\lambda 35} = 1.00$ (time effect factor, Strength-III & V, recalled)

$$F_{v1.post} := \left(F_{vo}\right) \cdot \left(C_{KF.v}\right) \cdot \left(C_{M.v.post}\right) \cdot \left(C_{i.v}\right) \cdot \left(C_{\lambda 1}\right) = 0.37\ \boldsymbol{ksi}$$

(adjusted design value, Strength-I, LRFD Eq. 8.4.4.1-2)

$$F_{v35.post} := \left(F_{vo}\right) \cdot \left(C_{KF.v}\right) \cdot \left(C_{M.v.post}\right) \cdot \left(C_{i.v}\right) \cdot \left(C_{\lambda 35}\right) = 0.47\ \boldsymbol{ksi}$$

(adjusted design value, Strength-III & V, LRFD Eq. 8.4.4.1-2)

Components Under Shear (LRFD 8.7) --

Nominal Resistance (LRFD Eq. 8.7-2) --

$$V_{n1.post} := \frac{\left(F_{v1.post}\right) \cdot \left(b_r\right) \cdot \left(h_r\right)}{1.5} = 2.05\ \boldsymbol{kip}$$

(Strength-I)

$$V_{n35.post} := \frac{\left(F_{v35.post}\right) \cdot \left(b_r\right) \cdot \left(h_r\right)}{1.5} = 2.56\ \boldsymbol{kip}$$

(Strength-III & V)

Factored Resistance (LRFD Eq. 8.7-2) --

$$V_{r1.post} := \left(\phi_v\right) \cdot \left(V_{n1.post}\right) = 1.54\ \boldsymbol{kip}$$

(Strength-I)

$$CDR_{v1.post} := \frac{V_{r1.post}}{V_{u1.post}} = 2.57$$

>= 1.0, OKAY (capacity-to-demand ratio)

$V_{r35.post} := (\phi_v) \cdot (V_{n35.post}) = 1.92\ kip$ (Strength-III & V)

$CDR_{v3.post} := \frac{V_{r35.post}}{V_{u3.post}} = 2.15$ >= 1.0, OKAY (Strength-III, capacity-to-demand ratio)

$CDR_{v5.post} := \frac{V_{r35.post}}{V_{u5.post}} = 2.35$ >= 1.0, OKAY (Strength-V, capacity-to-demand ratio)

WESTERN ALASKA BOARD ROAD ASP
FLOORBEAM CALCULATIONS
Case 3: 1.25*DC + 1.35*LL + 1.00*IC

Narrative:

This file checks the structural capacity of the floorbeams supported by helical pile or timber column substructures. Floorbeams supported by cribbing are acceptable by observation.
See Longitudinal Deck Planks_Case 3.mcdx for discussion on the Case 3 load combination.

Table of Contents:

1. References:

AASHTO LRFD Bridge Design Specifications, 9th Ed. (LRFD)
AASHTO LRFD Guide Specifications for the Design of Pedestrian Bridges,
2nd Ed. w/2015 Interims (AASHTO Ped)

2. Constants/Input Information:

General --

$kcf := 1000\ pcf$ (unit definition)

Geometry --

Span Length Between Floorbeams --

$L_{span} := 6\ ft + 6\ in = 6.50\ ft$

Walkway Cross-Section --

$L_{fb} := 12\ ft + 0\ in = 12.00\ ft$ (total length of floorbeam)

$S_{col} := 7\ ft + 0\ in = 7.00\ ft$ (spacing of "columns")

$L_{oh} := (0.5) \cdot (L_{fb} - S_{col}) = 2.50\ ft$ (length of floorbeam overhang)

$b_{wlk} := 9\ ft + 7\ in = 9.58\ ft$ (width of walkway, from Longitudinal Deck Planks_Case 3.mcdx)

Deck Planks (3 x 12 S4S) --

$t_{plank} := 2.5\ in$ (thickness)

$b_{dk} := 10\ ft + 6\ in = 10.50\ ft$ (total width of decking, from Longitudinal Deck Planks_Case 3.mcdx)

$A_{dk} := (t_{plank}) \cdot (b_{dk}) = 315.0\ in^2$ (total cross-sectional area)

Floorbeam Member (4 x 12 S4S) --

$b_{fb} := 3.5\ in$ (width)

$h_{fb} := 11.25\ in$ (height)

$A_{fb} := (b_{fb}) \cdot (h_{fb}) = 39.38\ in^2$ (cross-sectional area)

$S_{fb} := \dfrac{(b_{fb}) \cdot (h_{fb}^{\ 2})}{6} = 73.83\ in^3$ (section modulus)

$I_{fb} := \dfrac{(b_{fb}) \cdot (h_{fb}^{\ 3})}{12} = 415.3\ in^4$ (moment of inertia)

Beam Saddle --

$L_{saddle} := 6\ in$ (length)

Vehicle Live Load --

$cl_{wl} := 6\ in$ (clear distance from curb to wheel line, assumed)

$s_{wl} := 48\ in$ (wheel line spacing, Loads.mcdx)

Material (Wood) --

Timber Reference Design Values (Douglas Fir-Larch, No. 1 or better, LRFD Tbl. 8.4.1.1.4-1) --

The floorbeam is classified as "dimension lumber".

$F_{bo} := 1.200\ ksi$ (bending)

$F_{vo} := 0.180\ ksi$ (shear parallel to grain)

$F_{cpo} := 0.625\ ksi$ (compression perpendicular to grain)

$E_o := 1800\ ksi$ (modulus of elasticity)

$\gamma_w := 0.050\ kcf$ (unit weight, LRFD Tbl. 3.5.1-1)

Load Modifiers (LRFD 1.3.1.2) --

$\eta_D := 1.00$ (ductility factor, conventional design, LRFD 1.3.3)

$\eta_R := 1.05$ (redundancy factor, floorbeams are nonredundant, LRFD 1.3.4)

$\eta_I := 1.00$ (operational importance factor, typical bridge, LRFD 1.3.5)

$\eta := (\eta_D) \cdot (\eta_R) \cdot (\eta_I) = 1.05$ (load modifier, max. γi appropriate, LRFD Eq. 1.3.2.1-2)

Load Factors (LRFD Tbls. 3.4.1-1 & 2) --

$\gamma_{DC} := 1.25$ (component dead load)

$\gamma_{LL} := 1.35$ (live load, ATV and trailer per Strength II limit state used for Case 3 combination)

$\gamma_{IC} := 1.00$ (ice and snow accumulation per Extreme Event II limit state used for Case 3 combination)

Resistance Factors (LRFD 8.5.2.2) --

$\phi_f := 0.85$ (flexure)

$\phi_v := 0.75$ (shear)

$\phi_{cp} := 0.90$ (compression perpendicular to grain)

<u>3. Loads on Floorbeam:</u>

Floorbeam Self-Weight (DC) --

$w_{fb} := (A_{fb}) \cdot (\gamma_w) = 13.67\ \boldsymbol{plf}$ (unit weight of floorbeam)

Loads from Spans --

Span reactions will conservatively be based on the middle support of a 2-span continuous configuration using a finite element analysis in GT Strudl (2 Span Loads.gti).

Reaction Influence Values for Uniformly Distributed Loads in Spans --

$R_{infl} := 0.00750 \cdot \dfrac{\boldsymbol{kip}}{\boldsymbol{plf}}$ (Load 'LD_UNIF', jt. 135)

Component Dead Load (DC, deck planks) --

Note: The rails and curb weights are not included. This is conservative for mid-span moment calculations and do not contribute to shear between the supports.

$w_{dk.span} := (A_{dk}) \cdot (\gamma_w) = 109.4\ \boldsymbol{plf}$ (distributed load along span)

$w_{dk} := \dfrac{(w_{dk.span}) \cdot (R_{infl})}{b_{dk}} = 78.13\ \boldsymbol{plf}$ (distributed load along floorbeam)

Pedestrian Live Load (PL, LRFD Ped 3.1) --

$p_{ped} := 0.090\ \boldsymbol{ksf}$ (pressure)

$w_{ped.span} := (p_{ped}) \cdot (b_{wlk}) = 0.863\ \boldsymbol{klf}$ (distributed load along span)

$w_{ped} := \dfrac{(w_{ped.span}) \cdot (R_{infl})}{b_{dk}} = 616.07\ \boldsymbol{plf}$ (distributed load along floorbeam)

Vehicle Live Load (VL) --

$R_{wl} := 0.933\ kip = 933.0\ lbf$ (wheel line reaction, from GT Strudl, 2 Span Loads.gti, jt. 135)

Snow Load (IC) --

$p_f := 54\ psf$ (pressure, Loads.mcdx)

$w_{snow.span} := \left(p_f\right) \cdot \left(b_{dk}\right) = 0.567\ klf$ (distributed load along span)

$w_{snow} := \dfrac{\left(w_{snow.span}\right) \cdot \left(R_{infl}\right)}{b_{dk}} = 405.00\ plf$ (distributed load along floorbeam)

<u>4. Unfactored Results from GT Strudl Model (Floorbeam.gti):</u>

Column Reactions --

$R_{DC} := 492\ lbf$ (component dead load)

$R_{PL} := 3234\ lbf$ (pedestrian live load)

$R_{IC} := 2126\ lbf$ (snow load)

$R_{VL} := 1543\ lbf$ (vehicle live load)

$R_{max} := R_{DC} + R_{IC} + \max\left(R_{PL}, R_{VL}\right) = 5.85\ kip$ (total controlling unfactored reaction)

Moments @ Mid-Span Between Columns (jt. 13) --

$M_{DC} := 4799\ in \cdot lbf$ (component dead load)

$M_{PL} := 33961\ in \cdot lbf$ (pedestrian live load)

$M_{IC} := 22326\ in \cdot lbf$ (snow load)

$M_{VL} := 16794\ in \cdot lbf$ (vehicle live load)

Shears @ Columns (jt. 6) --

$V_{DC} := 321.3\ lbf$ (component dead load)

$V_{PL} := 2156.2\ lbf$ (pedestrian live load)

$V_{IC} := 1417.5\ lbf$ (snow load)

$V_{VL} := 1332.9\ lbf$ (vehicle live load)

5. Factored Moments and Shears:

Use Case 3 Combination: 1.25*DC + 1.35*LL + 1.00*IC.

Moments @ Mid-Span Between Columns --

$M_{uDC} := (M_{DC}) \cdot (\gamma_{DC}) = 6.00\ in \cdot kip$ (component dead load)

$M_{uLL} := \max(M_{PL}, M_{VL}) \cdot (\gamma_{LL}) = 45.85\ in \cdot kip$ (controlling live load)

$M_{uIC} := (M_{IC}) \cdot (\gamma_{IC}) = 22.33\ in \cdot kip$ (snow load)

$M_u := M_{uDC} + M_{uLL} + M_{uIC} = 74.17\ in \cdot kip$ (total moment)

Shears @ Columns --

$V_{uDC} := (V_{DC}) \cdot (\gamma_{DC}) = 0.40\ kip$ (component dead load)

$V_{uLL} := \max(V_{PL}, V_{VL}) \cdot (\gamma_{LL}) = 2.91\ kip$ (controlling live load)

$V_{uIC} := (V_{IC}) \cdot (\gamma_{IC}) = 1.42\ kip$ (snow load)

$V_u := V_{uDC} + V_{uLL} + V_{uIC} = 4.73\ kip$ (total shear)

Compression @ Column --

$P_{uDC} := (R_{DC}) \cdot (\gamma_{DC}) = 0.62\ kip$ (component dead load)

$P_{uLL} := \max(R_{PL}, R_{VL}) \cdot (\gamma_{LL}) = 4.37\ kip$ (controlling live load)

$P_{uIC} := (R_{IC}) \cdot (\gamma_{IC}) = 2.13\ kip$ (snow load)

$P_u := P_{uDC} + P_{uLL} + P_{uIC} = 7.11\ kip$ (total compression)

6. Flexural Resistance (LRFD 8.6):

Adjustment Factors for Reference Design Values and Adjusted Design Value (LRFD 8.4.4) --

$C_{KF.b} := \dfrac{2.5}{\phi_f} = 2.94$ (format conversion factor, LRFD 8.4.4.2)

$C_{F.b} := 1.1$ (size factor, 4 x 12, LRFD Tbl. 8.4.4.4-1)

$$C_{M.b} := \begin{array}{l} \text{if } (F_{bo}) \cdot (C_{F.b}) \le 1.15\ ksi \\ \quad 1.00 \\ \text{else} \\ \quad 0.85 \end{array} = 0.85$$

(wet service factor, t <= 4", LRFD Tbl. 8.4.4.3-1)

$C_{fu} := 1.00$ (flat-use factor, not applicable, LRFD Tbl. 8.4.4.6-1)

$C_{i.b} := 0.80$ (incising factor, LRFD Tbl. 8.4.4.7-1)

$C_d := 1.0$ (not applicable, LRFD 8.4.4.8)

$C_\lambda := 0.8$ (time effect factor, Strength-I, LRFD Tbl. 8.4.4.9-1)

