8. WOOD STRUCTURES

Wood is used for many bridge applications. It is used as a primary structural material for permanent bridges on secondary roads (e.g., decks, beams, and pile caps), and is used in temporary bridges on both secondary and major roads. It is often used for formwork and falsework on bridges with cast-in-place concrete elements. This section provides general design and detailing guidance for the LRFD design of longitudinal and transverse decks, glulam beams, and pile caps. It concludes with four design examples: a longitudinal spike laminated deck, a timber pile cap, a glulam beam superstructure, and a transverse deck on glulam beams. The transverse deck example goes through the design of two deck types, a transverse spike laminated and a transverse glulam. Wood bridge design is governed by the current edition of *AASHTO LRFD Bridge Design Specifications* including current interims, hereinafter referred to as AASHTO LRFD.

The design examples are followed by load rating examples for the elements designed in the design examples, except for the timber cap, because substructures are typically not load rated on new structures. Information on wood incorporated into the design of formwork and falsework can be found in the *MnDOT Bridge Construction Manual*. The construction of timber bridges is governed by *MnDOT Standard Specifications for Construction*, (MnDOT Std. Spec.,) Article 2403, Wood Bridge Construction.

8.1 Materials

A variety of materials are incorporated into timber bridges, ranging from treated solid and laminated wood members to steel fasteners and hardware, as well as steel plates and shapes used as bracing or in connections.

This section briefly defines some commonly used terms for various wood materials:

Lumber
In general, lumber is defined as wood that is sawed, or sawed and planed.

In this chapter, lumber is commonly used in the term "dimension lumber", which is lumber that is nominal 2 to 4 inches thick on its edge, by 2 inches or more in width.

Timber

Timber is a term referring to larger pieces of lumber. For the purposes of this chapter the ASTM definition is applied, timber is lumber that is 5 inches thick and larger on its least dimension face.

Wood

The part of a tree inside of the bark, harvested and prepared for use as lumber and timbers to build structures; in the case of this section, constructing bridges. Specific species to be used are given in Article 8.1.1 below.

Glulam Timber

Glulam is short for "glued laminated" timber. Glued laminated timber is comprised of surfaced dimension lumber used as laminates and glued together in a factory to form larger timbers. The glulam timbers are commonly used for bridge beams and also for decks. The decks span either longitudinally between supports or transversely on beams. Frequency of glulam usage in decks varies by region around the country.

Spike Laminated Decks

Spike laminated decks are comprised of dimension lumber assembled in the shop to form deck panels, which are installed on supports in the field. Older spike laminated decks (generally 1970's and prior) were completely assembled in the field. Assembly (in the field or panels in the shop) consists of laying dimension lumber edgewise as laminates and driving large steel spikes through the wider faces of multiple layers of laminates in a pattern specified in AASHTO LRFD. These spike laminated decks are used transverse to the center line of road and supported on beams (deck thicknesses usually 6 to 8 inches thick measured vertically) or are used parallel to the centerline of road as longitudinal decks spanning between floor beams or substructures (deck thicknesses usually 8 to 18 inches thick). In AASHTO LRFD the term "spike laminated decks" is used, but these decks are sometimes also referred to as nail laminated or dowel laminated.

8.1.1 Wood Products

Structural wood products typically shall be visually graded West Coast Douglas Fir or Southern (Yellow) Pine as a standard. Other species should receive State Bridge Design Engineer approval prior to final design if it is intended to specify another species for use in a bridge. Refer to MnDOT Standard Spec., Art. 3426 Structural Wood. Designs should be based on the design values found in AASHTO LRFD. Design values not given in

AASHTO LRFD shall be obtained from the *National Design Specification for Wood Construction* (NDS).

[Table 8.4.1.1.4-1]

The AASHTO LRFD tabulated design values assume dry-use conditions. These tabulated values shall be modified if wood will be subject to wet use conditions. Table 8.1.1.1. has an abbreviated list of some typical design values for Douglas Fir-Larch, which is a common species used in bridges.

Table 8.1.1.1 – Reference Design and Modulus of Elasticity Values

Visually-Graded Sawn Lumber								
Species and Commercial Grade	Size Classification	Design Values (KSI)						
		F_{bo}	F_{to}	F_{vo}	F_{cpo}	F_{co}	E_o
Douglas Fir-Larch							
No. 1	Dimension* ≥ 2 in. Wide	1.00	0.675	0.18	0.625	1.50	1,700
Select Structural	B&S**	1.60	0.95	0.17	0.625	1.10	1,600
Select Structural	P&T***	1.50	1.00	0.17	0.625	1.15	1,600

[8.2 - Definitions]

* Dimension Lumber Sizes, see AASHTO LRFD for definition

** Beams and Stringers Sizes, see AASHTO LRFD for definition

*** Posts and Timbers Sizes, see AASHTO LRFD for definition

All wood members, that become part of the permanent bridge structure, should be treated with a preservative. Preservatives protect the wood against decay and organisms. Refer to Article 8.1.3 in this section for wood preservative information.

[8.4.1.1.2]

Lumber and timbers can be supplied in various finished sizes, depending on the sawing and planing done at the time of manufacture. Following are general definitions of some common finished sizes. Grading rules for specific species should be referenced if dimensions are important to the design for lumber that is not dressed (not planed), or surfacing can be specified as needed.

Full sawn
Sawed full to the specified size with no undersize tolerance allowed at the time that the lumber is manufactured.

Rough sawn
Lumber sawed to the specified size and not planed, and with small tolerances permitted under the specified size.

Standard sawn
Lumber sawed to size but not planed, and with minimum rough green sizes slightly less than rough sawn.

Dressed lumber, or surfaced lumber (S4S, S1S, etc.)
Lumber that has been sawed, and then surfaced by planing on one or more sides or edges. The most common is surfaced 4 sides (S4S). Sometimes if a specific dimension is needed by the design only 1 side is surfaced (S1S), or other combinations of sides and edges can be specified. Standard surfaced sizes can be referenced in the NDS.

The actual dimensions and moisture content used in the design should be indicated in the contract documents. MnDOT policy is to design for wet-use conditions (8.2.1 and 8.4.3).

[Table 3.5.1-1]

The design unit weight of most components is 0.050 kcf. Douglas Fir and Southern Pine are considered soft woods. For special designs using hard woods, the design unit weight is 0.060 kcf.

[9.9.3.4]

The coefficient of thermal expansion of wood parallel to its fibers is 0.000002 inch/inch/°F. AASHTO LRFD Article 9.9.3.4 provides design guidance on applicability of considering thermal effects.

8.1.2 Fasteners and Hardware

Structural steel elements incorporated into timber bridges must satisfy the strength and stability checks contained in Section 6 of the LRFD Specifications. For durability, generally all steel elements incorporated into timber bridges are hot-dipped galvanized. Compatibility of steel elements and hardware with the specified wood preservative shall be investigated. Some waterborne treatments actively corrode steel and hardware. Oil-type preservatives are generally compatible with steel and hardware and do not directly cause damage from reactivity. Use of uncoated steel (such as weathering steel) in wood bridges should be used with great caution to make certain durability is not compromised.

8.1.3 Wood Preservatives

Wood preservatives are broadly classified as oil-type or waterborne preservatives. All wood used in permanent structures must be treated with a preservative. Preservatives on the MnDOT approved list are to be specified for treated wood materials. Other preservative treatments can be used on an individual basis if a local agency conducts its own liability analysis for the preservative treatment proposed. Oil-type preservatives are not to be used where contact with pedestrians occurs. Preservatives used for pedestrian applications shall be safe for skin contact.

Oil-Type Preservatives

The three most common oil-type preservatives that have been used in the past, or are currently being used in bridge applications are: creosote, pentachlorophenol, and copper naphthenate. The descriptions below are provided for general information only. As stated above, the MnDOT approved list shall be reviewed by the designer and owner. For bridge applications, oil-type preservatives are used almost exclusively for treating structural components. They provide good protection from decay, and provide a moisture barrier for wood that does not have splits. Because most oil-type treatments can cause skin irritations, they should not be used for applications that require repeated human or animal contact, such as handrails, safety rails, rub rails, or decks.

Creosote

Historically, creosote has been the most commonly used preservative in bridge applications in Minnesota. The high level of insoluables can result in excessive bleeding of the treatment from the timber surface, which can create a hazard when it contacts human skin. Creosote is an Environmental Protection Agency (EPA) restricted use pesticide. It should be noted that creosote is no longer on MnDOT's list of approved preservatives for the treatment of timber products.

Pentachlorophenol

As a wood preservative penta is effective when used in ground contact, in freshwater, or used above ground. Penta is difficult to paint and should not be used in applications subject to prolonged human or animal contact. Penta is an EPA restricted use pesticide. The penta producers have created guidance on the handling and site precautions with using this product.

Copper Naphthenate

Copper Napthenate is effective when used in ground or water contact, and above ground. Unlike creosote and penta, Copper Napthenate is not listed as a restricted use pesticide. However, precautions (dust masks, gloves, etc.) should be used when working with this wood treatment.

Waterborne Preservatives

Waterborne preservatives are used most frequently for railings and floors on bridge sidewalks, pedestrian bridges and boardwalks, or other areas that may receive human contact. After drying, wood surfaces treated with these preservatives can also be painted or stained. Of the numerous waterborne preservatives, CCA, ACQ, and CA have been used in bridge applications in the past. Each of these preservatives is strongly bound to the wood, thereby reducing the risk of chemical leaching.

CCA (Chromated Copper Arsenate)
CCA is an EPA restricted use pesticide that was generally used in the past to treat Southern Pine and other (easier to treat) wood species. The use of this product has been phased out because of environmental concerns with arsenic.

EnviroSafe Plus®
EnviroSafe Plus® is a borate based preservative treatment using Disodium Octaborate Tetrahydrate and a patented polymer binder. It contains no heavy metals, which can raise health, environmental, and disposal concerns. This treatment is not considered a problem for human contact, but it is not to be used for members in contact with the ground.

ACQ (Alkaline Copper Quaternary)
Multiple variations of ACQ have been standardized. ACQ was developed to meet market demands for alternatives to CCA. This product accelerates corrosion of metal fasteners. Hot dipped galvanized metal or stainless steel fasteners must be used to avoid premature fastener failure.