$F_b := (F_{bo}) \cdot (C_{KF.b}) \cdot (C_{M.b}) \cdot (C_{F.b}) \cdot (C_{fu}) \cdot (C_{i.b}) \cdot (C_d) \cdot (C_\lambda)$ (adjusted design value, LRFD Eq. 8.4.4.1-1)

$F_b = 2.11\ ksi$

Components in Flexure, Rectangular Section (LRFD 8.6.2) --

$$L_e := \begin{array}{l} L_u \leftarrow S_{col} \\ \lambda_{fb} \leftarrow \dfrac{L_u}{h_{fb}} \\ \text{if } \lambda_{fb} < 7 \\ \quad 2.06 \cdot L_u \\ \text{else if } 7 \le \lambda_{fb} \le 14.3 \\ \quad 1.63 \cdot L_u + 3 \cdot h_{fb} \\ \text{else} \\ \quad 1.84 \cdot L_u \end{array} = 14.22\ ft$$

(effective unbraced length, LRFD 8.6.2)

$R_b := min\left(\sqrt{\frac{L_e \cdot h_{fb}}{{b_{fb}}^2}}, 50\right) = 12.52$ (LRFD Eq. 8.6.2-5)

$K_{bE} := 0.76$ (Euler buckling coefficient, visually graded lumber, LRFD 8.6.2)

$C_{M.E} := 0.90$ (wet service factor, LRFD Tbl. 8.4.4.3-1)

$C_{i.E} := 0.95$ (incising factor, LRFD Tbl. 8.4.4.7-1)

$E := (E_o) \cdot (C_{M.E}) \cdot (C_{i.E}) = 1539\ ksi$ (modulus of elasticity, LRFD Eq. 8.4.4.1-6)

$F_{bE} := \frac{K_{bE} \cdot E}{{R_b}^2} = 7.46\ ksi$ (elastic buckling stress, LRFD Eq. 8.6.2-4)

$A := \frac{F_{bE}}{F_b} = 3.53$ (parameter for beam stability, LRFD Eq. 8.6.2-3)

$C_L := \frac{1+A}{1.9} - \sqrt{\frac{(1+A)^2}{3.61} - \frac{A}{0.95}} = 0.98$ (beam stability factor, LRFD Eq. 8.6.2-2)

$M_n := (F_b) \cdot (S_{fb}) \cdot (C_L) = 152.98\ in \cdot kip$ (nominal resistance, LRFD Eq. 8.6.2-1)

$M_r := (\phi_f) \cdot (M_n) = 130.04\ in \cdot kip$ (factored resistance, LRFD Eq. 8.6.1-1)

$CDR_f := \frac{M_r}{M_u} = 1.75$ >= 1.0, OKAY (capacity-to-demand ratio)

7. Shear Resistance (LRFD 8.6):

Adjustment Factors for Reference Design Values and Adjusted Design Value (LRFD 8.4.4) --

$C_{KF.v} := \frac{2.5}{\phi_v} = 3.33$ (format conversion factor, LRFD 8.4.4.2)

$C_{M.v} := 0.97$ (wet service factor, t <= 4", LRFD Tbl. 8.4.4.3-1)

$C_{i.v} := 0.80$ (incising factor, LRFD Tbl. 8.4.4.7-1)

$C_{\lambda} = 0.80$ (time effect factor, Strength-I, recalled)

$F_v := (F_{vo}) \cdot (C_{KF.v}) \cdot (C_{M.v}) \cdot (C_{i.b}) \cdot (C_{\lambda}) = 0.37\ \boldsymbol{ksi}$ (adjusted design value, LRFD Eq. 8.4.4.1-2)

Components Under Shear (LRFD 8.7) --

$V_n := \dfrac{(F_v) \cdot (b_{fb}) \cdot (h_{fb})}{1.5} = 9.78\ \boldsymbol{kip}$ (nominal resistance, LRFD Eq. 8.7-2)

$V_r := (\phi_v) \cdot (V_n) = 7.33\ \boldsymbol{kip}$ (factored resistance, LRFD Eq. 8.7-1)

$CDR_v := \dfrac{V_r}{V_u} = 1.55$ >= 1.0, OKAY (capacity-to-demand ratio)

8. Compression Perpendicular to Grain Resistance (LRFD 8.8):

Adjustment Factors for Reference Design Values and Adjusted Design Value (LRFD 8.4.4) --

$C_{KF.cp} := \dfrac{2.1}{\phi_{cp}} = 2.33$ (format conversion factor, LRFD 8.4.4.2)

$C_{M.cp} := 0.67$ (wet service factor, LRFD Tbl. 8.4.4.3-1)

$C_{i.cp} := 1.00$ (incising factor, LRFD Tbl. 8.4.4.7-1)

$C_{\lambda} = 0.80$ (time effect factor, Strength-I, recalled)

$F_{cp} := (F_{cpo}) \cdot (C_{KF.cp}) \cdot (C_{M.cp}) \cdot (C_{i.cp}) \cdot (C_{\lambda}) = 0.78\ \boldsymbol{ksi}$ (adjusted design value, LRFD Eq. 8.4.4.1-5)

Compression Perpendicular to Grain (LRFD 8.8.3) --

$L_b := L_{saddle} = 6.00\ in$ (length of bearing along grain)

$A_b := \left(b_{fb}\right)\ \left(L_b\right) = 21.00\ in^2$ (bearing area)

$C_b := 1.00$ (adjustment factor for bearing, LRFD Tbl. 8.8.3-1)

$P_n := \left(F_{cp}\right) \cdot \left(A_b\right) \cdot \left(C_b\right) = 16.42\ kip$ (nominal resistance, LRFD Eq. 8.8.3-1)

$P_r := \left(\phi_{cp}\right)\ \left(P_n\right) = 14.77\ kip$ (factored resistance, LRFD Eq. 8.8.1-1)

$CDR_{cp} := \dfrac{P_r}{P_u} = 2.08$ >= 1.0, OKAY (capacity-to-demand ratio)

```
STRUDL 'WESTERN ALASKA BOARDWALK ASP'

$ FLOORBEAM MODEL

$ ENGINEER:  ERIC E. BONN, P.E.
$ COMPANY:  DOWL, LLC


$ ***** JOINT DEFINITIONS *****


UNITS FEET

JOINT COORDINATES

 1 -6.00 0.00 $ LEFT END OF FLOORBEAM
 2 -5.25 0.00 $ LEFT EDGE OF DECK
 3 -4.79 0.00 $ LEFT EDGE OF WALKWAY
 4 -4.29 0.00 $ LEFT EDGE OF WHEEL LINE RANGE
 5 -4.00 0.00
 6 -3.50 0.00 $ LEFT COLUMN
 7 -3.00 0.00
 8 -2.50 0.00
 9 -2.00 0.00
10 -1.50 0.00
11 -1.00 0.00
12 -0.50 0.00
13 -0.00 0.00 $ CENTERLINE
14  0.50 0.00
15  1.00 0.00
16  1.50 0.00
17  2.00 0.00
18  2.50 0.00
19  3.00 0.00
20  3.50 0.00 $ RIGHT COLUMN
21  4.00 0.00
22  4.29 0.00 $ RIGHT EDGE OF WHEEL LINE RANGE
23  4.79 0.00 $ RIGHT EDGE OF WALKWAY
24  5.25 0.00 $ RIGHT EDGE OF DECK
25  6.00 0.00 $ RIGHT END OF FLOORBEAM

STATUS SUPPORT 6 20

JOINT RELEASES
 6 MOMENT Z
20 FORCE X MOMENT Z


$ ***** MEMBER DEFINITIONS *****


TYPE PLANE FRAME

GENERATE 24 MEMBERS ID 'FB1' INC 1 FROM 1 INC 1 TO 2 INC 1

DEFINE GROUP 'gFLRBM' 'FLOORBEAM MEMBERS' MEMBERS 'FB1' TO 'FB24'

UNITS INCHES

MEMBER PROPERTIES PRISMATIC
 GROUP 'gFLRBM' AX 39.38 IZ 415.3

MATERIAL STEEL

UNITS KIP INCHES

CONSTANTS
 E 1539 GROUP 'gFLRBM'
```

```
$ ***** LOADINGS *****


UNITS LBS FEET

LOADING 'DC_FB' 'COMPONENT DEAD LOAD_FLOORBEAM'
 MEMBER LOADS
  GROUP 'gFLRBM' FORCE Y GLOBAL UNIFORM FRACTIONAL W -13.67 LA 0.0 LB 1.0

LOADING 'DC_DECK' 'COMPONENT DEAD LOAD_DECK'
 MEMBER LOADS
  'FB2' TO 'FB23' FORCE Y GLOBAL UNIFORM FRACTIONAL W -78.13 LA 0.0 LB 1.0

FORM LOADING 'DC' 'COMPONENT DEAD LOAD_TOTAL' FROM -
 'DC_FB' 1.0 'DC_DECK' 1.0

LOADING 'PL' 'PEDESTRIAN LIVE LOAD'
 MEMBER LOADS
  'FB2' TO 'FB23' FORCE Y GLOBAL UNIFORM FRACTIONAL W -616.07 LA 0.0 LB 1.0

LOADING 'IC' 'SNOW LOAD'
 MEMBER LOADS
  'FB2' TO 'FB23' FORCE Y GLOBAL UNIFORM FRACTIONAL W -405.00 LA 0.0 LB 1.0

$ VEHICLE LIVE LOAD - ATV W/TRAILER WHEEL LINE
MOVING LOAD GENERATOR
 SUPERSTRUCTURE N 2 MEMBER 'FB4' TO 'FB21'
 TRUCK LOAD DIRECTION FORWARD -
 VEHICLE GENERAL TRUCK 933.0 4.00 933.0
 GENERATE LOAD Y SCALE -1.00 INITIAL 'atv1'
END LOAD GENERATOR

DEFINE GROUP 'ATV' 'VEHICLE LIVE LOAD' LOADS 'atv18' TO 'atv37'

$ ***** RUN ANALYSIS AND OUTPUT RESULTS *****


STIFFNESS ANALYSIS

UNITS INCHES LBS

OUTPUT DECIMAL 1
LOAD LIST 'DC' 'PL' 'IC'
LIST REACTIONS JOINTS 6
LIST FORCES MEMBERS 'FB6' 'FB12'

LOAD LIST GROUP 'ATV'
LIST FORCE ENVELOPE MEMBERS EXISTING 'FB1' TO 'FB12' SECTION FR DS 0.0 1.0
LIST MAXIMUM REACTION ENVELOPE FOR LOADS ACTIVE
```

WESTERN ALASKA BOARD ROAD ASP
FLOORBEAM CALCULATIONS - ON CRIBBING
Case 3: 1.25*DC + 1.35*LL + 1.00*IC

Narrative:

This file checks the structural capacity of the floorbeams supported by helical pile or timber column substructures. Floorbeams supported by cribbing are acceptable by observation.
See Longitudinal Deck Planks_Case 3.mcdx for discussion on the Case 3 load combination.

Table of Contents:

1. References:

AASHTO LRFD Bridge Design Specifications, 9th Ed. (LRFD)
AASHTO LRFD Guide Specifications for the Design of Pedestrian Bridges,
2nd Ed. w/2015 Interims (AASHTO Ped)

2. Constants/Input Information:

General --

$kcf := 1000\ pcf$ (unit definition)

Geometry --

Span Length Between Floorbeams --

$L_{span} := 6\ ft + 6\ in = 6.50\ ft$

Walkway Cross-Section --

$L_{fb} := 12\ ft + 0\ in = 12.00\ ft$ (total length of floorbeam)

$S_{col} := 5\ ft + 0\ in = 5.00\ ft$ (spacing of "columns")

$L_{oh} := (0.5) \cdot (L_{fb} - 2 \cdot S_{col}) = 1.00\ ft$ (length of floorbeam overhang)

$b_{wlk} := 10\ ft + 5.375\ in = 10.45\ ft$ (width of walkway, Sht. 2)

Deck Planks (3 x 12 S4S) --

$t_{plank} := 2.5\ in$ (thickness)

$b_{dk} := 11\ ft + 4.375\ in = 11.36\ ft$ (total width of decking, from Longitudinal Deck Planks_Case 3.mcdx)

$A_{dk} := (t_{plank}) \cdot (b_{dk}) = 340.9\ in^2$ (total cross-sectional area)

Floorbeam Member (4 x 12 S4S, trimmed to 6" minimum height) --

$b_{fb} := 3.5\ in$ (width)

$h_{fb} := 7\ in$ (height)

$A_{fb} := (b_{fb}) \cdot (h_{fb}) = 24.50\ in^2$ (cross-sectional area)

$S_{fb} := \dfrac{(b_{fb}) \cdot (h_{fb}^{\ 2})}{6} = 28.58\ in^3$ (section modulus)

$I_{fb} := \dfrac{(b_{fb}) \cdot (h_{fb}^{\ 3})}{12} = 100.0\ in^4$ (moment of inertia)

Vehicle Live Load --

$cl_{wl} := 6\ in$ (clear distance from curb to wheel line, assumed)

$s_{wl} := 48\ in$ (wheel line spacing, Loads.mcdx)

Material (Wood) --

Timber Reference Design Values (Douglas Fir-Larch, No. 1 or better, LRFD Tbl. 8.4.1.1.4-1) --

The floorbeam is classified as "dimension lumber".

$F_{bo} := 1.200\ ksi$ (bending)

$F_{vo} := 0.180\ ksi$ (shear parallel to grain)

$F_{cpo} := 0.625\ ksi$ (compression perpendicular to grain)

$E_o := 1800\ ksi$ (modulus of elasticity)

$\gamma_w := 0.050\ kcf$ (unit weight, LRFD Tbl. 3.5.1-1)

Load Modifiers (LRFD 1.3.1.2) --

$\eta_D := 1.00$ (ductility factor, conventional design, LRFD 1.3.3)

$\eta_R := 1.05$ (redundancy factor, floorbeams are nonredundant, LRFD 1.3.4)

$\eta_I := 1.00$ (operational importance factor, typical bridge, LRFD 1.3.5)

$\eta := (\eta_D) \cdot (\eta_R) \cdot (\eta_I) = 1.05$ (load modifier, max. γi appropriate, LRFD Eq. 1.3.2.1-2)

Load Factors (LRFD Tbls. 3.4.1-1 & 2) --

$\gamma_{DC} := 1.25$ (component dead load)

$\gamma_{LL} := 1.35$ (live load, ATV and trailer per Strength II limit state used for Case 3 combination)

$\gamma_{IC} := 1.00$ (ice and snow accumulation per Extreme Event II limit state used for Case 3 combination)

Resistance Factors (LRFD 8.5.2.2) --

$\phi_f := 0.85$ (flexure)

$\phi_v := 0.75$ (shear)

$\phi_{cp} := 0.90$ (compression perpendicular to grain)

3. Loads on Floorbeam:

Floorbeam Self-Weight (DC) --

$w_{fb} := (A_{fb}) \cdot (\gamma_w) = 8.51\ plf$ (unit weight of floorbeam)

Loads from Spans --

Span reactions will conservatively be based on the middle support of a 2-span continuous configuration using a finite element analysis in GT Strudl (2 Span Loads.gti).

Reaction Influence Values for Uniformly Distributed Loads in Spans --

$R_{infl} := 0.00750 \cdot \frac{kip}{plf}$ (Load 'LD_UNIF', jt. 135)

Component Dead Load (DC, deck planks) --

Note: The rails and curb weights are not included. This is conservative for mid-span moment calculations and do not contribute to shear between the supports.

$w_{dk.span} := (A_{dk}) \cdot (\gamma_w) = 118.4\ plf$ (distributed load along span)

$w_{dk} := \frac{(w_{dk.span}) \cdot (R_{infl})}{b_{dk}} = 78.13\ plf$ (distributed load along floorbeam)

Pedestrian Live Load (PL, LRFD Ped 3.1) --

$p_{ped} := 0.090\ ksf$ (pressure)

$w_{ped.span} := (p_{ped}) \cdot (b_{wlk}) = 0.940\ klf$ (distributed load along span)

$w_{ped} := \frac{(w_{ped.span}) \cdot (R_{infl})}{b_{dk}} = 620.55\ plf$ (distributed load along floorbeam)

Vehicle Live Load (VL) --

$R_{wl} := 0.933\ kip = 933.0\ lbf$ (wheel line reaction, from GT Strudl, 2 Span Loads.gti, jt. 135)

Snow Load (IC) --

$p_f := 54\ psf$ (pressure, Loads.mcdx)

$w_{snow.span} := (p_f) \cdot (b_{dk}) = 0.614\ klf$ (distributed load along span)

$w_{snow} := \dfrac{(w_{snow.span}) \cdot (R_{infl})}{b_{dk}} = 405.00\ plf$ (distributed load along floorbeam)

4. Unfactored Results from GT Strudl Model (Floorbeam_Cribbing.gti):

Column Reactions (jt. 4 and 14) --

$R_{DC} := \begin{bmatrix} 230 \\ 534 \end{bmatrix} lbf$ (component dead load)

$R_{PL} := \begin{bmatrix} 1621 \\ 3835 \end{bmatrix} lbf$ (pedestrian live load)

$R_{IC} := \begin{bmatrix} 1058 \\ 2503 \end{bmatrix} lbf$ (snow load)

$R_{VL} := \begin{bmatrix} 870 \\ 1478 \end{bmatrix} lbf$ (vehicle live load)

$R_{max} := R_{DC} + R_{IC} + \max(R_{PL}, R_{VL}) = \begin{bmatrix} 5.12 \\ 6.87 \end{bmatrix} kip$ (total controlling unfactored reaction)

Moments @ Centerline "Column" (jt. 14, the controlling moments are negative over interior "column") --

$M_{DC} := 3169\ in \cdot lbf$ (component dead load)

$M_{PL} := 22840\ in \cdot lbf$ (pedestrian live load)

$M_{IC} := 14907\ in \cdot lbf$ (snow load)

$M_{VL} := 10748\ in \cdot lbf$ (vehicle live load)

Shears @ Columns (jt. 14) --

$V_{DC} := 269\ lbf$ (component dead load)

$V_{PL} := 1932\ lbf$ (pedestrian live load)

$V_{IC} := 1261\ lbf$ (snow load)

$V_{VL} := 1082\ lbf$ (vehicle live load)

5. Factored Moments and Shears:

Use Case 3 Combination: 1.25*DC + 1.35*LL + 1.00*IC.

Moments @ Mid-Span Between Columns --

$M_{uDC} := (M_{DC}) \cdot (\gamma_{DC}) = 3.96\ in \cdot kip$ (component dead load)

$M_{uLL} := \max(M_{PL}, M_{VL}) \cdot (\gamma_{LL}) = 30.83\ in \cdot kip$ (controlling live load)

$M_{uIC} := (M_{IC}) \cdot (\gamma_{IC}) = 14.91\ in \cdot kip$ (snow load)

$M_u := M_{uDC} + M_{uLL} + M_{uIC} = 49.70\ in \cdot kip$ (total moment)

Shears @ Columns --

$V_{uDC} := (V_{DC}) \cdot (\gamma_{DC}) = 0.34\ kip$ (component dead load)

$V_{uLL} := \max(V_{PL}, V_{VL}) \cdot (\gamma_{LL}) = 2.61\ kip$ (controlling live load)

$V_{uIC} := (V_{IC}) \cdot (\gamma_{IC}) = 1.26\ kip$ (snow load)

$V_u := V_{uDC} + V_{uLL} + V_{uIC} = 4.21\ kip$ (total shear)

Compression @ Column --

$P_{uDC} := (R_{DC}) \cdot (\gamma_{DC}) = \begin{bmatrix} 0.29 \\ 0.67 \end{bmatrix} kip$ (component dead load)

$P_{uLL} := \max(R_{PL}, R_{VL}) \cdot (\gamma_{LL}) = 5.18\ kip$ (controlling live load)

$P_{uIC} := (R_{IC}) \cdot (\gamma_{IC}) = \begin{bmatrix} 1.06 \\ 2.50 \end{bmatrix} kip$ (snow load)

$P_u := P_{uDC} + P_{uLL} + P_{uIC} = \begin{bmatrix} 6.52 \\ 8.35 \end{bmatrix}\ kip$ (total compression)

6. Flexural Resistance (LRFD 8.6):

Adjustment Factors for Reference Design Values and Adjusted Design Value (LRFD 8.4.4) --

$C_{KF.b} := \frac{2.5}{\phi_f} = 2.94$ (format conversion factor, LRFD 8.4.4.2)

$C_{F.b} := 1.1$ (size factor, 4 x 12, LRFD Tbl. 8.4.4.4-1)

$$C_{M.b} := \left\| \begin{array}{l} \text{if } (F_{bo}) \cdot (C_{F.b}) \le 1.15\ ksi \\ \quad \| 1.00 \\ \text{else} \\ \quad \| 0.85 \end{array} \right\| = 0.85$$

(wet service factor, t <= 4", LRFD Tbl. 8.4.4.3-1)

$C_{fu} := 1.00$ (flat-use factor, not applicable, LRFD Tbl. 8.4.4.6-1)

$C_{i.b} := 0.80$ (incising factor, LRFD Tbl. 8.4.4.7-1)

$C_d := 1.0$ (not applicable, LRFD 8.4.4.8)

$C_\lambda := 0.8$ (time effect factor, Strength-I, LRFD Tbl. 8.4.4.9-1)

$F_b := (F_{bo}) \cdot (C_{KF.b}) \cdot (C_{M.b}) \cdot (C_{F.b}) \cdot (C_{fu}) \cdot (C_{i.b}) \cdot (C_d) \cdot (C_\lambda)$ (adjusted design value, LRFD Eq. 8.4.4.1-1)

$F_b = 2.11\ ksi$

Components in Flexure, Rectangular Section (LRFD 8.6.2) --

$$L_e := \begin{array}{l} L_u \leftarrow S_{col} \\ \lambda_{fb} \leftarrow \dfrac{L_u}{h_{fb}} \\ \text{if } \lambda_{fb} < 7 \\ \quad 2.06 \cdot L_u \\ \text{else if } 7 \le \lambda_{fb} \le 14.3 \\ \quad 1.63 \cdot L_u + 3 \cdot h_{fb} \\ \text{else} \\ \quad 1.84 \cdot L_u \end{array} = 9.90\ ft$$

(effective unbraced length, LRFD 8.6.2)

$$R_b := min\left(\sqrt{\frac{L_e \cdot h_{fb}}{{b_{fb}}^2}}, 50\right) = 8.24$$

(LRFD Eq. 8.6.2-5)

$$K_{bE} := 0.76$$

(Euler buckling coefficient, visually graded lumber, LRFD 8.6.2)

$$C_{M.E} := 0.90$$

(wet service factor, LRFD Tbl. 8.4.4.3-1)

$$C_{i.E} := 0.95$$

(incising factor, LRFD Tbl. 8.4.4.7-1)

$$E := (E_o) \cdot (C_{M.E}) \cdot (C_{i.E}) = 1539\ ksi$$

(modulus of elasticity, LRFD Eq. 8.4.4.1-6)

$$F_{bE} := \frac{K_{bE} \cdot E}{{R_b}^2} = 17.23\ ksi$$

(elastic buckling stress, LRFD Eq. 8.6.2-4)

$$A := \frac{F_{bE}}{F_b} = 8.16$$

(parameter for beam stability, LRFD Eq. 8.6.2-3)

$$C_L := \frac{1+A}{1.9} - \sqrt{\frac{(1+A)^2}{3.61} - \frac{A}{0.95}} = 0.99$$

(beam stability factor, LRFD Eq. 8.6.2-2)

$$M_n := (F_b) \cdot (S_{fb}) \cdot (C_L) = 59.95\ in \cdot kip$$

(nominal resistance, LRFD Eq. 8.6.2-1)

$$M_r := (\phi_f) \cdot (M_n) = 50.96\ in \cdot kip$$

(factored resistance, LRFD Eq. 8.6.1-1)

$CDR_f := \frac{M_r}{M_u} = 1.03$ >= 1.0, OKAY (capacity-to-demand ratio)

7. Shear Resistance (LRFD 8.6):

Adjustment Factors for Reference Design Values and Adjusted Design Value (LRFD 8.4.4) --

$C_{KF.v} := \frac{2.5}{\phi_v} = 3.33$ (format conversion factor, LRFD 8.4.4.2)

$C_{M.v} := 0.97$ (wet service factor, t <= 4", LRFD Tbl. 8.4.4.3-1)

$C_{i.v} := 0.80$ (incising factor, LRFD Tbl. 8.4.4.7-1)

$C_{\lambda} = 0.80$ (time effect factor, Strength-I, recalled)

$F_v := (F_{vo}) \cdot (C_{KF.v}) \cdot (C_{M.v}) \cdot (C_{i.b}) \cdot (C_{\lambda}) = 0.37\ ksi$ (adjusted design value, LRFD Eq. 8.4.4.1-2)

Components Under Shear (LRFD 8.7) --

$V_n := \frac{(F_v) \cdot (b_{fb}) \cdot (h_{fb})}{1.5} = 6.08\ kip$ (nominal resistance, LRFD Eq. 8.7-2)

$V_r := (\phi_v) \cdot (V_n) = 4.56\ kip$ (factored resistance, LRFD Eq. 8.7-1)

$CDR_v := \frac{V_r}{V_u} = 1.08$ >= 1.0, OKAY (capacity-to-demand ratio)

8. Compression Perpendicular to Grain Resistance (LRFD 8.8):

Adjustment Factors for Reference Design Values and Adjusted Design Value (LRFD 8.4.4) --

$C_{KF.cp} := \frac{2.1}{\phi_{cp}} = 2.33$ (format conversion factor, LRFD 8.4.4.2)

$C_{M.cp} := 0.67$ (wet service factor, LRFD Tbl. 8.4.4.3-1)

$C_{i.cp} := 1.00$ (incising factor, LRFD Tbl. 8.4.4.7-1)