MCA (Micronized Copper Azole)
As the use of CCA was phased out, some wood suppliers began using CA waterborne preservatives, which evolved into the use of micronized CA (which uses micro sized copper particles). MCA treatments are considered to be less corrosive than CA and ACQ. However, at minimum to ensure durability, hot dipped galvanized hardware and steel should be used with MCA treated wood.

8.2 Timber Bridge Decks

Wood or timber decks can be incorporated into a bridge in a number of different ways. Decks can be the primary structural element that spans from substructure unit to substructure unit or floor beam to floor beam, such as a longitudinal spike laminated deck.

Wood decks can also be secondary members used to carry vehicle or pedestrian loads to other primary members such as beams, stringers, or girders. As secondary members decks can be transverse spike laminated, transverse glulam, or simple transverse planks which are installed flatwise. Analysis modelling is described in 8.4.3.

8.2.1 General Design and Detailing

Section 9 of the AASHTO LRFD Specifications (Decks and Deck Systems) provides information on the design and detailing of decks. Designing specifically for wood decks is covered in Article 9.9. Some common longitudinal deck types are further described in Article 8.2.3 of this section.

Applicability of Use

AASHTO LRFD recommends limitations on the use of deck types as a guide to bridge owners and designers so that maintenance over the life of the bridge remains within expectations and does not become excessive.

[C9.9.6.1] The use of spike laminated decks should be limited to secondary roads with low truck volumes, ADTT significantly less than 100 trucks per day.

[C9.9.4.1] The recommended use for glulam decks is somewhat vague, but glulam decks should also be limited to secondary roads with low truck volumes. AASHTO LRFD states that this form of deck is appropriate only for roads having low to medium volumes of commercial vehicles.

[9.9.2] Minimum thicknesses are specified in AASHTO LRFD for wood decks. The nominal thickness of wood decks other than plank decks shall not be less than 6.0 in. The nominal thickness of plank decks for roadways shall not be less than 4.0 in.

[C9.9.7.1] Plank decks should be limited to low volume roads that carry little or no heavy vehicles. Plank decks do not readily accept and/or retain a bituminous surface. This deck type can sometimes be used economically on temporary bridges where wear course maintenance is less important. Thicker planks that provide higher capacity are economical if used or salvaged lumber can be incorporated into a temporary bridge.

In addition to reviewing applicability of a timber bridge based on traffic demands at the site, hydraulic considerations also need to be considered and the State Aid Bridge Hydraulic Guidelines must be followed in determining a low member elevation.

Geometry

Spike laminated timber deck panels should be laid out with panel widths that are multiples of 4 inches, which currently is the typical deck laminate width dimension. Glulam deck panels should be designed for standard laminate sizes based on the wood species. To facilitate shipping, deck panels should be detailed with plan widths less than 7’–6”. Large and thick deck panels should have the lifting method and weight reviewed, to prevent damage to the wood.

[8.4.4.3]

Moisture Conditions

MnDot policy is for designs to be based on wet use conditions (>16% moisture content for glulam and >19% for sawn members). Applicable moisture factors are provided in AASHTO LRFD Table 8.4.4.3-1 for sawn lumber and 8.4.4.3–2 for glulam.

Bituminous Wearing Surface

[9.9.3.5] *[C9.9.7.1]* *[9.9.8.2]*

AASHTO LRFD Article 9.9.3.5 requires a wearing surface conforming to Article 9.9.8 on wood decks. AASHTO LRFD Article C9.9.8.1 recommends bituminous wearing surfaces for timber decks, except for decks consisting of planks installed flatwise that will not readily accept and/or retain a bituminous wearing surface. It also recommends that deck material be treated using the empty cell process followed by an expansion bath or steaming. The bituminous wearing course should have a minimum compacted depth of 2 inches. For proper drainage, MnDOT recommends a cross slope of 0.02 ft/ft whenever practicable. The Spike Laminated Decks section below includes some discussion pertaining to maintenance of bituminous wearing surface, which has some applicability to all deck types.

8.2.2 Loads

Dead Load

MnDOT uses a unit weight of 0.150 kcf for the bituminous wearing surface dead load (MnDOT Table 3.3.1). A 0.020 ksf dead load is to be included in all designs in order to accommodate a possible future wearing surface. The timber rail system is equally distributed across the deck, or equally to all beams.

Live Load

[3.6.1/3.6.2.3]

Live load and live load application shall be in accordance with AASHTO LRFD. Dynamic load allowance need not be applied to wood components.

[9.9.3.1]

For timber structures with longitudinal flooring, the live load shall be distributed using the appropriate method. Glulam and spike laminated are discussed below including under the spreader beam section because the appropriate method will typically require the use of a spreader beam. Transverse and longitudinal decks with planks installed flatwise (wood plank decks) are discussed in AASHTO LRFD Article 4.6.2.1.3. Tire contact area and dimensions are defined in LRFD Article 3.6.1.2.5.

8.2.3 Longitudinal Wood Decks

Three types of wood decks that function as primary structural elements spanning longitudinally are used in Minnesota; glulam panels, stress laminated decks, and spike laminated decks. However, stress-laminated decks are considered non-standard and the design approach should receive approval from the State Bridge Design Engineer prior to final design. Calculations with validation are required for non-standard designs. Approval should also be obtained for other less common deck types and for less common materials, such as Parallel Strand Lumber (PSL), Fiber Reinforced Polymer wood (FRP), or wood species other than Douglas Fir or Southern (Yellow) Pine.

In addition, skews over 20° require special consideration and coordination with the State Bridge Design Engineer to assure proper support for the top of the abutments to prevent superstructure instability, and to confirm the method of analysis for the longitudinal deck. Individual designs may require more or less attention depending on magnitude of skew, abutment type (concrete or timber), abutment height, soil conditions, etc.

To prevent movement of the deck panels in the completed structure, positive attachment is required between the panels and the supporting component (See Article 8.2.5 of this manual).

[9.9.4]

Glulam Decks

Glulam wood deck panels consist of a series of panels, prefabricated with water-resistant adhesives, which are tightly abutted along their edges. Stiffener beams, or spreader beams, are used to ensure load distribution between panels. It is recommended to obtain approval on the design approach for this deck type since it is not a common design in Minnesota.

[9.9.5]

Stress Laminated Decks

Stress laminated decks consist of a series of wood laminations that are placed edgewise and post-tensioned together, normal to the direction of the lamination.

[9.9.5.6]

In stress laminated decks, with skew angles less than 25°, stressing bars should be detailed parallel to the skew. For skew angles between 25° and 45°, the bars should be detailed perpendicular to the laminations, and in the end zones, the transverse prestressing bars should be fanned in plan or arranged in a step pattern. Stress laminated decks should not be used for skew angles exceeding 45°. AASHTO LRFD Article 9.9.5 contains design and detailing guidance for stress laminated decks.

[9.9.6]

Spike Laminated Decks

Spike laminated decks consist of a series of dimension lumber laminations that are placed edgewise between supports and spiked together on their wide face. The laminated deck is prefabricated at a plant in panels that are shipped to the site. The connection between adjacent panels most commonly used in current industry practice is a ship-lap joint, but AASHTO LRFD does not directly give credit to the ship-lap joint for transfer of wheel loads. In accordance with AASHTO LRFD, spreader beams are required to ensure proper load distribution between panels (see below). The laminates are treated with preservative after drilling pilot holes for the spikes, and prior to assembling and installing spikes in the panels. Butt splicing of laminations within their unsupported length is not allowed.

The use of these decks is limited to secondary roads with low truck volumes (i.e. ADTT significantly less than 100 trucks per day). Frequent heavy truck loading may increase bituminous cracking resulting in accelerated bituminous deterioration and increased maintenance. To reduce future bituminous maintenance, the owner could elect to over design the deck or incorporate the use of geotextiles in the bituminous wearing surface. Waterproofing may be considered, but careful attention to details is required to avoid direct contact between fresh oil-type treatments and rubberized water proofing, to prevent degradation of the waterproofing membrane which results in liquidation of the membrane.

[4.6.2.3]

Spreader Beams

Spreader beams, or transverse stiffener beams, are attached to the underside of longitudinal glulam and spike laminated decks as a method for panels to be considered interconnected by design.

AASHTO LRFD Table 4.6.2.3-1 shows a schematic for longitudinal laminated decks (glulam and spike laminated). AASHTO LRFD requires spans exceeding 15.0 feet to be designed according to the provisions of Article 4.6.2.3, which includes the use of spreader beams. AASHTO LRFD Article 9.9.4.3 gives minimum spreader (or stiffener) beam requirements. The rigidity, EI, of each spreader beam cannot be less than 80,000 kip-in^2. The spreader beams must be attached to each deck panel near the panel edges and at intervals not exceeding 15.0 inches. The spreader beam spacing is not to exceed 8.0 ft.

Research has shown spreader beams to be effective in transferring load between panels and the spreader beams stiffen longitudinal decks in the transverse direction. One such research project by the University of Minnesota that was published in January 2003 used 6 inch wide x 12 inch deep spreader beams which are a common industry standard. MnDOT approves of using 6 inch wide x 12 inch deep spreader beams at the AASHTO specified maximum spreader beam spacing of 8 feet. Closer spacing can be used to reduce bituminous cracking, including on an existing bridge.

[9.9.3]

Decks with spans 15.0 feet and less may be designed by one of the three methods given in AASHTO LRFD. The simplest method is Article 4.6.2.1. However, experience has shown that this method may result in thicker decks compared to other methods. If approved by the State Bridge Design Engineer on a per project basis, spans 15.0 feet and less could be designed by Article 4.6.2.3, which includes the use of a spreader beam.

8.2.4 Design/ Analysis

Most longitudinal wood decks will be designed per AASHTO LRFD Article 4.6.2.3 and incorporate the use of spreader beams. Exterior strips or edge beams are not specifically designed for on timber deck bridges with spreader beams. MnDOT designs are performed on a unit strip one foot wide. Manipulate the code values (invert and multiply by 12) to determine distribution factors on a per foot basis.