$C_{\lambda} = 0.80$ (time effect factor, Strength-I, recalled)

$F_{cp} := (F_{cpo}) \cdot (C_{KF.cp}) \cdot (C_{M.cp}) \cdot (C_{i.cp}) \cdot (C_{\lambda}) = 0.78\ ksi$ (adjusted design value, LRFD Eq. 8.4.4.1-5)

Compression Perpendicular to Grain (LRFD 8.8.3) --

$L_b := 11.25\ in = 11.25\ in$ (length of bearing along grain)

$A_b := (b_{fb})\ (L_b) = 39.38\ in^2$ (bearing area)

$C_b := 1.00$ (adjustment factor for bearing, LRFD Tbl. 8.8.3-1)

$P_n := (F_{cp}) \cdot (A_b) \cdot (C_b) = 30.78\ kip$ (nominal resistance, LRFD Eq. 8.8.3-1)

$P_r := (\phi_{cp})\ (P_n) = 27.70\ kip$ (factored resistance, LRFD Eq. 8.8.1-1)

$CDR_{cp} := \frac{P_r}{P_u} = \begin{bmatrix} 4.25 \\ 3.32 \end{bmatrix}$ >= 1.0, OKAY (capacity-to-demand ratio)

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STRUDL 'WESTERN ALASKA BOARDWALK ASP'

$ FLOORBEAM MODEL - ON CRIBBING

$ ENGINEER:  ERIC E. BONN, P.E.
$ COMPANY:  DOWL, LLC


$ ***** JOINT DEFINITIONS *****


UNITS FEET

JOINT COORDINATES

 1 -6.00 0.00 $ LEFT END OF FLOORBEAM
 2 -5.68 0.00 $ LEFT EDGE OF DECK
 3 -5.22 0.00 $ LEFT EDGE OF WALKWAY
 4 -5.00 0.00 $ LEFT COLUMN
 5 -4.72 0.00 $ LEFT EDGE OF WHEEL LINE RANGE
 6 -4.00 0.00
 7 -3.50 0.00
 8 -3.00 0.00
 9 -2.50 0.00
10 -2.00 0.00
11 -1.50 0.00
12 -1.00 0.00
13 -0.50 0.00
14 -0.00 0.00 $ CENTERLINE COLUMN
15  0.50 0.00
16  1.00 0.00
17  1.50 0.00
18  2.00 0.00
19  2.50 0.00
20  3.00 0.00
21  3.50 0.00
22  4.00 0.00
23  4.72 0.00 $ RIGHT EDGE OF WHEEL LINE RANGE
24  5.00 0.00 $ RIGHT COLUMN
25  5.22 0.00 $ RIGHT EDGE OF WALKWAY
26  5.68 0.00 $ RIGHT EDGE OF DECK
27  6.00 0.00 $ RIGHT END OF FLOORBEAM

STATUS SUPPORT 4 14 24

JOINT RELEASES
 4 MOMENT Z
14 24 FORCE X MOMENT Z


$ ***** MEMBER DEFINITIONS *****


TYPE PLANE FRAME

GENERATE 26 MEMBERS ID 'FB1' INC 1 FROM 1 INC 1 TO 2 INC 1

DEFINE GROUP 'gFLRBM' 'FLOORBEAM MEMBERS' MEMBERS 'FB1' TO 'FB26'

UNITS INCHES

MEMBER PROPERTIES PRISMATIC
 GROUP 'gFLRBM' AX 21.00 IZ 63.0

MATERIAL STEEL

UNITS KIP INCHES

CONSTANTS
```

```
 E 1539 GROUP 'gFLRBM'


$ ***** LOADINGS *****


UNITS LBS FEET

LOADING 'DC_FB' 'COMPONENT DEAD LOAD_FLOORBEAM'
 MEMBER LOADS
  GROUP 'gFLRBM' FORCE Y GLOBAL UNIFORM FRACTIONAL W -8.51 LA 0.0 LB 1.0

LOADING 'DC_DECK' 'COMPONENT DEAD LOAD_DECK'
 MEMBER LOADS
  'FB2' TO 'FB23' FORCE Y GLOBAL UNIFORM FRACTIONAL W -78.13 LA 0.0 LB 1.0

FORM LOADING 'DC' 'COMPONENT DEAD LOAD_TOTAL' FROM -
 'DC_FB' 1.0 'DC_DECK' 1.0

LOADING 'PL' 'PEDESTRIAN LIVE LOAD'
 MEMBER LOADS
  'FB2' TO 'FB23' FORCE Y GLOBAL UNIFORM FRACTIONAL W -620.55 LA 0.0 LB 1.0

LOADING 'IC' 'SNOW LOAD'
 MEMBER LOADS
  'FB2' TO 'FB23' FORCE Y GLOBAL UNIFORM FRACTIONAL W -405.00 LA 0.0 LB 1.0

$ VEHICLE LIVE LOAD - ATV W/TRAILER WHEEL LINE
MOVING LOAD GENERATOR
 SUPERSTRUCTURE N 2 MEMBER 'FB5' TO 'FB22'
 TRUCK LOAD DIRECTION FORWARD -
 VEHICLE GENERAL TRUCK 933.0 4.00 933.0
 GENERATE LOAD Y SCALE -1.00 INITIAL 'atv1'
END LOAD GENERATOR

DEFINE GROUP 'ATV' 'VEHICLE LIVE LOAD' LOADS 'atv17' TO 'atv37'

$ ***** RUN ANALYSIS AND OUTPUT RESULTS *****


STIFFNESS ANALYSIS

UNITS INCHES LBS

OUTPUT DECIMAL 1
LOAD LIST 'DC' 'PL' 'IC'
LIST REACTIONS JOINTS 4 14
LIST FORCES MEMBERS 'FB4' 'FB14'