MnDOT design span lengths are center to center of bearing at support for the longitudinal wood member being designed. This simplification was adopted in response to what designers in the local industry generally use.

The maximum span length for a given deck thickness is dependent on several factors including: superstructure type, wood species and grade, deck width, and live load deflection. Table 8.2.4.1 provides typical deck thicknesses and design span lengths for various longitudinal deck configurations. Table 8.2.4.2 contains typical design span lengths for longitudinal spike laminated deck thicknesses ranging from 10 to 18 inches. Actual design span lengths must be verified with calculations for the species and grade of wood used in a particular deck.

Table 8.2.4.1 – Typical Designs Spans for Various Longitudinal Timber Deck Systems

Superstructure Type	Deck Thickness (in)	Design Span Length (ft)
Sawn Lumber Deck Systems		
Spike-Laminated	10-18	10-35
Stress-Laminated	10-18	10-35
Glulam Deck Systems		
Standard Panel	8-16	10-37
Post-Tensioned	9-24	10-50

Table 8.2.4.2 – Typical Span Lengths for Longitudinal Spike Laminated Sawn Deck Thicknesses

Deck Thickness (in)	Typical Max. Design Span Length (ft)
10	≤ 10
12	≤ 17
14	≤ 25
16	≤ 31
18	≤ 35

Load Distribution and Modeling

All spans are designed as simple spans. Check bending of deck using size factor, if applicable. Also check deflection, horizontal shear, and compression perpendicular to the grain.

8.2.5 Detailing

[9.9.4.2]

[9.9.5.5]

Typically metal plate connectors are used to attach longitudinal deck panels to pile caps at piers to engage the deck in each span. Lag screws or deformed shank spikes can be used through the metal plate connectors down to wood supports. At minimum, detail no less than two metal tie-down plates per deck panel. The spacing of the tie-downs along each support shall not exceed 3.0 feet for stress laminated decks. Tie-downs at abutments shall have the same quantity and spacing requirements, but metal plates are not required unless large washers are determined as needed by the designer.

AASHTO LRFD provides guidance for longitudinal deck tie-downs based on standard practice for glulam and spike laminated decks, and higher strength tie-down for stress laminated decks. The designer shall consider individual site conditions (such as design flood elevation and possible buoyancy forces) to make the determination as to if tie-downs are adequate for a specific structure. The USDA Forest Service recommends through bolting from the superstructure to substructure with timber cap beams, and grouted anchors if concrete substructures are used.

[9.9.6.1]

The requirements in Article 9.9.6.1 of AASHTO LRFD are to be followed for spike placement in spike laminated decks. Spikes shall be of sufficient length to totally penetrate four laminations, and placed in lead holes through pairs of laminations at intervals not greater than 12.0 inches in an alternating pattern top and bottom. (AASHTO Figure 9.9.6.1-1). Laminations shall not be butt spliced within their unsupported length. Drive spike spacing at ship-lap joints is calculated by the designer.

8.3 Timber Bridge Superstructures

Wood components can be and have been incorporated into bridge superstructures in a wide variety of applications. Article 8.2 outlined several different deck types that can span longitudinally from substructure to substructure or from floor beam to floor beam. The longitudinal spike laminated deck was the most common timber bridge type constructed in Minnesota for many years, and a large number of these bridges remain in existence.

The most common timber bridge type in Minnesota for longer spans consists of glulam beams with transverse wood decks. In Minnesota, the transverse decks on glulam beams traditionally have been spike laminated. Transverse glulam decks recently have become more common for some newer installations. Nationwide, transverse glulam decks are the more common deck type on glulam beams. The analysis and detailing of this bridge type is not complex and a design example is provided in this section. Transverse wood decks are also used on sawn beams, but in the span ranges that sawn timber beams can be used longitudinally, spike laminated deck superstructures currently are usually more economical. Many sawn beam bridges remain in existence around Minnesota.

Wood is also used in hybrid superstructures. The most common is transverse wood decking on steel beams. Although this superstructure type is currently considered non-standard for new permanent bridge installations with State funding, it is commonly used for temporary bridges. It is also used for bridges on very low volume roads and private bridges.

Other less common hybrids and configurations exist for timber bridge superstructures. Special designs incorporating wood components are sometimes desired for aesthetic purposes, especially in span lengths that traditionally accommodate wood members. Once again, if considering non-standard superstructure types, the design approach should receive approval from the State Bridge Design Engineer prior to final design. Some examples of special designs that increase strength of timber components are transverse post-tensioned glulam beams with a laminated deck and fiber reinforced polymer glulam beams (FRP). Examples of special designs with increased aesthetic appeal are glulam girder or arch spans, and wood truss spans.

8.3.1 Camber / Deflections

MnDOT does not require wood decks to be fabricated with specific camber values. During fabrication of panels, if there is any natural camber of the deck it should be planned to be placed up to reduce the appearance of sag in a span. Longitudinal panels comprised of glulam laminates spiked together can reach longer span lengths and may need to be designed with camber. Design glulam beams for camber of dead load deflection plus long term creep.

8.4 Timber Pile Caps/Substructures

Timber pile caps are most commonly used for timber bridges, supported on cast-in-place piles. As a standard, large sawn timbers are used for caps. Special designs sometimes use glulam caps. Due to the low stiffness of timber caps that are relatively slender, equal load distribution to the piles supporting the cap is not to be assumed when calculating pile loads. A continuous beam model similar to that used for analyzing the cap to determine reactions (see Art. 8.4.3 below), is to be used when calculating the loads for the piles supporting a timber cap.

8.4.1 Substructure Details

Typically, 12 inch cast-in-place piles are to be used in abutments, and 16 inch cast-in-place piles are to be used in piers unless project specific approval is obtained. MnDOT does not allow the use of timber piles for main structural support (support of caps). Timber piles may be used at wingwall ends. If soil conditions do not allow the use of cast-in-place piles, steel H-piles with special details may be used. If H-piles are used, all pier piles shall be encased in pile shells.

To prevent uplift and movements, pile caps must have positive attachment to the piles. Similar to detailing for decks, the designer shall review individual site conditions and determine adequate cap to pile connections. Consider using concrete caps at sites with high debris, ice jams, or potentially high buoyancy forces. Concrete caps can be painted brown if desired for aesthetic reasons. In reviewing site conditions, the State Aid Bridge Hydraulic Guidelines must be followed, and pile embedment and unsupported length considering scour also need to be evaluated.

8.4.2 Geometry

MnDOT's standard timber abutment is 4 foot maximum clear height on the front face from ground elevation to bottom of superstructure. Tie backs for abutments are not standard. Backing planks are normally 3 inch x 12 inch or 4 inch x 12 inch. The designer shall verify backing plank size and pile spacing based on at-rest earth pressure. Passive pressure used for concrete abutment design need not be considered since timber abutments are less rigid, and wood bridges have negligible temperature expansion. Other abutment configurations, dimensions, or with tie-backs (which may be required, for example, on larger skews) are to receive approval by the State Bridge Design Engineer prior to final design.

The standard timber size for abutment pile caps is 14 inch x 14 inch. Pier pile caps are 16 inch x 16 inch. Designers should use a maximum length of 36 feet for cap timbers, or verify availability of longer lengths. This constraint may require a splice in the pile cap. If a splice is necessary, it should be located over an internal pile.

8.4.3 Design / Analysis

Design for a wet-use condition.

For design of the cap, assume that the railing weight is uniformly distributed across the cap.

When analyzing pile caps and transverse decks use three different models:

1) a simply supported span in determining the positive bending moment
2) a fixed-fixed span in determining the negative bending moment
3) a continuous beam (with a hinge to represent a splice) in determining the shear forces and reactions

The third model requires the live load to be placed at various locations along the span to determine the critical member forces. This is illustrated in the design examples.

8.4.4 Camber / Deflections

Timber pile caps are not cambered. Deflection normally does not control the design of a cap due to the short design spans.

8.5 Railings

Railings used on timber bridges shall be crash tested rail systems for the appropriate application; such as longitudinal timber deck, transverse timber deck on beams, etc. Timber railings are sometimes used on concrete decks for aesthetic reasons, and standard plans of crash tested railings for this application are also available.

In general, rail systems must conform to the requirements of Section 13 of the AASHTO LRFD and crash tested in accordance with *NCHRP Report 350 Recommended Procedures for the Safety Performance Evaluation of Highway Features.*

Crash tested timber railing systems can be found on the FHWA website: http://safety.fhwa.dot.gov/roadway_dept/policy_guide/road_hardware/barriers/bridgerailings/docs/appendixb7h.pdf

http://safety.fhwa.dot.gov/roadway_dept/policy_guide/road_hardware/barriers/bridgerailings/docs/appendixb5.pdf

Standard plan sheets are available on the USDA Forest Services Website: www.fpl.fs.fed.us A search for "standard plans" produces many standard plans related to timber bridges, including for crash tested rail systems that were created under a cooperative effort including the University of Nebraska–Lincoln, the USDA Forest Service, Forest Products Laboratory, and FHWA.

[13.7.2]

In addition to a crash tested rail system for the proper bridge superstructure configuration, the rail system must be crash tested at the proper Test Level for the bridge traffic usage. Test Level selection criteria can be found in Article 13.7.2 of AASHTO LRFD, and Table 13.7.2-1 has crash test criteria.

Section 13 of this Manual covers bridge railings and barriers. Article 13.2.1 gives requirements based on speed.

8.6 Additional References

Additional wood design information for use in designing wood bridges is available in the following references:

1) *National Design Specifications – Wood Construction* (NDS)
2) *Timber Construction Manual* (AITC)
3) Ritter, M.A., *Timber Bridges, Design, Construction, Inspection and Maintenance*, EM7700-B. Forest Service, U.S. Department of Agriculture, Washington, D.C., 1990
4) *National Conference on Wood Transportation Structures* (NCWTS)
5) AASHTO LRFD 8.14 has an extensive list of References

8.7 Design Examples

Article 8.7 demonstrates the design of multiple bridge elements in accordance with AASHTO LRFD through several design examples. The design examples include a longitudinal spike laminated deck, a timber pile cap on pier piling, a glulam beam superstructure, and the transverse deck on the glulam beams. The transverse deck example goes through the design of two different deck types, a transverse spike laminated and a transverse glulam.

8.7.1 Longitudinal Spike Laminated Timber Deck Design Example

This first example goes through the design of a longitudinal spike laminated timber bridge deck. There are no longitudinal girders in the bridge, and so this bridge type is also sometimes generically referred to as a timber slab span. It should be noted that these bridge decks are usually reserved for secondary roads with low truck traffic volumes.

The deck panel span under investigation is an "interior" strip of an intermediate span, which spans from one pile cap to another pile cap. Refer to Figure 8.7.1.1 which shows the general layout and dimensions. In addition, Article 8.7.2 of this manual contains the example design of the timber pile cap which provides support bearing for the beginning and end of this longitudinal deck span.

A. Material and Design Parameters

[Figure 8.3-1]

The dimension annotations used throughout this design example are as follows. The vertical dimension of a member is considered its depth. The transverse and longitudinal measurements of a member are considered its width and length, respectively. These dimension annotations are consistent with Figure 8.3-1 of the *2014 AASHTO LRFD Bridge Design Specifications*, except for sawn lumber descriptive names. The letter notations will be used in this example (b, d, etc.).

Nominal dimensions of sawn lumber are always used for dead load calculations. The dimensions used for calculating member capacity need to be determined for each individual case depending on the actual surfacing specified and supplied. These are commented on below.

[8.4.1.1]

1. Pile Cap

Width of the pile cap member = b_{cap} = 16 in
Depth of the pile cap member = d_{cap} = 16 in

16 inch x 16 inch pile caps are supplied as rough sawn. For rough sawn, MnDOT allows the use of these dimensions as actual (for rough sawn, slight tolerances are permitted at the time of manufacture). The validity of the pile cap dimensions used here will be later checked in Article 8.7.2 of this manual.

[9.9.8]

2. Bituminous Wearing Surface

MnDOT uses a 2% cross slope whenever practicable. In this case, a minimum thickness of 2 in at edge of roadway (face of curb) and 6 in thickness at centerline of the road gives an average depth of wearing course = 4 in. Therefore, the bituminous wearing course thickness used for dead load calculations = d_{ws} = 4 in.

3. Curb and Railing [TL-4 Glulam Timber Rail with Curb]
Width of timber curb = b_{curb} = 12 in
Depth of timber curb = d_{curb} = 6 in
Width of timber rail post = b_{post} = 10 in
Length of timber rail post = L_{post} = 8 in
Depth of timber rail post = d_{post} = 47 in
Width of timber spacer block = b_{spacer} = 4.75 in
Length of timber spacer block = L_{spacer} = 8 in
Depth of timber spacer block = d_{spacer} = 13.5 in
Width of timber scupper = $b_{scupper}$ = 12 in
Length of timber scupper = $L_{scupper}$ = 48 in
Depth of timber scupper = $d_{scupper}$ = 8 in
Width of timber rail = b_{rail} = 6.75 in
Depth of timber rail = d_{rail} = 13.5 in

Spacing between barrier posts = s_{post} = 6.25 ft = 75 in (maximum)

The timber barrier design is not a part of this design example, but the dimensions are used for weight considerations. Refer to the resources noted in Article 8.5 of this manual for TL-4 crash tested bridge rail details.

[8.4.1.1, 9.9.2]

4. Deck Laminates
Assumed depth of timber deck panel laminates = d_{lam} = 14 in
Assumed width of timber deck panel laminates = b_{lam} = 4 in

[8.4.1.1.2]

Visually-graded longitudinal deck panel lumber is normally supplied rough sawn and surfaced on one side so that panels can be fabricated to the specified dimensions. The nominal dimensions are used for both dead load calculations and section properties for member capacity because the effective net dimensions can be considered the same as the nominal dimensions in the overall finished deck panels. This is true for a longitudinal spike laminated deck with the many individual laminates, if they are made up of rough sawn lumber.

5. Span Lengths
Actual longitudinal length of deck panels, which for an intermediate bridge span is also the distance between the centerlines of adjacent supporting pile caps, are usually in multiples of two feet which is how the lumber is supplied.

L = 22.0 ft

MnDOT uses the effective span, or design span, as center to center of the deck bearing length on each cap.

Because of the end/end deck placement on the pier caps, the intermediate span of the longitudinal deck panels in a multi-span bridge has the longest effective span, L_e.

$$L_e = L - \frac{1}{2} \cdot b_{cap} = 22.0 - \frac{1}{2} \cdot \frac{16}{12} = 21.33 \text{ ft}$$

Figure 8.7.1.1 illustrates the basic layout and dimension used in the design.

6. Unit Weights and Moisture Content

Type of deck panel wood material = Douglas Fir-Larch (No.1)

[Table 3.5.1-1]
[MnDOT Table 3.3.1]
[MnDOT 3.3]

Unit weight of soft-wood = γ_{DFL} = 0.050 kcf
Unit weight of bituminous wearing surface = γ_{ws} = 0.150 kcf

Standard MnDOT practice is to apply a future wearing course of 20 psf.

[8.4.1.1.3]

Moisture content (MC) of timber at the time of installation shall not exceed 19.0%

MnDOT designs for in-service wet-use only which is a MC of greater than 19% for sawn lumber.

7. Douglas Fir-Larch Deck (No. 1) Strength Properties

[Table 8.4.1.1.4-1]

Reference Design Value for flexure = F_{bo} = 1.00 ksi
Reference Design Value for compression perpendicular to the grain = F_{cpo} =0.625 ksi
Modulus of elasticity = E_o = 1700 ksi
Note: F_{cpo} shown for the deck lumber is equal to or less than for the cap, so for the Bearing Strength check, F_{cpo} =0.625 ksi for the deck lumber will be used.

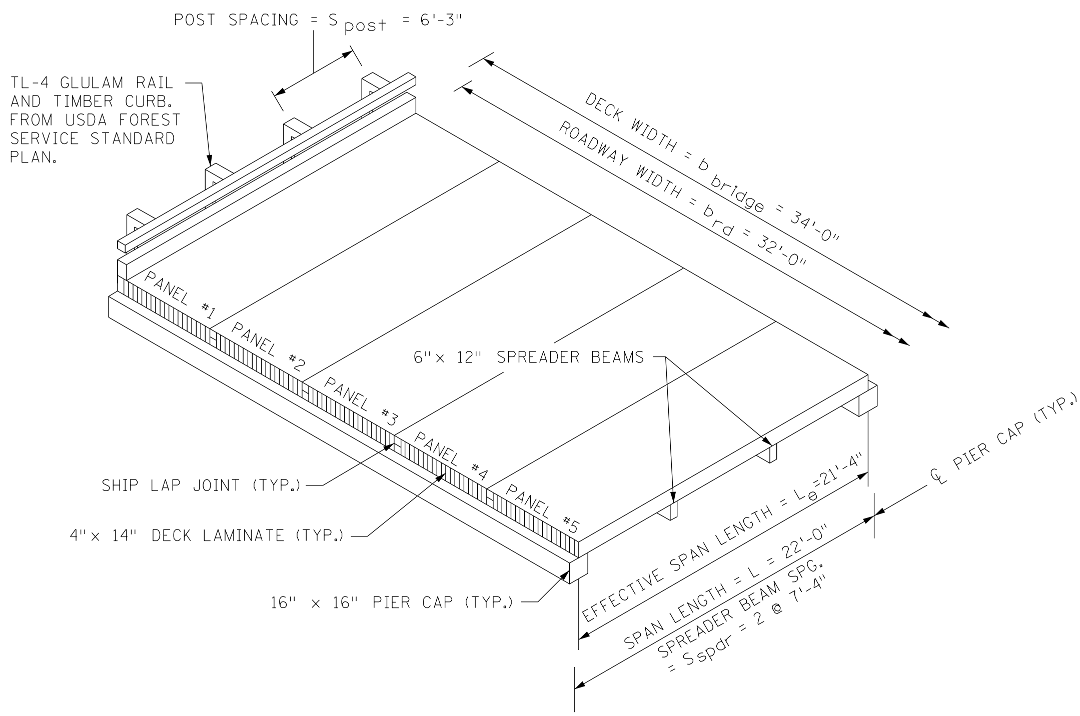


Figure 8.7.1.1 – Longitudinal Timber Deck Layout*

*For clarity, the timber curb/railing on the near side and the bituminous wearing surface are not shown.

Select the Basic Configuration

The bridge deck consists of 5 deck panels that are designed as interconnected, and are oriented parallel to traffic. The laminates of each panel are connected using horizontal spikes. The panels are attached to each other using vertical spikes through ship lap joints, and transverse stiffener beams, also called spreader beams, provide the interconnection per AASHTO LRFD.

The deck panel depth and spreader beam sizes are based on deflection limits as well as strength considerations. The interconnection provided by the spreader beams enable the longitudinal deck panels to act as a single unit under deflection. In addition, each spike laminated deck span is designed as a simply supported member.

A. Deck Panel Widths

The deck panel sizes are given here to clarify the sketches contained throughout this design example.

Width of bridge deck panel #1 = b_1 = 7.33 ft
Width of bridge deck panel #2 = b_2 = 6.33 ft

Width of bridge deck panel #3 = b_3 = 6.67 ft
Width of bridge deck panel #4 = b_4 = 6.33 ft
Width of bridge deck panel #5 = b_5 = 7.33 ft
Overall width of bridge deck = $b_{bridge} = \sum(b_{\#})$ = 34.0 ft

Width of each timber barrier = $b_{barrier}$ = 1.0 ft

Width of roadway = $b_{rd} = b_{bridge} - 2 \cdot b_{barrier} = 34.0 - (2 \cdot 1) = 32.0$ ft

[9.9.6.3]

B. Spreader Beam Dimensions

For interconnection of the deck panels, the spreader beam dimensions that MnDOT uses, based on research (refer to Art. 8.2.3), are as follows:

Width of spreader beams = b_{spdr} = 6 in
Depth of spreader beams = d_{spdr} = 12 in

[9.9.4.3.1]

The size of the spreader beam exceeds the minimum specified in AASHTO LRFD. The spreader beams will be further investigated later in this example.