LOAD LIST GROUP 'ATV'
LIST FORCE ENVELOPE MEMBERS EXISTING 'FB1' TO 'FB13' SECTION FR DS 0.0 1.0
LIST MAXIMUM REACTION ENVELOPE FOR LOADS ACTIVE
```

Substructure

WESTERN ALASKA BOARD ROAD ASP
COLUMN CALCULATIONS
Case 3: 1.25*DC + 1.35*LL + 1.00*IC

Narrative:

This file checks the structural capacity of the timber columns and steel pipe.

The Case 3 load combination is not in the LRFD. This combination was developed based on the Strength II limit state combination with the addition of snow accumulation, which is not normally included. The probability of having a design snow load in combination with a heavily overloaded design vehicle was deemed low, justifying a lower live load factor than the 1.75 used for Strength I limit state combinations.

Table of Contents:

1. References:

AASHTO LRFD Bridge Design Specifications, 9th Ed. (LRFD)
AASHTO LRFD Guide Specifications for the Design of Pedestrian Bridges, 2nd Ed. w/2015 Interims (AASHTO Ped)
AISC Steel Construction Manual, 15th Ed. (AISC)
Alaska Highway Preconstruction Manual, January 2025 Ed. (AHPM)
Alaska Bridges and Structures Manual, June 2025 Ed. (ABSM)
National Design Specifications for Wood Construction, 2018 Ed. (NDS)

2. Constants/Input Information:

General --

$kcf := 1000\ pcf$ (unit definition)

Timber Members (S4S) --

Column (6 x 6) --

$d_{col} := 5.5\ in$ (side dimension)

$A_{g.t} := d_{col}^2 = 30.25\ in^2$ (cross-sectional area)

$L_{col} := 5\ ft + 0\ in = 5.00\ ft$ (effective length, assumed)

$K_{col} := 1.00$ (effective length factor, treat as pinned-pinned, LRFD Tbl. C4.6.2.5-1, Case "c")

$L_{e.col} := (K_{col}) \cdot (L_{col}) = 5.00\ ft$ (effective length, LRFD Eq. 8.8.2-4)

Helical Anchor Pipe Extension (Std. 2.5" pipe assumed, AISC) --

$A_{g.s} := 1.70\ in^2$ (cross-sectional area)

$r_s := 0.947\ in$ (radius of gyration)

$Kl := 9\ ft + 0\ in = 9.00\ ft$ (effective length, assumed exposed + depth to helix)

Materials --

Timber --

Coefficients for Compressive Resistance (LRFD Eq. 8.8.2-4) --

$K_{cE} := 0.52$ (Euler buckling coefficient, visually graded lumber)

$c_t := 0.8$ (sawn lumber)

Timber Reference Design Values (Douglas Fir-Larch, No. 1, LRFD Tbl. 8.4.1.1.4-1) --

Columns are "post and timber"

$F_{co} := 1.000\ ksi$ (compression parallel to grain)

$E_o := 1600\ ksi$ (modulus of elasticity)

Steel --

$E_s := 29000\ ksi$ (modulus of elasticity, LRFD 6.4.1)

$F_y := 35\ ksi$ (yield stress, ASTM A53, Gr. B)

Loading (from Floorbeam_Case 3.mcdx) --

$P_u := 7.11\ kip$ (factored)

Resistance Factors --

Timber (LRFD 8.5.2.2) --

$\phi_{c.t} := 0.90$ (compression parallel to grain)

Steel (LRFD 6.5.4.2)

$\phi_{c.s} := 0.95$ (compression steel only)

<u>3. Compressive Resistance - Timber:</u>

Adjusted Design Values (LRFD 8.4.4.1) --

Compression Parallel to Grain --

$C_{KF} := \frac{2.5}{\phi_{c.t}} = 2.78$ (format conversion factor, LRFD 8.4.4.2)

$C_{M.c} := 0.91$ (wet service factor, > 4", LRFD Tbl. 8.4.4.3-1)

$C_F := 1.1$ (size effect factor, LRFD Tbl. 8.4.4.4-1, 6" width)

$C_i := 1.00$ (incising factor, not dimensional lumber, LRFD 8.4.4.7)

$C_\lambda := 0.80$ (time effect factor, STR-I used, LRFD Tbl. 8.4.4.9-1)

$F_c := (F_{co}) \cdot (C_{KF}) \cdot (C_{M.c}) \cdot (C_F) \cdot (C_i) \cdot (C_\lambda) = 2.22\ ksi$ (adjusted design value, LRFD Eq. 8.4.4.1-4)

Modulus of Elasticity --

$C_{M.E} := 1.00$ (wet service factor, > 4", LRFD Tbl. 8.4.4.3-1)

$C_i = 1.00$ (incising factor, recalled)

$E_t := (E_o) \cdot (C_{M.E}) \cdot (C_i) = 1600\ ksi$ (adjusted design value, LRFD Eq. 8.4.4.1-6)

Compression Parallel to Grain Resistance (LRFD 8.8.2) --

$$F_{cE} := \frac{\left(K_{cE}\right) \cdot \left(E_t\right) \cdot \left(d_{col}^{\ 2}\right)}{L_{e.col}^{\ 2}} = 6.99\ \textit{ksi}$$

(Euler buckling stress, LRFD Eq. 8.8.2-4)

$$B := min\left(\frac{F_{cE}}{F_c}, 1.0\right) = 1.00$$

(parameter for compression, LRFD Eq. 8.8.2-3)

$$C_p := min\left(\frac{1+B}{(2) \cdot \left(c_t\right)} - \sqrt{\left(\frac{1+B}{(2) \cdot \left(c_t\right)}\right)^2 - \frac{B}{c_t}}, 1.0\right) = 0.69$$

(column stability factor, LRFD Eq. 8.8.2-2)

$$P_{n.t} := \left(F_c\right) \cdot \left(A_{g.t}\right) \cdot \left(C_p\right) = 46.5\ \textit{kip}$$

(nominal resistance, LRFD Eq. 8.8.2-1)

$$P_{r.t} := \left(P_{n.t}\right) \cdot \left(\phi_{c.t}\right) = 41.8\ \textit{kip}$$

(factored resistance, LRFD Eq. 8.8.1-1)

$\frac{P_{r.t}}{P_u} = 5.89$ > 1.00, OKAY

3. Compressive Resistance - Steel:

Limiting Slenderness Ratio for Compression Members (LRFD 6.9.3) --

$\frac{Kl}{r_s} = 114$ <= 120, OKAY

Nominal Compressive Resistance (LRFD 6.9.4.1) --

Only elastic flexural buckling applies to circular pipe members per LRFD Tbl. 6.9.4.1.1-1).

$$P_e := \frac{\left(\pi^2\right) \cdot \left(E_s\right)}{\left(\frac{Kl}{r_s}\right)^2} \cdot \left(A_{g.s}\right) = 37.4\ \textit{kip}$$

(critical buckling resistance, LRFD Eq. 6.9.4.1.2-1)

$$P_o := \left(F_y\right) \cdot \left(A_{g.s}\right) = 59.5\ \textit{kip}$$

(nominal yield resistance, LRFD 6.9.4.1.1)

$$P_{n.s} := \begin{cases} \text{if } \dfrac{P_o}{P_e} \le 2.25 \\ \quad \left(0.658^{\left(\frac{P_o}{P_e}\right)}\right) \cdot \left(P_o\right) \\ \text{else} \\ \quad \left(0.877\right) \cdot \left(P_e\right) \end{cases} = 30.6\ \mathbf{kip}$$

(nominal resistance, LRFD Eqs. 6.9.4.1.1-1 & 2)

$$P_{r.s} := \left(\phi_{c.s}\right) \cdot \left(P_{n.s}\right) = 29.0\ \mathbf{kip}$$

(factored resistance, LRFD Eq. 6.9.2.1-1)

$$\frac{P_{r.s}}{P_u} = 4.09$$ > 1.00, OKAY

WESTERN ALASKA BOARD ROAD ASP
COLUMN TOE-NAIL CONNNECTION CALCULATIONS

Narrative:

This file checks the structural capacity of the toe-nail connection at the bottom of the timber columns.

The connection will be designed for the design wind load.

Table of Contents:

1. References:

AASHTO LRFD Bridge Design Specifications, 9th Ed. (LRFD)
National Design Specifications for Wood Construction, 2018 Ed. (NDS)

2. Constants/Input Information:

General --

$kcf := 1000\ pcf$ (unit definition)

Timber Members (S4S) --

Column (6 x 6) --

$N_{col} := 2$ (number per bent)

$d_{col} := 5.5\ in$ (side dimension)

$A_{g.t} := {d_{col}}^2 = 30.25\ in^2$ (cross-sectional area)

Wood Screws (No. 10) --

$N_{scr} := 4$ (number per connection)

$D := 0.190\ in$ (diameter, NDS Tbl. 12L)

Materials --

Timber --

Douglas Fir-Larch

$G := 0.50$ (specific gravity, NDS Tbl. 12.3.3A)

Wood Screws --

Meet requirements of ANSI/ASME Standard B18.6.1.

Shear Loading on Connection (from Sway Bracing_IBC.mcdx) --

$V_{WSpost} := 849\ lbf$ (unfactored)

$\gamma_{WS} := 1.00$ (load factor, wind on structure, LRFD Tbl. 3.4.1, STR-III and V)

$V_u := \left(\frac{V_{WSpost}}{N_{col}}\right) \cdot \left(\gamma_{WS}\right) = 425\ lbf$ (factored per column)

Resistance Factor --

Timber (NDS Tbl. 11.3.1) --

$\phi_{conn} := 0.65$ (connection)

3. Connection Resistance:

Reference and Adjusted Design Values (NDS Tbl. 11.3.1) --

Wood Screws in Shear --

$Z := 117\ lbf$ (reference design value, side member thickness >1", NDS Tbl. 12L)

$K_F := 3.32$ (format conversion factor)

$C_M := 0.7$ (wet service factor, > 19%, NDS Tbl. 11.3.3)

$C_t := 1.0$ (temperature factor, NDS Tbl. 11.3.4, < 100°F, Wet)

$C_g := 1.0$ (group action factor, screws not in rows, NDS 11.3.6)

$C_{\Delta} := 1.0$ (geometry factor, D < 1/4", NDS 12.5.1)

$C_{eg} := 1.00$ (end grain factor, only applies for withdrawal from end grain, NDS 12.2.2)

$C_{di} := 1.0$ (diaphragm factor, not diaphragm construction, NDS 12.5.3)

$C_{tn} := 0.83$ (toe-nail factor, NDS 12.5.4)

$\lambda := 0.80$ (time effect factor, NDS Tbl. N3)

$$Z' := (Z) \cdot (K_F) \cdot (C_M) \cdot (C_t) \cdot (C_g) \cdot (C_{\Delta}) \cdot (C_{eg}) \cdot (C_{di}) \cdot (C_{tn}) \cdot (\lambda) \cdot (\phi_{conn}) = 117 \ lbf$$

(adjusted design value, LRFD Eq. 8.4.4.1-4)

Shear Check --

$V_r := (N_{scr}) \cdot (Z') = 469 \ lbf$ (factored shear resistance)

$$CDR := \frac{V_r}{V_u} = 1.11$$

> 1.00 OKAY

WESTERN ALASKA BOARDWALK ASP
SWAY BRACING (SUBSTRUCTURE) CONNECTION CALCULATIONS - IBC LOADING

Narrative:

This file is for the load demands and resistances for sway bracing and other substructure connections.
Pedestrian live loadings on the rails are per the International Building Code.

Table of Contents:

1. References:

AASHTO LRFD Bridge Design Specifications, 9th Ed. (LRFD)
AASHTO LRFD Guide Specifications for the Design of Pedestrian Bridges (AASHTO Ped)
International Building Code, XX Ed. (IBC)
National Design Specification, 2018 Ed. (NDS)
Alaska Bridges and Structures Manual (ABSM)

2. Constants/Input Information:

General --

$kcf := 1000\ pcf$ (unit definition)

Geometry --

Deck (3 x 12, S4S) --

$t_{dk} := 2.5\ in$ (thickness)

$b_{wlk} := 10\ ft + 0\ in = 10.00\ ft$

Floorbeam (4 x 12, S4S) --

$s_{fb} := 6\ ft + 6\ in = 6.50\ ft$ (spacing)

$h_{fb} := 11.25\ in$ (height)

$b_{fb} := 3.5\ in$ (width)

$L_{fb} := 12\ ft + 0\ in = 12.00\ ft$ (length)

Posts (4 x 6, S4S) --

$s_{post} := s_{fb} = 6.50\ ft$ (spacing)

$h_{post.dk} := 42\ in = 3.50\ ft$ (height of post + rail cap, above top of deck)

$b_{post} := 3.5\ in$ (width)

$d_{post} := 5.5\ in$ (depth)

$A_{post} := \left(b_{post}\right) \cdot \left(d_{post}\right) = 19.25\ in^2$ (cross-sectional area)

$S_{x.post} := \dfrac{\left(b_{post}\right) \cdot \left(d_{post}^{\ 2}\right)}{6} = 17.65\ in^3$ (section modulus, about axis parallel to boardwalk)

$S_{y.post} := \dfrac{\left(d_{post}\right) \cdot \left(b_{post}^{\ 2}\right)}{6} = 11.23\ in^3$ (section modulus, about axis perpendicular to boardwalk)

$I_{post} := \dfrac{\left(b_{post}\right) \cdot \left(d_{post}^{\ 3}\right)}{12} = 48.53\ in^4$ (moment of inertia)

$d_{h.post} := 0.5\ in$ (bolt hole diameter in connection)

$S_{x.post.mod} := S_{x.post} - \dfrac{\left(d_{h.post}\right) \cdot \left(d_{post}^{\ 2}\right)}{6} = 15.13\ in^3$ (modified section modulus to account for bolt hole)

Rail Cap (2 x 8, S4S) --

$t_{rc} := 1.5\ in$ (thickness)

$b_{rc} := 7.25\ in$ (width)

$A_{rc} := \left(b_{rc}\right) \cdot \left(t_{rc}\right) = 10.88\ in^2$ (cross-sectional area)

$S_{rc} := \frac{(b_{rc}) \cdot (t_{rc}^{2})}{6} = 2.72\ in^3$ (section modulus)

$I_{rc} := \frac{(b_{rc}) \cdot (t_{rc}^{3})}{12} = 2.04\ in^4$ (moment of inertia)

$y_{rc} := h_{post.dk} - \frac{t_{rc}}{2} = 3.44\ ft$ (centroid of rail cap above top of deck)

Rails (2 x 6, S4S) --

$N_r := 4$ (number)

$h_r := 5.5\ in$ (height)

$b_r := 1.5\ in$ (width)

$gap_r := 3.75\ in$ < 4" IBC requirement < 6" LRFD requirement (open gap between rails)

$y_r := y_{rc} - \frac{t_{rc} + h_r}{2} - (h_r + gap_r) \cdot \begin{bmatrix} 0 \\ 1 \\ 2 \\ 3 \end{bmatrix} = \begin{bmatrix} 3.15 \\ 2.38 \\ 1.60 \\ 0.83 \end{bmatrix} ft$ (centroid of rails above top of deck)

$A_r := (b_r) \cdot (h_r) = 8.25\ in^2$ (cross-sectional area)

$S_{x.r} := \frac{(b_r) \cdot (h_r^{2})}{6} = 7.56\ in^3$ (section modulus, about horizontal axis)

$S_{y.r} := \frac{(h_r) \cdot (b_r^{2})}{6} = 2.06\ in^3$ (section modulus, about vertical axis)

$I_{x.r} := \frac{(b_r) \cdot (h_r^{3})}{12} = 20.80\ in^4$ (moment of inertia, about horizontal axis)

$I_{y.r} := \frac{(h_r) \cdot (b_r^{3})}{12} = 1.55\ in^4$ (moment of inertia, about vertical axis)

Curb Spacers and Curbs --

Curb Spacers (3 x 6) --

$h_{ks} := 2.5\ in$ (height of curb spacer)

$y_{ks} := \frac{h_{ks}}{2} = 0.10\ ft$ (centroid of curb spacer above top of deck)

Curbs (4 x 6) --

$h_k := 3.5\ in$ (height of curb)

$b_k := 5.5\ in$ (width of curb)

$S_{x.k} := \frac{(b_k) \cdot (h_k^2)}{6} = 11.23\ in^3$ (section modulus, about horizontal axis)

$S_{y.k} := \frac{(h_k) \cdot (b_k^2)}{6} = 17.65\ in^3$ (section modulus, about vertical axis)

$I_{x.k} := \frac{(b_k) \cdot (h_k^3)}{12} = 19.65\ in^4$ (moment of inertia, about horizontal axis)

$I_{y.k} := \frac{(h_k) \cdot (b_k^3)}{12} = 48.53\ in^4$ (moment of inertia, about vertical axis)

$y_k := h_{ks} + \frac{h_k}{2} = 0.35\ ft$ (centroid of curb above top of deck)

Sway Brace (L 2 x 2 x 3/16) --

$A_{gL} := 0.715\ in^2$ (cross-sectional area, gross)

$t_L := \frac{3}{16} \cdot in$ (thickness)

$x_{bar} := 0.569\ in$ (back of angle to centroid)

$d_{hL} := \frac{5}{8} \cdot in$ (diameter of hole for 1/2" dia. bolt)

$A_{nL} := A_{gL} - (t_L) \cdot (d_{hL}) = 0.598\ in^2$ (cross-sectional area, net)

$\theta_{sway} := \operatorname{asin}\left(\frac{16\ in}{2\ ft + 10\ in}\right) = 28.07\ deg$ (angle of brace to horizontal)

$L_{Lconn} := 1\ in$ (length of bolted connection)

$end_L := 1\ in$ (distance from end of angle to center of bolt hole)

$t_{pl.L} := \frac{3}{16} \cdot in$ (thickness of connection plate)

Beam Saddle (connecting columns to floorbeam) --

$N_{sddl} := 2$ (number of saddles per floorbeam)

$N_{b.sddl} := 2$ (number of bolts per saddle)

$t_{sddl} := \frac{1}{8} \cdot in$ (thickness, assumed)

$d_{b.sddl} := 0.50\ in$ (diameter of bolts)

$D_{r.sddl} := 0.406\ in$ (root diameter of bolt, NDS Appendix Tbl. L1)

$A_{b.sddl} := 0.20\ in^2$ (area of bolts)

Material (Wood) --

Wood --

Timber Reference Design Values (Douglas Fir-Larch, No. 1 or better, LRFD Tbl. 8.4.1.1.4-1) --

The posts, rail caps, and rails are classified as "dimension lumber"

$F_{bo} := 1.200\ ksi$ (bending)

$F_{vo} := 0.180\ ksi$ (shear parallel to grain)

$F_{cpo} := 0.625\ ksi$ (compression perpendicular to grain)

$E_o := 1800\ ksi$ (modulus of elasticity)

$\gamma_w := 0.050\ kcf$ (unit weight, LRFD Tbl. 3.5.1-1)

$G_w := 0.50$ (assigned specific gravity, NDS Tbl 12.3.3A)

Adjusted Design Value for Modulus of Elasticity --

$C_{M.E} := 0.90$ (wet service factor, LRFD Tbl. 8.4.4.3-1)

$C_{i.E} := 0.95$ (incising factor, LRFD Tbl. 8.4.4.7-1)

$E := \left(E_o\right) \cdot \left(C_{M.E}\right) \cdot \left(C_{i.E}\right) = 1539\ ksi$ (modulus of elasticity, LRFD Eq. 8.4.4.1-6)

Steel (sway brace angle, ASTM A36) --

Sway Brace Angle (ASTM A36) --

$F_{yL} := 36\ ksi$ (yield stress)

$F_{uL} := 58\ ksi$ (tensile strength)

Beam Saddle (ASTM A36, assumed) --

$F_{y.sddl} := 36\ ksi$ (yield stress)

$F_{u.sddl} := 58\ ksi$ (tensile strength)

Bolts (ASTM A307 Gr. A) --

$F_{yb} := 36\ ksi$ (yield stress, not required but generally taken as this per Gr. C)

$F_{ub} := 60\ ksi$ (tensile strength, LRFD 6.4.3.2)

Load Modifiers (LRFD 1.3.1.2) --

$\eta_D := 1.00$ (ductility factor, conventional design, LRFD 1.3.3)

$\eta_R := 1.05$ (redundancy factor, rail components are nonredundant, LRFD 1.3.4)

$\eta_I := 1.00$ (operational importance factor, typical bridge, LRFD 1.3.5)

$\eta := (\eta_D) \cdot (\eta_R) \cdot (\eta_I) = 1.05$ (load modifier, max. γi appropriate, LRFD Eq. 1.3.2.1-1)

Load Factors (LRFD Tbls. 3.4.1-1 & 2) --

$\gamma_{DC} := 1.25$ (component dead load)

$\gamma_{LL1} := 1.75$ (live load, forces on rail, in Strength I limit state)

$\gamma_{LL5} := 1.35$ (live load, forces on rail, in Strength V limit state, ABSM 12.1.3.5)

$\gamma_{WS3} := 1.00$ (wind load on structure, in Strength III limit state)

$\gamma_{WS5} := 0.40$ (wind load on structure, in Strength V limit state, ABSM 12.1.3.5)

Resistance Factors --

Wood (LRFD 8.5.2.2) --

$\phi_f := 0.85$ (flexure)

$\phi_v := 0.75$ (shear)

$\phi_{cp} := 0.90$ (compression perpendicular to grain)

$\phi_{conn} := 0.65$ (connections)

Steel (LRFD 6.5.4.2) --

$\phi_u := 0.80$ (tension, facture in net section)

$\phi_y := 0.95$ (tension, yield in gross section)

$\phi_s := 0.75$ (bolts in shear, A307)

$\phi_{bb} := 0.80$ (bolts bearing on material)

3. Loads (from Loads.mcdx):

Live Loads on Rail (LLr) --

Use maximum effects due to distributed or concentrated loads.

$w_{LLr} := 0.050\ klf$ (distributed load, transversely or vertically)

$P_{LLr} := 0.20\ kip$ (concentrated load)

Wind Load on Structure (WS) --

$P_Z := 65.2\ psf$ (pressure)

$w_{WSpost} := (b_{post}) \cdot (P_Z) = 19.0\ plf$ (distributed load, on post)

$w_{WSrc} := (t_{rc}) \cdot (P_Z) = 8.2\ plf$ (distributed load, on rail cap)

$w_{WSr} := (h_r) \cdot (P_Z) = 29.9\ plf$ (distributed load, on rail)

$w_{WSk} := (h_k) \cdot (P_Z) = 19.0\ plf$ (distributed load, on curb)

7. Substructure Connection Analyses:

Beam Saddles Analyses --

Shear on Bolts Load and Resistance --

Lateral Load (Shear) Due to Wind Load on Structure --

Unfactored (from Rails_IBC.mcdx) --

$V_{uDCpost} := 0\ lbf$ (lateral dead load)

$P_{WSpost.rc} := 53\ lbf$ (due to rail cap)

$P_{WSpost.r} := 194\ lbf$ (per rail)

$$P_{WSpost.post} := \begin{bmatrix} 6 \\ 6 \\ 6 \\ 2 \end{bmatrix} lbf$$ (on post)

$V_{WSpost} := P_{WSpost.rc} + \left(N_r\right) \cdot \left(P_{WSpost.r}\right) + \sum P_{WSpost.post} = 849 \; lbf$ (total)

Factored --

$V_{uWSpost3} := \left(V_{WSpost}\right) \cdot \left(\gamma_{WS3}\right) = 0.85 \; kip$ (STR-III)

$V_{uWSpost5} := \left(V_{WSpost}\right) \cdot \left(\gamma_{WS5}\right) = 0.34 \; kip$ (STR-V)

Due to Load Combinations --

Strength III Limit State (1.25*DC + 1.00*WS) --

$V_{u3.post} := \left(\eta\right) \cdot \left(V_{uDCpost} + V_{uWSpost3}\right) = 0.89 \; kip$

$V_{u.sddl} := \dfrac{V_{u3.post}}{N_{sddl}} = 0.45 \; kip$ (per saddle)

$V_{u.b.sddl} := \dfrac{V_{u.sddl}}{N_{b.sddl}} = 223 \; lbf$ (per bolt)

Dowel Bearing (NDS 12.3) --

General --

$l_{m.sddl} := b_{fb} = 3.50 \; in$ (main member dowel bearing length)

$l_{s.sddl} := t_{sddl} = 0.125 \; in$ (side member dowel bearing length)

$F_{em.sddl} := 5600 \; psi$ (main member dowel bearing strength, parallel to grain, G = 0.50, NDS Tbl. 12.3.3)

$L_{c.sddl} := \left(2\right) \cdot \left(d_{b.sddl}\right) = 1.00 \; in$ (clear distance to edge of saddle, assumed)

$$F_{es.sddl} := \begin{array}{l} \text{if } L_{c.sddl} \le \left(2\right) \cdot \left(d_{b.sddl}\right) \\ \quad \left(2.4\right) \cdot \left(F_{u.sddl}\right) \\ \text{else} \\ \quad \left(1.2\right) \cdot \left(F_{u.sddl}\right) \end{array} = 139200 \; psi$$

(side member dowel bearing strength, LRFD Eqs. 6.13.2.9-1 & 2 modified)

$R_{e.sddl} := \frac{F_{em.sddl}}{F_{es.sddl}} = 0.04$ (bearing strength ratio, NDS Tbl 12.3.1A)

$D_{sddl} := D_{r.sddl} = 0.406\ in$ (effective diameter, threaded fastener, NDS 12.3.7(c))

$k_{3.sddl} := -1 + \sqrt{\frac{(2)\cdot(1+R_{e.sddl})}{R_{e.sddl}} + \frac{(2)\cdot(F_{yb})\cdot(2+R_{e.sddl})\cdot(D_{sddl}^{2})}{(3)\cdot(F_{em.sddl})\cdot(l_{s.sddl}^{2})}} = 11.00$ (calculation factor, NDS Tbl. 12.3.1A)

Reduction Terms (0.25" < D < 1", NDS Tbl. 12.3.1B) --

$\theta_{sddl} := 0\ deg$ (angle between direction of load and grain)

$K_{\theta.sddl} := 1 + (0.25)\cdot\left(\frac{\theta_{sddl}}{90\ deg}\right) = 1.00$ (load angle coefficient)

$R_{d.sddl.Im} := (4)\cdot(K_{\theta.sddl}) = 4.00$ (Yield Mode Im)

$R_{d.sddl.Is} := (4)\cdot(K_{\theta.sddl}) = 4.00$ (Yield Mode Is)

$R_{d.sddl.IIIs} := (3.2)\cdot(K_{\theta.sddl}) = 3.20$ (Yield Mode IIIs)

$R_{d.sddl.IV} := (3.2)\cdot(K_{\theta.sddl}) = 3.20$ (Yield Mode IV)

Yield Limit Equations (double shear, NDS Tbl. 12.3.1A)

$Z_{sddl.Im} := \frac{(D_{sddl})\cdot(l_{m.sddl})\cdot(F_{em.sddl})}{R_{d.sddl.Im}} = 1989\ lbf$ (Yield Mode Im)

$Z_{sddl.Is} := \frac{(2)\cdot(D_{sddl})\cdot(l_{s.sddl})\cdot(F_{es.sddl})}{R_{d.sddl.Is}} = 3532\ lbf$ (Yield Mode Is)

$Z_{sddl.IIIs} := \frac{(2)\cdot(k_{3.sddl})\cdot(D_{sddl})\cdot(l_{s.sddl})\cdot(F_{em.sddl})}{(2+R_{e.sddl})\cdot(R_{d.sddl.IIIs})} = 958\ lbf$ (Yield Mode IIIs)

$Z_{sddl.IV} := \frac{(2)\cdot(D_{sddl}^{2})}{R_{d.sddl.IV}}\cdot\sqrt{\frac{(2)\cdot(F_{em.sddl})\cdot(F_{yb})}{(3)\cdot(1+R_{e.sddl})}} = 1171\ lbf$ (Yield Mode IV)

$Z_{sddl} := min\left(Z_{sddl.Im}, Z_{sddl.Is}, Z_{sddl.IIIs}, Z_{sddl.IV}\right) = 958\ lbf$ (controlling, NDS 12.3.1)

Resistance and Capacity-to-Demand Check --

$V_{r.conn.sddl} := \left(\phi_{conn}\right) \cdot \left(Z_{sddl}\right) = 622.39\ lbf$ (factored resistance)