Determine Dead and Live Load Bending Moments

A. Dead Loads per Unit Strip (1 ft)

The units for the dead load results are given in kips for a 1 ft wide longitudinal strip.

1. Dead Loads per Longitudinal Foot (these units could also be given as kips per square foot).

Weight of deck = $w_{deck} = \gamma_{DFL} \cdot d_{lam} = 0.050 \cdot 14/12 = 0.058$ klf/ft

Weight of wearing surface = $w_{ws} = \gamma_{ws} \cdot d_{ws} = 0.150 \cdot 4/12 = 0.050$ klf/ft

Weight of future wearing course = w_{FWC} = 0.020 klf/ft

2. Determine Linear Weight of Rail System Elements

Volume of timber curb per foot of bridge length = v_{curb}

$v_{curb} = (b_{curb} \cdot d_{curb} \cdot 12 \text{ in/ft}) = (12 \cdot 6 \cdot 12) = 864.0 \text{ in}^3\text{/ft}$

Volume of rail post and spacer block per foot of bridge length = v_{post}

$v_{post} = [b_{post} \cdot L_{post} \cdot d_{post} + b_{spacer} \cdot L_{spacer} \cdot d_{spacer}] / s_{post}$

$v_{post} = [(10 \cdot 8 \cdot 47) + (4.75 \cdot 8 \cdot 13.5)] / 6.25 = 683.7 \text{ in}^3\text{/ft}$

Volume of scupper per foot of bridge length = $v_{scupper}$

$v_{scupper} = (b_{scupper} \cdot L_{scupper} \cdot d_{scupper}) / s_{post}$

$v_{scupper} = (12 \cdot 48 \cdot 8) / 6.25 = 737.3 \text{ in}^3\text{/ft}$

Volume of timber rail per foot of bridge length = v_{rail}

$v_{rail} = (b_{rail} \cdot d_{rail} \cdot 12 \text{ in/ft}) = (6.75 \cdot 13.5 \cdot 12) = 1093.5 \text{ in}^3/\text{ft}$

Volume of timber railing per longitudinal foot of bridge length = $v_{barrier}$

$v_{barrier} = v_{curb} + v_{post} + v_{scupper} + v_{rail}$

$v_{barrier} = 864 + 683.7 + 737.3 + 1093.5 = 3378.5 \text{ in}^3/\text{ft}$

$v_{barrier} = 3378.5/12^3 = 1.955 \text{ ft}^3/\text{ft}$

Total linear weight of combined timber curbs/railings = $w_{barrier}$

$$w_{barrier} = \frac{2 \cdot \gamma_{DFL} \cdot v_{barrier}}{b_{bridge}} = \frac{2 \cdot 0.050 \cdot 1.955}{34.0} = 0.006 \text{ klf}$$

This linear weight result assumes that the curb/railing weight acts uniformly over the entire deck width.

3. Spreader Beam Point Loads on 1 ft Wide Longitudinal Strip

Area of spreader beam = $A_{spdr} = d_{spdr} \cdot b_{spdr} = (12 \cdot 6)/144 = 0.5 \text{ ft}^2$

Spreader beam load = $P_{spdr} = \gamma_{DFL} \cdot A_{spdr} = 0.050 \cdot 0.50 = 0.025$ kips/ft

[AISC 14th p. 3-213]

B. Dead Load Bending Moments per Unit Strip (1 ft)

Maximum bending moment due to deck weight = M_{deck}

$$M_{deck} = \frac{w_{deck} \cdot (L_e)^2}{8} = \frac{0.058 \cdot 21.33^2}{8} = 3.30 \ \frac{\text{kip–ft}}{\text{ft}}$$

Maximum bending moment due to wearing surface weight = M_{ws}

$$M_{ws} = \frac{w_{ws} \cdot (L_e)^2}{8} = \frac{0.050 \cdot 21.33^2}{8} = 2.84 \ \frac{\text{kip–ft}}{\text{ft}}$$

Maximum bending moment due to future wearing surface weight = M_{FWC}

$$M_{FWC} = \frac{w_{FWC} \cdot (L_e)^2}{8} = \frac{0.020 \cdot 21.33^2}{8} = 1.14 \ \frac{\text{kip–ft}}{\text{ft}}$$

Maximum bending moment due to spreader beam weight = M_{spdr}

$$M_{spdr} = \frac{P_{spdr} \cdot L_e}{3} = \frac{0.025 \cdot 21.33}{3} = 0.18 \ \frac{\text{kip–ft}}{\text{ft}}$$

Maximum bending moment due to curb/railing weight = $M_{barrier}$

$$M_{barrier} = \frac{w_{barrier} \cdot (L_e)^2}{8} = \frac{0.006 \cdot 21.33^2}{8} = 0.34 \ \frac{\text{kip–ft}}{\text{ft}}$$

Maximum bending moment due to bridge component dead loads = M_{dc}

$M_{dc} = M_{deck} + M_{spdr} + M_{barrier}$

$M_{dc} = 3.30 + 0.18 + 0.34 = 3.82$ kip·ft/ft

Maximum bending moments due to wearing course loads = M_{dw}

$M_{dw} = M_{ws} + M_{FWC}$

$M_{dw} = 2.84 + 1.14 = 3.98$ kip·ft/ft

[3.6.1.2]

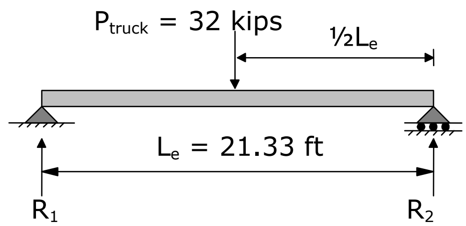
C. Live Load Moments per Lane (12 ft)

The live load bending moment will be calculated per lane (12 ft) and later converted to a per unit strip (1 ft) format.

1. Design Truck Axle Loads

[3.6.1.2.2]

Point load of design truck axle = P_{truck} = 32 kips



[AISC 14th p. 3-215]

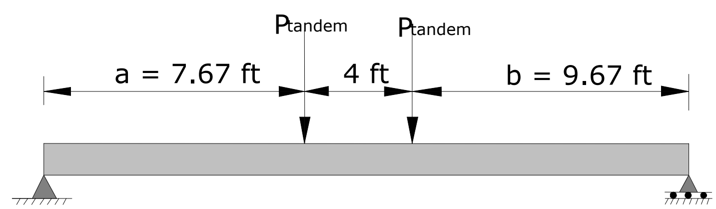
Maximum bending moment due to design truck axle load = M_{truck}

$$M_{truck} = \frac{P_{truck} \cdot L_e}{4} = \frac{32 \cdot 21.33}{4} = 170.64 \ \frac{\text{kip-ft}}{\text{lane}}$$

2. Design Tandem Axle Loads

[3.6.1.2.3]

Point load of design tandem axle = P_{tandem} = 25 kips



[AISC 14th p. 3-228]

Maximum bending moment due to design tandem axle loads = M_{tandem}

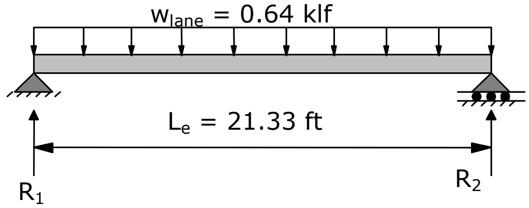
$$M_{tandem} = 12.5 \cdot L_e - 50 + \frac{50}{L_e} = 12.5 \cdot 21.33 - 50 + \frac{50}{21.33} = 218.97 \ \frac{\text{kip-ft}}{\text{lane}}$$

This moment is assumed to occur at the span 0.50 point.

3. Design Lane Loads

[3.6.1.2.4]

Uniform design lane load = w_{lane} = 0.64 klf



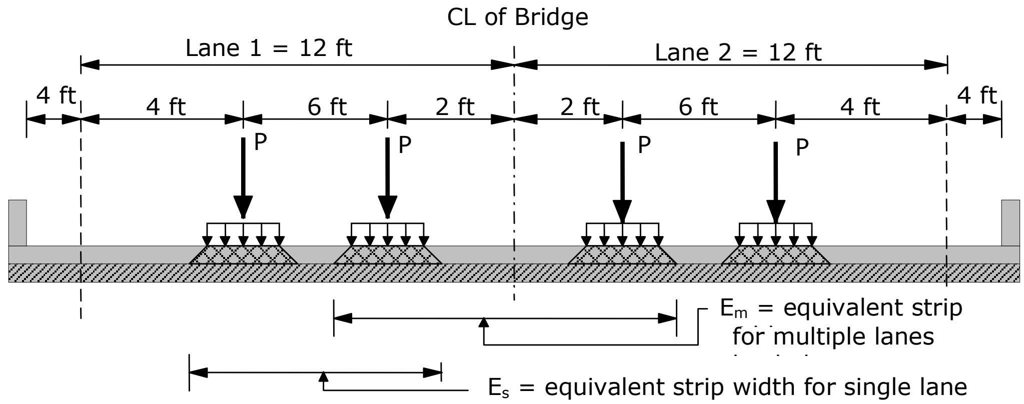
Maximum bending moment due to design lane load = M_{lane}

$$M_{lane} = \frac{w_{lane} \cdot (L_e)^2}{8} = \frac{0.64 \cdot 21.33^2}{8} = 36.40 \ \frac{\text{kip–ft}}{\text{lane}}$$

[4.6.2.3]

D. Live Load Equivalent Lane Strip Width

The live load bending moments, calculated above, will now be distributed over the transverse equivalent lane distance (E_m or E_s).



Physical edge-to-edge bridge deck width = W = b_{bridge} = 34.0 ft

L_e = 21.33 ft $\leq$ 60 ft

Therefore, modified span length = L_1 = L_e = 21.33 ft

[3.6.1.1.1] Number of traffic lanes on the deck = N_L

$$N_L = \frac{b_{rd}}{12 \frac{\text{ft}}{\text{lane}}} = \frac{32}{12} = 2.67 \cong 2 \text{ lanes}$$

1. Single Lane Loaded

[Eqn. 4.6.2.3-1] W = b_{bridge} = 34.0 ft $>$ 30 ft

Therefore, the modified edge-to-edge bridge width for single lane load case = W_1 = 30 ft

Equivalent lane strip width for single lane loaded = E_s

$$E_s = 10 + 5.0 \cdot \sqrt{L_1 \cdot W_1} = 10 + 5 \cdot \sqrt{21.33 \cdot 30} = 136.48 \frac{\text{in}}{\text{lane}} = 11.37 \frac{\text{ft}}{\text{lane}}$$

[Eqn. 4.6.2.3-2]

2. Multiple Lanes Loaded

W = b_{bridge} = 34.0 ft $\leq$ 60 ft

Therefore, the modified edge-to-edge bridge width for multiple lanes loaded case = W_1 = 34.0 ft.