$CDR_{sddl} := \dfrac{V_{r.conn.sddl}}{V_{u.b.sddl}} = 2.79$ > 1.00 OKAY

Sway Brace Analyses --

Tensile Load and Resistance --

Tensile Load --

$Infl_L := 199.7 \dfrac{lbf}{100\ lbf}$ (influence value for axial load in brace per 100 lbs. of loading on frame, GT Strudl Bent Frame.gti)

$P_{u.L} := \left(Infl_L\right) \cdot \left(V_{u3.post}\right) = 1.78\ kip$ (sway braces alternate, so 2 post spacings contribute per brace)

Tensile Resistance (LRFD 6.8.2) --

$R_p := 0.90$ (hole type reduction factor, punched full size)

$U := 1 - \dfrac{x_{bar}}{L_{Lconn}} = 0.43$ (shear lag reduction factor, LRFD Tbl. 6.8.2.2-1)

$$P_{rt.L} := \left\| \begin{array}{l} pr_1 \leftarrow \left(\phi_y\right) \cdot \left(F_{yL}\right) \cdot \left(A_{gL}\right) \\ pr_2 \leftarrow \left(\phi_u\right) \cdot \left(F_{uL}\right) \cdot \left(A_{nL}\right) \cdot \left(R_p\right) \cdot \left(U\right) \\ \max\left(pr\right) \end{array} \right. = 24.45\ kip$$

(factored resistance, LRFD Eqs. 6.8.2.1-1 & 2)

$\dfrac{P_{rt.L}}{P_{u.L}} = 13.74$ > 1.00 OKAY

Bolted Connection at Column --

The connection is a bearing-type connection.

Shear Resistance (LRFD 6.13.2.7) --

Assume threads are included in the shear plane.

$d_{bL} := 0.50\ in$ (diameter of bolt)

$A_{bL} := 0.20\ in^2$ (area of bolt)

$N_{sL} := 1$ (number of shear planes)

$R_{ns.L} := (0.45) \cdot (A_{bL}) \cdot (F_{ub}) \cdot (N_{sL}) = 5.40\ kip$ (nominal resistance, LRFD Eq. 6.13.2.7-2)

$R_{rs.L} := (\phi_s) \cdot (R_{ns.L}) = 4.05\ kip$ (factored resistance, LRFD 6.13.2.2-1)

$\frac{R_{rs.L}}{P_{u.L}} = 2.27$ > 1.00 OKAY

Bearing Resistance at Bolt Holes (LRFD 6.13.2.9) --

$L_{c.L} := end_L - (0.5)\ (d_{hL}) = 0.69\ in$ (clear end distance)

$t_{min.L} := min\left(t_L, t_{pl.L}\right) = 0.19\ in$ (controlling minimum thickness)

$$R_{nbb.L} := \begin{Vmatrix} \text{if } L_{c.L} \le (2) \cdot (d_{bL}) \\ \quad \Vert (2.4) \cdot (d_{bL}) \cdot (t_{min.L}) \cdot (F_{uL}) \\ \text{else} \\ \quad \Vert (1.2) \cdot (L_{c.L}) \cdot (t_{min.L}) \cdot (F_{uL}) \end{Vmatrix} = 13.05\ kip$$

(nominal resistance,LRFD Eqs. 6.13.2.9-1 & 2)

$R_{rbb.L} := (\phi_{bb}) \cdot (R_{nbb.L}) = 10.44\ kip$ (factored resistance, LRFD 6.13.2.2-1)

$\frac{R_{rbb.L}}{P_{u.L}} = 5.86$ > 1.00 OKAY

Dowel Bearing (NDS 12.3) --

$$P_{u.L} = 1.78\ kip$$

$$V_{r.conn.L} := 679\ lbf$$

$$\frac{2 \cdot V_{r.conn.L}}{P_{u.L}} = 0.76$$

WESTERN ALASKA BOARDWALK ASP

SWAY BRACING (SUBSTRUCTURE) CONNECTION CALCULATIONS - IBC LOADING

Narrative:

This file is for the load demands and resistances for sway bracing and other substructure connections w/o rails.
Pedestrian live loadings on the rails are per the International Building Code.

Table of Contents:

1. References:

AASHTO LRFD Bridge Design Specifications, 9th Ed. (LRFD)
AASHTO LRFD Guide Specifications for the Design of Pedestrian Bridges (AASHTO Ped)
International Building Code, XX Ed. (IBC)
National Design Specification, 2018 Ed. (NDS)
Alaska Bridges and Structures Manual (ABSM)

2. Constants/Input Information:

General --

$kcf := 1000\ pcf$ (unit definition)

Geometry --

Deck (3 x 12, S4S) --

$t_{dk} := 2.5\ in$ (thickness)

$b_{wlk} := 10\ ft + 0\ in = 10.00\ ft$

Floorbeam (4 x 12, S4S) --

$s_{fb} := 6\ ft + 6\ in = 6.50\ ft$ (spacing)

$h_{fb} := 11.25\ in$ (height)

$b_{fb} := 3.5\ in$ (width)

$L_{fb} := 12\ ft + 0\ in = 12.00\ ft$ (length)

Posts (4 x 6, S4S) --

$s_{post} := s_{fb} = 6.50\ ft$ (spacing)

$h_{post.dk} := 42\ in = 3.50\ ft$ (height of post + rail cap, above top of deck)

$b_{post} := 3.5\ in$ (width)

$d_{post} := 5.5\ in$ (depth)

$A_{post} := (b_{post}) \cdot (d_{post}) = 19.25\ in^2$ (cross-sectional area)

$$S_{x.post} := \frac{(b_{post}) \cdot (d_{post}^{\ 2})}{6} = 17.65\ in^3$$ (section modulus, about axis parallel to boardwalk)

$$S_{y.post} := \frac{(d_{post}) \cdot (b_{post}^{\ 2})}{6} = 11.23\ in^3$$ (section modulus, about axis perpendicular to boardwalk)

$$I_{post} := \frac{(b_{post}) \cdot (d_{post}^{\ 3})}{12} = 48.53\ in^4$$ (moment of inertia)

$d_{h.post} := 0.5\ in$ (bolt hole diameter in connection)

$$S_{x.post.mod} := S_{x.post} - \frac{(d_{h.post}) \cdot (d_{post}^{\ 2})}{6} = 15.13\ in^3$$ (modified section modulus to account for bolt hole)

Rail Cap (2 x 8, S4S) --

$t_{rc} := 1.5\ in$ (thickness)

$b_{rc} := 7.25\ in$ (width)

$A_{rc} := (b_{rc}) \cdot (t_{rc}) = 10.88\ in^2$ (cross-sectional area)

$S_{rc} := \frac{(b_{rc}) \cdot (t_{rc}^{2})}{6} = 2.72\ in^3$ (section modulus)

$I_{rc} := \frac{(b_{rc}) \cdot (t_{rc}^{3})}{12} = 2.04\ in^4$ (moment of inertia)

$y_{rc} := h_{post.dk} - \frac{t_{rc}}{2} = 3.44\ ft$ (centroid of rail cap above top of deck)

Rails (2 x 6, S4S) --

$N_r := 4$ (number)

$h_r := 5.5\ in$ (height)

$b_r := 1.5\ in$ (width)

$gap_r := 3.75\ in$ < 4" IBC requirement < 6" LRFD requirement (open gap between rails)

$y_r := y_{rc} - \frac{t_{rc} + h_r}{2} - (h_r + gap_r) \cdot \begin{bmatrix} 0 \\ 1 \\ 2 \\ 3 \end{bmatrix} = \begin{bmatrix} 3.15 \\ 2.38 \\ 1.60 \\ 0.83 \end{bmatrix} ft$ (centroid of rails above top of deck)

$A_r := (b_r) \cdot (h_r) = 8.25\ in^2$ (cross-sectional area)

$S_{x.r} := \frac{(b_r) \cdot (h_r^{2})}{6} = 7.56\ in^3$ (section modulus, about horizontal axis)

$S_{y.r} := \frac{(h_r) \cdot (b_r^{2})}{6} = 2.06\ in^3$ (section modulus, about vertical axis)

$I_{x.r} := \frac{(b_r) \cdot (h_r^{3})}{12} = 20.80\ in^4$ (moment of inertia, about horizontal axis)

$I_{y.r} := \frac{(h_r) \cdot (b_r^{3})}{12} = 1.55\ in^4$ (moment of inertia, about vertical axis)

Curb Spacers and Curbs --

Curb Spacers (3 x 6) --

$h_{ks} := 2.5\ in$ (height of curb spacer)

$y_{ks} := \frac{h_{ks}}{2} = 0.10\ ft$ (centroid of curb spacer above top of deck)

Curbs (4 x 6) --

$h_k := 3.5\ in$ (height of curb)

$b_k := 5.5\ in$ (width of curb)

$S_{x.k} := \frac{(b_k) \cdot (h_k^2)}{6} = 11.23\ in^3$ (section modulus, about horizontal axis)

$S_{y.k} := \frac{(h_k) \cdot (b_k^2)}{6} = 17.65\ in^3$ (section modulus, about vertical axis)

$I_{x.k} := \frac{(b_k) \cdot (h_k^3)}{12} = 19.65\ in^4$ (moment of inertia, about horizontal axis)

$I_{y.k} := \frac{(h_k) \cdot (b_k^3)}{12} = 48.53\ in^4$ (moment of inertia, about vertical axis)

$y_k := h_{ks} + \frac{h_k}{2} = 0.35\ ft$ (centroid of curb above top of deck)

Sway Brace (L 2 x 2 x 1/4) --

$A_{gL} := 0.715\ in^2$ (cross-sectional area, gross)

$t_L := \frac{1}{4} \cdot in$ (thickness)

$x_{bar} := 0.592\ in$ (back of angle to centroid)

$d_{hL} := \frac{9}{16} \cdot in$ (diameter of hole for 1/2" dia. bolt)

$A_{nL} := A_{gL} - (t_L) \cdot (d_{hL}) = 0.574\ in^2$ (cross-sectional area, net)

$\theta_{sway} := \text{asin}\left(\frac{16\ in}{2\ ft + 10\ in}\right) = 28.07\ deg$ (angle of brace to horizontal)

$L_{Lconn} := 1\ in$ (length of bolted connection)

$end_{L.col} := 1\ in$ (distance from end of angle to center of bolt hole, at column connection)

$end_{L.fb} := 1.5\ in$ (distance from end of angle to center of bolt hole, at floorbeam connection)

$t_{pl.L} := \frac{3}{16} \cdot in$ (thickness of connection plate)

$N_{b.L.fb} := 2$ (number of bolts at floorbeam end)

$D_{r.L} := 0.406\ in$ (root diameter of 1/2" bolt, NDS Appendix Tbl. L1)

$A_{b.L} := 0.20\ in^2$ (area of bolts)

Beam Saddle (connecting columns to floorbeam) --

$N_{sddl} := 2$ (number of saddles per floorbeam)

$N_{b.sddl} := 2$ (number of bolts per saddle)

$t_{sddl} := \frac{1}{8} \cdot in$ (thickness, assumed)

$d_{b.sddl} := 0.50\ in$ (diameter of bolts)

$D_{r.sddl} := 0.406\ in$ (root diameter of 1/2" bolt, NDS Appendix Tbl. L1)

$A_{b.sddl} := 0.20\ in^2$ (area of bolts)

Material (Wood) --

Wood --

Timber Reference Design Values (Douglas Fir-Larch, No. 1 or better, LRFD Tbl. 8.4.1.1.4-1) --

The posts, rail caps, and rails are classified as "dimension lumber"

$F_{bo} := 1.200\ ksi$ (bending)

$F_{vo} := 0.180\ ksi$ (shear parallel to grain)

$F_{cpo} := 0.625\ ksi$ (compression perpendicular to grain)

$E_o := 1800\ ksi$ (modulus of elasticity)

$\gamma_w := 0.050\ kcf$ (unit weight, LRFD Tbl. 3.5.1-1)

$G_w := 0.50$ (assigned specific gravity, NDS Tbl 12.3.3A)

Adjusted Design Value for Modulus of Elasticity --

$C_{M.E} := 0.90$ (wet service factor, LRFD Tbl. 8.4.4.3-1)

$C_{i.E} := 0.95$ (incising factor, LRFD Tbl. 8.4.4.7-1)

$E := \left(E_o\right) \cdot \left(C_{M.E}\right) \cdot \left(C_{i.E}\right) = 1539\ ksi$ (modulus of elasticity, LRFD Eq. 8.4.4.1-6)

Steel (sway brace angle, ASTM A36) --

Sway Brace Angle (ASTM A36) --

$F_{yL} := 36\ ksi$ (yield stress)

$F_{uL} := 58\ ksi$ (tensile strength)

Beam Saddle (ASTM A36, assumed) --

$F_{y.sddl} := 36\ ksi$ (yield stress)

$F_{u.sddl} := 58\ ksi$ (tensile strength)

Bolts (ASTM A307 Gr. A) --

$F_{yb} := 36\ ksi$ (yield stress, not required but generally taken as this per Gr. C)

$F_{ub} := 60\ ksi$ (tensile strength, LRFD 6.4.3.2)

Load Modifiers (LRFD 1.3.1.2) --

$\eta_D := 1.00$ (ductility factor, conventional design, LRFD 1.3.3)

$\eta_R := 1.05$ (redundancy factor, rail components are nonredundant, LRFD 1.3.4)

$\eta_I := 1.00$ (operational importance factor, typical bridge, LRFD 1.3.5)

$\eta := (\eta_D) \cdot (\eta_R) \cdot (\eta_I) = 1.05$ (load modifier, max. γi appropriate, LRFD Eq. 1.3.2.1-1)

Load Factors (LRFD Tbls. 3.4.1-1 & 2) --

$\gamma_{DC} := 1.25$ (component dead load)

$\gamma_{LL1} := 1.75$ (live load, forces on rail, in Strength I limit state)

$\gamma_{LL5} := 1.35$ (live load, forces on rail, in Strength V limit state, ABSM 12.1.3.5)

$\gamma_{WS3} := 1.00$ (wind load on structure, in Strength III limit state)

$\gamma_{WS5} := 0.40$ (wind load on structure, in Strength V limit state, ABSM 12.1.3.5)

Resistance Factors --

Wood (LRFD 8.5.2.2) --

$\phi_f := 0.85$ (flexure)

$\phi_v := 0.75$ (shear)

$\phi_{cp} := 0.90$ (compression perpendicular to grain)

$\phi_{conn} := 0.65$ (connections)

Steel (LRFD 6.5.4.2) --