Equivalent lane strip width for multiple lanes loaded = E_m = lesser of

$$E_m = 12 \cdot \frac{W}{N_L} = 12 \cdot \frac{34.0}{2} = 204.0 \ \frac{in}{lane} = 17.0 \ \frac{ft}{lane}$$

OR

$$E_m = 84 + 1.44 \cdot \sqrt{L_1 \cdot W_1} = 84 + 1.44 \cdot \sqrt{21.33 \cdot 34} = 122.78 \frac{in}{lane} = 10.23 \ \frac{ft}{lane}$$

Use E_m = 122.78 in/lane = 10.23 ft/lane

E. Modification of Live Load Bending Moments

[3.6.1.1.2, 4.6.2.3]

1. Multiple Presence Factors

The multiple presence factors cannot be used in conjunction with the equivalent lane strip widths of Article 4.6.2.3. The multiple presence factors have already been included in these equations.

[C3.6.1.1.2]

This design example is for an unspecified ADTT, although as stated in Article 8.2.1 of this manual, AASHTO LRFD recommends limitations on the use of wood deck types based on ADTT. If these recommendations are adhered to, AASHTO LRFD also allows reduction of force effects based on ADTT because the multiple presence factors were developed on the basis of an ADTT of 5000 trucks in one direction. A reduction of 5% to 10% may be applied if the ADTT is expected to be below specified limits during the life of the bridge. If the ADTT level is confirmed, the reduction may be applied subject to the judgment of the designer and approved by the State Bridge Design Engineer.

2. Convert Live Load Bending Moments to per Unit Strip

a. Single Lane Loaded Case

E_s = 11.37 ft/lane

Maximum moment from one lane of design truck loads = $M_{truck(s)}$

$$M_{truck(s)} = M_{truck} \cdot \frac{1}{E_s} = 170.64 \cdot \frac{1}{11.37} = 15.01 \ \frac{kip\text{–}ft}{ft}$$

Maximum moment from one lane of design tandem loads = $M_{tandem(s)}$

$$M_{tandem(s)} = M_{tandem} \cdot \frac{1}{E_s} = 218.97 \cdot \frac{1}{11.37} = 19.26 \ \frac{kip\text{–}ft}{ft}$$

Maximum moment from one design lane load case = $M_{lane(s)}$

$$M_{lane(s)} = M_{lane} \cdot \frac{1}{E_s} = 36.4 \cdot \frac{1}{11.37} = 3.20 \ \frac{kip\text{–}ft}{ft}$$

b. Multiple Lanes Loaded Case

E_m = 10.23 ft/lane

Maximum moment from two lanes of design truck loads = $M_{truck(m)}$

$$M_{truck(m)} = M_{truck} \cdot \frac{1}{E_m} = 170.64 \cdot \frac{1}{10.23} = 16.68 \ \frac{\text{kip-ft}}{\text{ft}}$$

Maximum moment from two lanes of design tandem loads = $M_{tandem(m)}$

$$M_{tandem(m)} = M_{tandem} \cdot \frac{1}{E_m} = 218.97 \cdot \frac{1}{10.23} = 21.40 \ \frac{\text{kip-ft}}{\text{ft}}$$

Maximum moment from two design lane loads = $M_{lane(m)}$

$$M_{lane(m)} = M_{lane} \cdot \frac{1}{E_m} = 36.4 \cdot \frac{1}{10.23} = 3.56 \ \frac{\text{kip-ft}}{\text{ft}}$$

F. Summary of Unfactored Dead and Live Load Bending Moments for a Unit Strip (1 ft) of Deck

Table 8.7.1.1 - Applied Bending Moments

Unfactored Load Case	Maximum Positive Bending Moment (kip·ft / ft)
Dead Loads	
Bridge Components (M_{dc})	3.82
Bridge Wearing Surface (M_{dw})	3.98
Live Loads (Single Lane Loaded)	
Design Truck	15.01
Design Tandem	19.26
Design Lane	3.20
Live Loads (Two Lanes Loaded)	
Design Truck	16.68
Design Tandem	21.40
Design Lane	3.56

G. Factored Bending Moment per Unit Strip (1 ft)

1. Load Modifiers

Standard MnDOT load modifiers are summarized in Table 3.2.1. of this manual.

For timber bridges η_D = 1.0. MnDOT considers spike laminated decks to have a conventional level of redundancy and uses η_R = 1.0. This example bridge is assumed to have a design ADT of over 500 for η_I = 1.0.

[1.3.2] Therefore, importance, redundancy, and ductility factors = η = 1.0

2. Strength I Limit State Load Factors

[3.4.1] Use the Strength I Limit State to determine the required resistance for the deck panels.

[3.6.2.3] Impact factor need not be applied to wood components.

[4.6.2.3] Skew factor (bridge is not skewed) = r = 1.0

Specific Strength I Limit State load factors are found in AASHTO Tables 3.4.1-1 and 3.4.1-2.

The earlier analysis showed that the tandem axle load controls the bending moment of the deck panels. Additionally, the previous results indicate that the live loads per unit strip are largest for the two lanes loaded case. Therefore, use the two lanes loaded case of the tandem axle loads with the uniform lane load in determining the critical live load bending moment acting on the deck panels.

3. Strength I Limit State Bending Moment per Unit Strip (1 ft)

[Tables 3.4.1-1 and 3.4.1-2] Factored bending moment for two lanes loaded case = $M_{u(m)}$

$$M_{u(m)} = \eta \cdot [1.25 \cdot M_{dc} + 1.50 \cdot M_{dw} + 1.75 \cdot r \cdot [M_{tandem(m)} + M_{lane(m)}]]$$

$$M_{u(m)} = 1.0 \cdot [1.25 \cdot 3.82 + 1.50 \cdot 3.98 + 1.75 \cdot 1.0 \cdot [(21.40 + 3.56)] = 54.43 \ \frac{\text{kip-ft}}{\text{ft}}$$

Check Flexural Resistance of Deck Panel

A. Factored Flexural Resistance

The factored bending moment ($M_{u(m)}$) is the required flexural resistance of the deck that needs to be compared with the actual factored flexural resistance of the deck panel (M_r).

[8.6.2] For a rectangular wood section $M_r = \phi_f \cdot F_b \cdot S_{req} \cdot C_L$.

[8.5.2.2] **1. Resistance Factors**

Flexural resistance factor = ϕ_f = 0.85

Compression perpendicular to grain resistance factor = ϕ_{cperp} = 0.90

[8.6.2] **2. Stability Factor**

Stability factor for sawn dimension lumber in flexure = C_L

Laminated deck planks are fully braced. C_L = 1.0

[8.4.4.4] *[Table 8.4.4.4-1]* **3. Adjustment Factors for Reference Design Value**

Size effect factor for sawn dimension lumber in flexure = C_F

d_{lam} = 14 in

b_{lam} = 4 in

C_F = 1.00

[8.4.4.2]

Format conversion factor for component in flexure = C_{KF}

$C_{KF} = 2.5/\phi = 2.5/0.85 = 2.94$

[8.4.4.3]
[Table 8.4.4.3-1]

Wet service factor for sawn dimension lumber in flexure = C_M

Check $F_{bo} \cdot C_F$: $1.00 \cdot 1.0 = 1.0 \leq 1.15$

$C_M = 1.00$

[8.4.4.7]

Incising factor for dimension lumber in flexure = C_i

Douglas Fir-Larch requires incising for penetration of treatment.

[Table 8.4.4.7-1]

$C_i = 0.80$

[8.4.4.8]
[Table 8.4.4.8-1]

Deck factor for a spike-laminated deck in flexure = C_d

$C_d = 1.15$

[8.4.4.9]
[Table 8.4.4.9-1]

Time effect factor for Strength I Limit State = C_λ

$C_\lambda = 0.80$

[Eqn. 8.4.4.1-1]

Adjusted design value = $F_b = F_{bo} \cdot C_{KF} \cdot C_M \cdot C_F \cdot C_i \cdot C_d \cdot C_\lambda$

$F_b = 1.00 \cdot 2.94 \cdot 1.00 \cdot 1.00 \cdot 0.80 \cdot 1.15 \cdot 0.80 = 2.16$ ksi

4. Required Section Modulus

The section modulus is dependent on the deck panel depth. The section modulus is used in Part B to solve for the deck panel depth.

B. Required Deck Panel Depth

Required deck flexural resistance = $M_{n(req)}$

For the deck panel depth to meet Strength I Limit State, M_r must equal (or exceed) $M_{u(m)}$, where $M_r = \phi M_{n(req)}$. Therefore, set $\phi M_{n(req)} = M_{u(m)}$.

$$M_{n(req)} = \frac{M_{u(m)}}{\phi_f} = \frac{54.43}{0.85} = 64.04 \text{ kip-ft}$$

Required section modulus of one foot of deck width = S_{req}

Required depth of deck laminates (panel) = d_{req}

$$S_{req} = \frac{12 \cdot d_{req}^{\;2}}{6}$$

$M_{n(req)} = F_b \cdot S_{req} \cdot C_L$, with $C_L = 1.0$

Substituting terms gives

$$d_{req} = \sqrt{\frac{6 \cdot M_{n(req)}}{12 \cdot F_b \cdot C_L}} = \sqrt{\frac{6 \cdot 64.04 \cdot 12}{12 \cdot 2.16 \cdot 1.0}} = 13.34 \text{ in} \leq 14.0 \text{ in}$$ <u>OK</u>

The required deck panel depth (13.34 inches) indicates that the originally assumed deck depth (14 inches) can be used. However, it is not uncommon that a deeper section could be required to satisfy the deflection limit, so that is checked next.

Investigate Deflection Requirements
[8.5.1]
[2.5.2.6.2]
[3.6.1.3.2]
[9.9.3.3]

A. Deck Live Load Deflection with Current Deck Parameters

The midspan deflections are estimated with the design truck or 25% of the design truck applied in conjunction with the design lane load.

Deflections are to be calculated using Service I Limit State.

Design for deflections using a per foot width approach. With all design lanes loaded, it is allowed to assume all supporting components deflect equally for straight girder systems. This approach can be used on a spike laminated deck with spreader beams meeting the requirements of AASHTO LRFD.

[2.5.2.6.2]
[C2.5.2.6.2]

In the absence of other criteria, the recommended deflection limit in AASHTO LRFD for wood construction is span/425, which will be used here. The designer and owner should determine if a more restrictive criteria is justified, such as to reduce bituminous wearing course cracking and maintenance.

1. Deck Stiffness

Moment of inertia of one foot width of deck panels = I_{prov}

$$I_{prov} = \frac{1}{12} \cdot b \cdot d_{lam}^{3} = \frac{1}{12} \cdot 12 \cdot (14)^3 = 2744 \text{ in}^4$$

Adjusted deck panel modulus of elasticity = E

[8.4.4.3]
[Table 8.4.4.3-1]

Wet service factor, modulus of elasticity of sawn dimension lumber = C_M

$C_M = 0.90$

[8.4.4.7]
[Table 8.4.4.7-1]

Incising factor, modulus of elasticity of sawn dimension lumber = C_i

Douglas Fir-Larch requires incising for penetration of treatment.

$C_i = 0.95$

[Eqn. 8.4.4.1-6]

Adjusted design value = $E = E_o \cdot C_M \cdot C_i$

$E = 1700 \text{ ksi} \cdot 0.90 \cdot 0.95 = 1453.5 \text{ ksi}$

2. Loads per Unit Strip Width (1 ft)

Design truck load used for deflection calculations = $P_{\Delta truck}$

$P_{\Delta truck}$ = [2 lanes of load] / b_{bridge}

$P_{\Delta truck}$ = [2 · 32 kips] / 34.0 ft = 1.882 kips/ft

Design lane load used for deflection calculations = $w_{\Delta lane}$

$w_{\Delta lane}$ = 2 lanes of load / b_{bridge} = 2 · 0.64 klf / 34.0 ft
= 0.038 klf/ft

[3.6.1.3.2]
[AISC 14th p. 3-213, 3-215]

3. Live Load Deflection Calculations

Deflection at deck midspan due to the design truck load = Δ_{truck}

$$\Delta_{truck} = \frac{P_{truck} \cdot L_e^3}{48 \cdot E \cdot I_{prov}} = \frac{1.882 \cdot (21.33 \cdot 12)^3}{48 \cdot 1453.5 \cdot 2744} = 0.16 \text{ in}$$

Deflection at deck midspan due to the design lane load = Δ_{lane}

$$\Delta_{lane} = \frac{5 \cdot w_{lane} \cdot L_e^4}{384 \cdot E \cdot I_{prov}} = \frac{5 \cdot \frac{0.038}{12} \cdot (21.33 \cdot 12)^4}{384 \cdot 1453.5 \cdot 2744} = 0.04 \text{ in}$$

Deflection at deck midspan due to a combination of truck (25%) and design lane loads = $\Delta_{combined}$

$\Delta_{combined} = 0.25 \cdot \Delta_{truck} + \Delta_{lane} = (0.25 \cdot 0.16) + 0.04$

$\Delta_{combined} = 0.08 \text{ in} \le \Delta_{truck} = 0.16 \text{ in}$

Therefore, the maximum deflection between the combination load deflection and the truck load deflection = $\Delta = \Delta_{truck}$ = 0.16 in.

[2.5.2.6.2]

Live load deflection limit at deck midspan = Δ_{max}

$\Delta_{max} = L_e / 425 = 21.33/ 425 = 0.0502 \text{ ft} = 0.60 \text{ in}$

$\Delta = 0.16 \text{ in} \le \Delta_{max} = 0.60 \text{ in}$ <u>OK</u>

The initial 14-inch deck panel depth and grade are adequate for deflection.

Check Shear Resistance Of Deck Panel [8.7, 9.9.3.2]

In longitudinal decks, maximum shear shall be computed in accordance with the provisions of AASHTO LRFD Article 8.7. For this example, shear loading is not close to governing the design of the deck panel and so the calculation is not shown here. Shear check for a transverse deck is shown in the glulam beam with transverse deck design example (Article 8.7.4).

Investigate Spreader Beam Requirements [9.9.6.3] [9.9.4.3]

A. Spreader Beam Parameters

A spreader beam is required to satisfy the AASHTO definition of interconnected spike laminated panels.

Maximum spreader beam spacing = s_{max} = 8.0 ft

Actual longitudinal spreader beam spacing = s_{spdr} = L / 3
= 22 / 3 = 7.33 ft

s_{spdr} = 7.33 ft ≤ s_{max} = 8.0 ft OK

Minimum allowed rigidity of the spreader beams = EI_{min} = 80,000 kip·in^2

The spreader beams shall be attached to each deck panel near the panel edges and at intervals less than or equal to 15 inches. The spreader beams also reduce the relative panel deflection, thus aiding to decrease wearing surface cracking. If bituminous maintenance is a concern, exceeding the minimum criteria for spacing (adding more spreader beams) may increase wearing surface expected life.

Required moment of inertia of spreader beams to accommodate the specified rigidity for a given species and grade of wood = I_{min}.

For Douglas Fir-Larch No. 1 Beams & Stringers (B & S), E_o =1600 ksi

Adjusted spreader beam modulus of elasticity = E

[8.4.4.3]
[Table 8.4.4.3-1]

Wet service factor for modulus of elasticity of B & S timber = C_M

For nominal thickness > 4.0 in, C_M = 1.0

[Eqn. 8.4.4.1-6]

Adjusted design value = E = $E_o \cdot C_M$

E = 1600 · 1.0 = 1600 ksi

$$I_{min} = \frac{80{,}000}{E} = \frac{80{,}000}{1600} = 50.0 \text{ in}^4$$

Find required depth of spreader beam = d_{min}

$$I_{min} = \frac{1}{12} \cdot b_{spdr} \cdot d_{min}^3$$

$$d_{min} = \sqrt[3]{\frac{12 \cdot I_{min}}{b_{spdr}}} = \sqrt[3]{\frac{12 \cdot 50.0}{6}} = 4.64 \text{ in} \le d_{spdr} = 12 \text{ in} \qquad \text{(OK)}$$

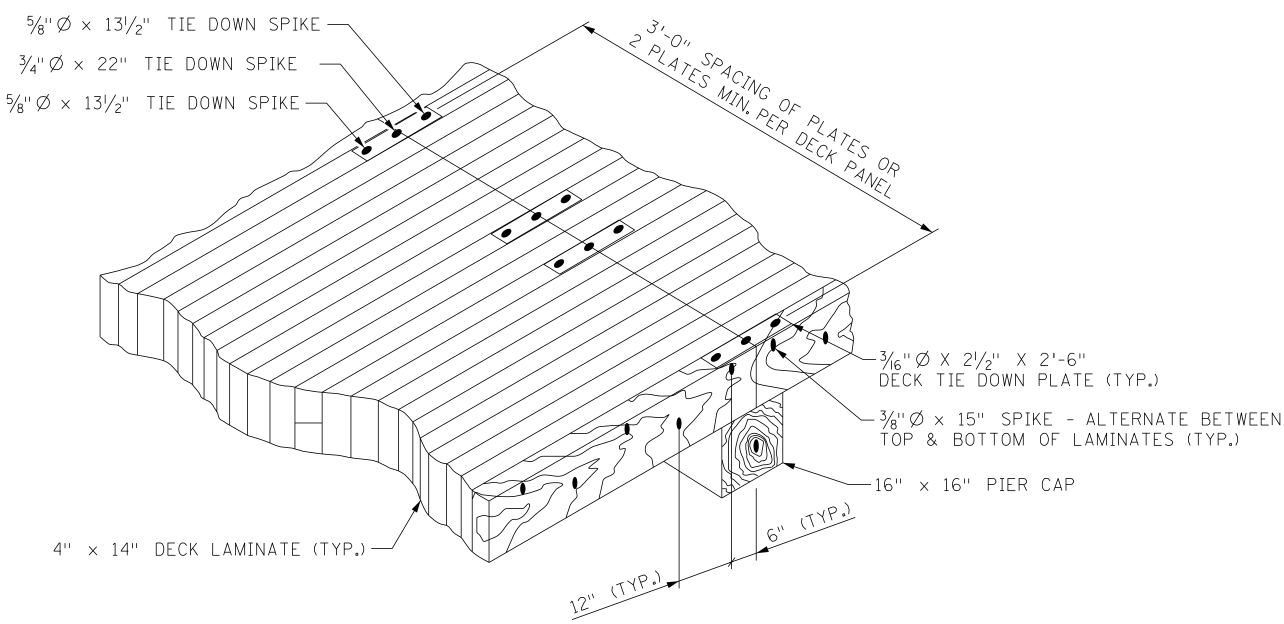
As described in Article 8.2.3 of this manual, MnDOT standard practice is to use 6 in X 12 in spreader beams, which exceed the specified minimum criteria.

[9.9.6.1]

B. Spike Lamination Deck Pattern

Spike-laminated decks shall consist of a series of lumber laminations that are placed edgewise between supports and spiked together on their wide face with deformed spikes of sufficient length to fully penetrate four laminations. The spikes shall be placed in lead holes that are bored through pairs of laminations at each end and at intervals not greater than 12.0 inches in an alternating pattern near the top and bottom of the laminations.

Laminations shall not be butt spliced within their unsupported length.