$\phi_u := 0.80$ (tension, facture in net section)

$\phi_y := 0.95$ (tension, yield in gross section)

$\phi_s := 0.75$ (bolts in shear, A307)

$\phi_{bb} := 0.80$ (bolts bearing on material)

<u>3. Loads (from Loads.mcdx):</u>

Live Loads on Rail (LLr) --

Use maximum effects due to distributed or concentrated loads.

$w_{LLr} := 0.050\ \mathit{klf}$ (distributed load, transversely or vertically)

$P_{LLr} := 0.20\ \mathit{kip}$ (concentrated load)

Wind Load on Structure (WS) --

$P_Z := 65.2\ \mathit{psf}$ (pressure)

$w_{WSk} := (h_k) \cdot (P_Z) = 19.0\ \mathit{plf}$ (distributed load, on curb)

$w_{WSdk} := (t_{dk}) \cdot (P_Z) = 13.6\ \mathit{plf}$ (distributed load, on decking)

$P_{WSfb} := (b_{fb}) \cdot (h_{fb}) \cdot (P_Z) = 18\ \mathit{lbf}$ (concentrated on floorbeam)

7. Substructure Connection Analyses:

Beam Saddles Analyses --

Shear on Bolts Load and Resistance --

Lateral Load (Shear) Due to Wind Load on Structure --

Unfactored (from Rails_IBC.mcdx) --

$V_{uDC} := 0 \ lbf$ (lateral dead load)

$P_{WSk} := (w_{WSk}) \cdot (s_{post}) = 124 \ lbf$ (due to curb)

$P_{WSdk} := (w_{WSdk}) \cdot (s_{post}) = 88 \ lbf$ (due to decking)

$P_{WSfb} = 18 \ lbf$ (due to decking)

$V_{WS} := P_{WSk} + P_{WSdk} + P_{WSfb} = 230 \ lbf$ (total)

Factored --

$V_{uWS3} := (V_{WS}) \cdot (\gamma_{WS3}) = 230 \ lbf$ (STR-III)

Due to Load Combinations --

Strength III Limit State (1.25*DC + 1.00*WS) --

$V_{u3} := (\eta) \cdot (V_{uDC} + V_{uWS3}) = 241 \ lbf$

$V_{u.sddl} := \dfrac{V_{u3}}{N_{sddl}} = 121 \ lbf$ (per saddle)

$V_{u.b.sddl} := \dfrac{V_{u.sddl}}{N_{b.sddl}} = 60 \ lbf$ (per bolt)

Dowel Bearing (NDS 12.3) --

General --

$l_{m.sddl} := b_{fb} = 3.50\ in$ (main member dowel bearing length)

$l_{s.sddl} := t_{sddl} = 0.125\ in$ (side member dowel bearing length)

$F_{em.sddl} := 5600\ psi$ (main member dowel bearing strength, parallel to grain, G = 0.50, NDS Tbl. 12.3.3)

$L_{c.sddl} := (2) \cdot (d_{b.sddl}) = 1.00\ in$ (clear distance to edge of saddle, assumed)

$$F_{es.sddl} := \begin{cases} \text{if } L_{c.sddl} \le (2) \cdot (d_{b.sddl}) \\ \quad (2.4) \cdot (F_{u.sddl}) \\ \text{else} \\ \quad (1.2) \cdot (F_{u.sddl}) \end{cases} = 139200\ psi$$

(side member dowel bearing strength, LRFD Eqs. 6.13.2.9-1 & 2 modified)

$$R_{e.sddl} := \frac{F_{em.sddl}}{F_{es.sddl}} = 0.04$$

(bearing strength ratio, NDS Tbl 12.3.1A)

$D_{sddl} := D_{r.sddl} = 0.406\ in$ (effective diameter, threaded fastener, NDS 12.3.7(c))

$$k_{3.sddl} := -1 + \sqrt{\frac{(2) \cdot (1 + R_{e.sddl})}{R_{e.sddl}} + \frac{(2) \cdot (F_{yb}) \cdot (2 + R_{e.sddl}) \cdot (D_{sddl}^{\ 2})}{(3) \cdot (F_{em.sddl}) \cdot (l_{s.sddl}^{\ 2})}} = 11.00$$

(calculation factor, NDS Tbl. 12.3.1A)

Reduction Terms (0.25" < D < 1", NDS Tbl. 12.3.1B) --

$\theta_{sddl} := 0\ deg$ (angle between direction of load and grain)

$$K_{\theta.sddl} := 1 + (0.25) \cdot \left(\frac{\theta_{sddl}}{90\ deg}\right) = 1.00$$

(load angle coefficient)

$R_{d.sddl.Im} := (4) \cdot (K_{\theta.sddl}) = 4.00$ (Yield Mode Im)

$R_{d.sddl.Is} := (4) \cdot (K_{\theta.sddl}) = 4.00$ (Yield Mode Is)

$R_{d.sddl.IIIs} := (3.2) \cdot (K_{\theta.sddl}) = 3.20$ (Yield Mode IIIs)

$R_{d.sddl.IV} := (3.2) \cdot (K_{\theta.sddl}) = 3.20$ (Yield Mode IV)

Yield Limit Equations (double shear, NDS Tbl. 12.3.1A)

$$Z_{sddl.Im} := \frac{(D_{sddl}) \cdot (l_{m.sddl}) \cdot (F_{em.sddl})}{R_{d.sddl.Im}} = 1989\ lbf$$ (Yield Mode Im)

$$Z_{sddl.Is} := \frac{(2) \cdot (D_{sddl}) \cdot (l_{s.sddl}) \cdot (F_{es.sddl})}{R_{d.sddl.Is}} = 3532\ lbf$$ (Yield Mode Is)

$$Z_{sddl.IIIs} := \frac{(2) \cdot (k_{3.sddl}) \cdot (D_{sddl}) \cdot (l_{s.sddl}) \cdot (F_{em.sddl})}{(2 + R_{e.sddl}) \cdot (R_{d.sddl.IIIs})} = 958\ lbf$$ (Yield Mode IIIs)

$$Z_{sddl.IV} := \frac{(2) \cdot (D_{sddl}^{\ 2})}{R_{d.sddl.IV}} \cdot \sqrt{\frac{(2) \cdot (F_{em.sddl}) \cdot (F_{yb})}{(3) \cdot (1 + R_{e.sddl})}} = 1171\ lbf$$ (Yield Mode IV)

$$Z_{sddl} := min(Z_{sddl.Im}, Z_{sddl.Is}, Z_{sddl.IIIs}, Z_{sddl.IV}) = 958\ lbf$$ (controlling, NDS 12.3.1)

Resistance and Capacity-to-Demand Check --

$V_{r.conn.sddl} := (\phi_{conn}) \cdot (Z_{sddl}) = 622.39\ lbf$ (factored resistance)

$$CDR_{sddl} := \frac{V_{r.conn.sddl}}{V_{u.b.sddl}} = 10.32$$ > 1.00 OKAY

Sway Brace Analyses --

Tensile Load and Resistance --

Tensile Load --

$$Infl_L := 199.7 \frac{lbf}{100\ lbf}$$

(influence value for axial load in brace per 100 lbs. of loading on frame, GT Strudl Bent Frame.gti)

$$P_{u.L} := (Infl_L) \cdot (V_{u3}) = 0.48\ kip$$

(sway braces at every bent)

Tensile Resistance (LRFD 6.8.2) --

$$R_p := 0.90$$

(hole type reduction factor, punched full size)

$$U := 1 - \frac{x_{bar}}{L_{Lconn}} = 0.41$$

(shear lag reduction factor, LRFD Tbl. 6.8.2.2-1)

$$P_{rt.L} := \left\| \begin{array}{l} pr_1 \leftarrow (\phi_y) \cdot (F_{yL}) \cdot (A_{gL}) \\ pr_2 \leftarrow (\phi_u) \cdot (F_{uL}) \cdot (A_{nL}) \cdot (R_p) \cdot (U) \\ \max(pr) \end{array} \right| = 24.45\ kip$$

(factored resistance, LRFD Eqs. 6.8.2.1-1 & 2

$$\frac{P_{rt.L}}{P_{u.L}} = 50.76 \quad > \quad 1.00 \quad \text{OKAY}$$

Bolted Connection at Column --

The connection is a bearing-type connection.

Shear Resistance (LRFD 6.13.2.7) --

Assume threads are included in the shear plane.

$$d_{bL} := 0.50\ in$$

(diameter of bolt)

$$A_{bL} := 0.20\ in^2$$

(area of bolt)

$$N_{sL} := 1$$

(number of shear planes)

$R_{ns.L} := (0.45) \cdot (A_{bL}) \cdot (F_{ub}) \cdot (N_{sL}) = 5.40\ kip$ (nominal resistance, LRFD Eq. 6.13.2.7-2)

$R_{rs.L} := (\phi_s) \cdot (R_{ns.L}) = 4.05\ kip$ (factored resistance, LRFD 6.13.2.2-1)

$\dfrac{R_{rs.L}}{P_{u.L}} = 8.41$ > 1.00 OKAY

Bearing Resistance at Bolt Holes (LRFD 6.13.2.9) --

$L_{c.L.col} := end_{L.col} - (0.5)\ (d_{hL}) = 0.72\ in$ (clear end distance)

$t_{min.L} := min(t_L, t_{pl.L}) = 0.19\ in$ (controlling minimum thickness)

$$R_{nbb.L} := \begin{array}{l} \text{if } L_{c.L.col} \geq (2) \cdot (d_{bL}) \\ \quad (2.4) \cdot (d_{bL}) \cdot (t_{min.L}) \cdot (F_{uL}) \\ \text{else} \\ \quad (1.2) \cdot (L_{c.L.col}) \cdot (t_{min.L}) \cdot (F_{uL}) \end{array} = 9.38\ kip$$

(nominal resistance,LRFD Eqs. 6.13.2.9-1 & 2)

$R_{rbb.L} := (\phi_{bb}) \cdot (R_{nbb.L}) = 7.50\ kip$ (factored resistance, LRFD 6.13.2.2-1)

$\dfrac{R_{rbb.L}}{P_{u.L}} = 15.58$ > 1.00 OKAY

Dowel Bearing (NDS 12.3) --

General --

$l_{m.L} := b_{fb} = 3.50\ in$ (main member dowel bearing length)

$l_{s.L} := t_L = 0.250\ in$ (side member dowel bearing length)

$F_{em.L} := 5600\ psi$ (main member dowel bearing strength, parallel to grain, G = 0.50, NDS Tbl. 12.3.3)

$L_{c.L.fb} := end_{L.fb} - (0.5)\ (d_{hL}) = 1.22\ in$ (clear distance to end of angle, recalled)

$$F_{es.L} := \begin{Vmatrix} \text{if } L_{c.L.fb} \geq (2) \cdot (d_{bL}) \\ \quad \| (2.4) \cdot (F_{uL}) \\ \text{else} \\ \quad \| (1.2) \cdot (F_{uL}) \end{Vmatrix} = 139200 \; psi$$

(side member dowel bearing strength, LRFD Eqs. 6.13.2.9-1 & 2 modified)

$$R_{e.L} := \frac{F_{em.L}}{F_{es.L}} = 0.04$$

(bearing strength ratio, NDS Tbl 12.3.1A)

$$R_{t.L} := \frac{l_{m.L}}{l_{s.L}} = 14.00$$

(thickness ratio, NDS Tbl 12.3.1A)

$$D_L := D_{r.L} = 0.406 \; in$$

(effective diameter, threaded fastener, NDS 12.3.7(c))

Calculation Factors (NDS Tbl. 12.3.1A) --

$$k_{1.L} := \frac{\sqrt{R_{e.L} + (2) \cdot \left(R_{e.L}^{\,2}\right) \cdot \left(1 + R_{t.L} + R_{t.L}^{\,2}\right) + \left(R_{t.L}^{\,2}\right) \cdot \left(R_{e.L}^{\,3}\right)} - \left(R_{e.L}\right) \cdot \left(1 + R_{t.L}\right)}{\left(1 + R_{e.L}\right)} = 0.24$$

$$k_{2.L} := -1 + \sqrt{(2) \cdot \left(1 + R_{e.L}\right) + \frac{(2) \cdot \left(F_{yb}\right) \cdot \left(1 + (2) \cdot \left(R_{e.L}\right)\right) \cdot \left(D_L^{\,2}\right)}{(3) \cdot \left(F_{em.L}\right) \cdot \left(l_{m.L}^{\,2}\right)}} = 0.46$$

$$k_{3.L} := -1 + \sqrt{\frac{(2) \cdot \left(1 + R_{e.L}\right)}{R_{e.L}} + \frac{(2) \cdot \left(F_{yb}\right) \cdot \left(2 + R_{e.L}\right) \cdot \left(D_L^{\,2}\right)}{(3) \cdot \left(F_{em.L}\right) \cdot \left(l_{s.L}^{\,2}\right)}} = 7.65$$

Reduction Terms (0.25" < D < 1", NDS Tbl. 12.3.1B) --

$$\theta_L := \operatorname{asin}\left(\frac{16 \; in}{2 \; ft + 10 \; in}\right) = 28.07 \; deg$$

(angle between direction of load and grain)

$$K_{\theta.L} := 1 + (0.25) \cdot \left(\frac{\theta_L}{90 \; deg}\right) = 1.08$$

(load angle coefficient)

$R_{d.L.Im} := (4) \cdot (K_{\theta.L}) = 4.31$ (Yield Mode Im)

$R_{d.L.Is} := (4) \cdot (K_{\theta.L}) = 4.31$ (Yield Mode Is)

$R_{d.L.II} := (3.6) \cdot (K_{\theta.L}) = 3.88$ (Yield Mode II)

$R_{d.L.III} := (3.2) \cdot (K_{\theta.L}) = 3.45$ (Yield Mode IIIm & IIIs)

$R_{d.L.IV} := (3.2) \cdot (K_{\theta.L}) = 3.45$ (Yield Mode IV)

Yield Limit Equations (single shear, NDS Tbl. 12.3.1A)

$$Z_{L.Im} := \frac{(D_L) \cdot (l_{m.L}) \cdot (F_{em.L})}{R_{d.L.Im}} = 1845 \; lbf$$ (Yield Mode Im)

$$Z_{L.Is} := \frac{(D_L) \cdot (l_{s.L}) \cdot (F_{es.L})}{R_{d.L.Is}} = 3277 \; lbf$$ (Yield Mode Is)

$$Z_{L.II} := \frac{(k_{1.L}) \cdot (D_L) \cdot (l_{s.L}) \cdot (F_{es.L})}{R_{d.L.II}} = 891 \; lbf$$ (Yield Mode II)

$$Z_{L.IIIm} := \frac{(k_{2.L}) \cdot (D_L) \cdot (l_{m.L}) \cdot (F_{em.L})}{(1 + (2) \cdot (R_{e.L})) \cdot (R_{d.L.III})} = 990 \; lbf$$ (Yield Mode IIIm)

$$Z_{L.IIIs} := \frac{(k_{3.L}) \cdot (D_L) \cdot (l_{s.L}) \cdot (F_{em.L})}{(2 + R_{e.L}) \cdot (R_{d.L.III})} = 618 \; lbf$$ (Yield Mode IIIs)

$$Z_{L.IV} := \frac{(2) \cdot (D_L{}^2)}{R_{d.L.IV}} \cdot \sqrt{\frac{(2) \cdot (F_{em.L}) \cdot (F_{yb})}{(3) \cdot (1 + R_{e.L})}} = 1086 \; lbf$$ (Yield Mode IV)

$$Z_L := min(Z_{L.Im}, Z_{L.Is}, Z_{L.II}, Z_{L.IIIm}, Z_{L.IIIs}, Z_{L.IV}) = 618 \; lbf$$ (controlling, NDS 12.3.1)

Resistance and Capacity-to-Demand Check --

$V_{r.conn.L} := (N_{b.L.fb}) \cdot (\phi_{conn}) \cdot (Z_L) = 803 \; lbf$ (factored resistance)

$$CDR_L := \frac{V_{r.conn.L}}{P_{u.L}} = 1.67$$ > 1.00 OKAY