*Typical each deck Tie-down

Figure 8.7.1.2 – Longitudinal Timber Deck to Cap Connections

[9.9.6.2, 9.9.4.2]

C. Deck Tie-Downs

Typically, MnDOT uses $^{5}/_{8}$ inch diameter spikes to attach the metal tie-down plates (brackets) to the deck panels, and $^{3}/_{4}$ inch diameter spikes are used to connect the plates to the pile cap. The plates are typically $^{3}/_{16}$ inch thick by $2^{1}/_{2}$ inches wide X 2'-6" long. These plates can be spaced at 3 feet maximum intervals transversely over the pile cap as specified for stress laminated decks or a minimum of two plates per deck panel, with the latter being more typical of MnDOT designs.

Investigate Bearing Strength Requirements

A. Maximum Support Reactions per Unit Strip (1 ft)

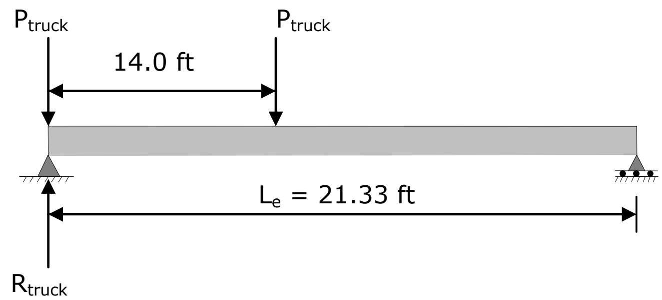
1. Live Load Reactions

The maximum live load reactions need to be calculated. The design truck and tandem axle loads have been oriented to produce the greatest reaction at the pile cap. The design truck, tandem, and lane reactions are assumed to be uniformly distributed over the equivalent live load strip width (E_s or E_m).

a. Multiple Lanes Loaded

The calculations below only consider the multiple lanes loaded case. Because the equivalent lane strip width for multiple lanes is less than that for the single lane loaded case (E_m<E_s), there is more force per transverse foot for the multiple lane load case.

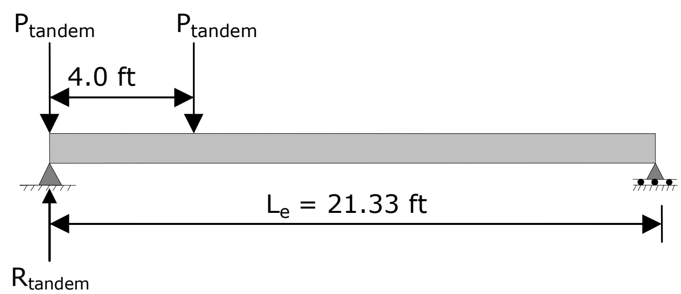
Maximum pile cap reaction due to the design truck loads = R_{truck}



$$R_{truck} = \left[P_{truck} + P_{truck} \cdot \frac{(L_e - 14)}{L_e}\right] \cdot \frac{1}{E_m}$$

$$R_{truck} = \left[32 + 32 \cdot \frac{(21.33 - 14)}{21.33}\right] \cdot \frac{1}{10.23} = 4.202 \ \frac{\text{kips}}{\text{ft}}$$

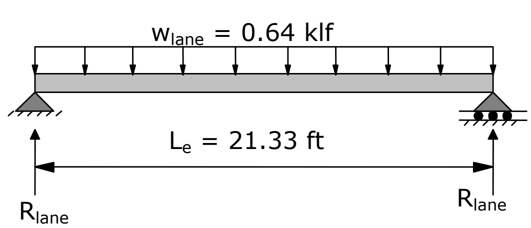
Maximum pile cap reaction due to the design tandem loads = R_{tandem}



$$R_{tandem} = \left[P_{tandem} + P_{tandem} \cdot \frac{(L_e - 4)}{L_e}\right] \cdot \frac{1}{E_m}$$

$$R_{tandem} = \left[25 + 25 \cdot \frac{(21.33 - 4)}{21.33}\right] \cdot \frac{1}{10.23} = 4.429 \frac{\text{kips}}{\text{ft}}$$

Maximum pile cap reaction due to the design lane load = R_{lane}



$$R_{lane} = \left[\frac{w_{lane} \cdot L_e}{2}\right] \cdot \frac{1}{E_m} = \left[\frac{0.64 \cdot 21.33}{2}\right] \cdot \frac{1}{10.23} = 0.667 \ \frac{\text{kips}}{\text{ft}}$$

2. Dead Load Reactions

Maximum reaction on pile cap due to the deck weight = R_{deck}

$$R_{deck} = \frac{w_{deck} \cdot L_e}{2} = \frac{0.058 \cdot 21.33}{2} = 0.622 \ \frac{kip}{ft}$$

Maximum reaction on pile cap due to the wearing surface weight = R_{ws}

$$R_{ws} = \frac{w_{ws} \cdot L_e}{2} = \frac{0.050 \cdot 21.33}{2} = 0.533 \ \frac{kips}{ft}$$

Maximum reaction on cap due to future wearing surface weight = R_{FWC}

$$R_{FWC} = \frac{w_{FWC} \cdot L_e}{2} = \frac{0.020 \cdot 21.33}{2} = 0.213 \ \frac{kips}{ft}$$

Maximum pile cap reaction due to spreader beam = R_{spdr}

R_{spdr}= 0.025 kips/ft

Maximum reaction on pile cap due to the curb/railing weight = $R_{barrier}$

$$R_{barrier} = \frac{w_{barrier} \cdot L_e}{2} = \frac{0.006 \cdot 21.33}{2} = 0.064 \ \frac{kips}{ft}$$

Maximum reaction on pile cap due to the component dead loads = R_{dc}

$R_{dc} = R_{deck} + R_{spdr} + R_{barrier}$

R_{dc} = 0.622 + 0.025 + 0.064 = 0.711 kips/ft

Maximum reaction on pile cap due to the wearing course = R_{dw}

$R_{dw} = R_{ws} + R_{FWC}$

R_{dw} = 0.533 + 0.213 = 0.746 kips/ft

B. Summary of Unfactored Support Reactions

Table 8.7.1.2 – Support Reactions

Unfactored Load Case	**Maximum Support Reaction (kips/ft)**
Dead Loads	
Bridge Components (Rdc)	0.711
Bridge Wearing Surface (R_{dw})	0.746
Live Loads (Two Lanes Loaded)	
Design Truck	4.202*
Design Tandem	4.429*
Design Lane	0.667*

* Kips per transverse foot of the equivalent lane strip (E_m)

C. Strength I Limit State Reaction per Unit Strip (1 ft)

[3.4.1]

Table 8.7.1.2 shows that the design tandem for the two lanes loaded case controls the reaction of the deck panels. Therefore, the design truck will be neglected for bearing calculations.

Maximum factored reaction when multiple lanes are loaded = $R_{u(m)}$

[Tables 3.4.1-1 and 3.4.1-2]

$$R_{u(m)} = \eta \cdot [1.25 \cdot R_{dc} + 1.50 \cdot R_{dw} + 1.75 \cdot r \cdot (R_{tandem(m)} + R_{lane(m)})]$$

$$R_{u(m)} = 1.0 \cdot [1.25 \cdot 0.711 + 1.50 \cdot 0.746 + 1.75 \cdot 1.0 \cdot (4.429 + 0.667)] = 10.926 \ \frac{kips}{ft}$$

D. Factored Bearing Resistance

[8.8.3]

[Eqns. 8.8.1-1, 8.8.3-1]

The factored resistance (P_r) of a component in compression perpendicular to grain shall be taken as $P_r = \phi_{cperp} \cdot F_{cp} \cdot A_b \cdot C_b$

1. Bearing Area

Width of bearing = b_b = 1 ft = 12 in (for unit strip)

Length of bearing = $L_b = ½ \cdot b_{cap} = ½ \cdot 16 = 8$ in

Provided bearing area = $A_b = b_b \cdot L_b = 12 \cdot 8 = 96$ in^2

[Table 8.8.3-1]

2. Bearing Adjustment Factor

$L_b = 8 \text{ in} \geq 6 \text{ in}$

$C_b = 1.0$

3. Adjustment Factors for Reference Design Value

[8.4.4.2]

Format conversion factor for compression perpendicular to grain = C_{KF}

$C_{KF} = 2.1/\phi_{cperp} = 2.1/0.90 = 2.33$

[8.4.4.3]
[Table 8.4.4.3-1]

Wet Service factor for sawn dimension lumber = C_M

$C_M = 0.67$

[8.4.4.7]
[Table 8.4.4.7-1]

Incising Factor for sawn dimension lumber in compression perpendicular to grain = C_i

$C_i = 1.00$

[8.4.4.9]
[Table 8.4.4.9-1]

Time effect factor for Strength I limit state = C_λ

$C_\lambda = 0.80$

Adjusted design value in compression perpendicular to grain = F_{cp}

[Eqn. 8.4.4.1-5]

$F_{cp} = F_{cpo} \cdot C_{KF} \cdot C_M \cdot C_i \cdot C_\lambda = 0.625 \cdot 2.33 \cdot 0.67 \cdot 1.00 \cdot 0.80$

$F_{cp} = 0.781$ ksi

4. Bearing Resistance Calculation Check

Nominal resistance of deck in compression perp. to the grain = P_n

$P_n = F_{cp} \cdot A_b \cdot C_b = 0.781 \cdot 96 \cdot 1.0 = 75.0$ kips/ft

Per foot of width of bearing, the factored resistance of deck in compression perp. to the grain = $P_r = \phi P_n$

$\phi P_n = \phi_{cperp} \cdot P_n = 0.90 \cdot 75.0$ kips/ft = 67.5 kips/ft

$\phi P_n = 67.5$ kips/ft $\geq R_{u(m)} = 10.9$ kips/ft OK

There is no need to attach a sill component to the cap for extending the bearing because the given bearing strength is more than adequate.

Summary of Connection Design

Figure 8.7.1.3 below indicates the position of the spreader beam connections, the ship lap joints (deck panel-to-deck panel connections), and deck panel-to-pile cap tie-down plates. For connections not specified in AASHTO, or for the use of connections that are not in accordance with AASHTO, State Bridge Design Engineer approval is needed.

[9.9.4.3]

The maximum spacing of the spreader beam connection bolts is 15 inches, and they shall be placed near the panel edges.

[8.4.2]

Minimum fastener and hardware requirements are specified in Section 8 of AASHTO LRFD.

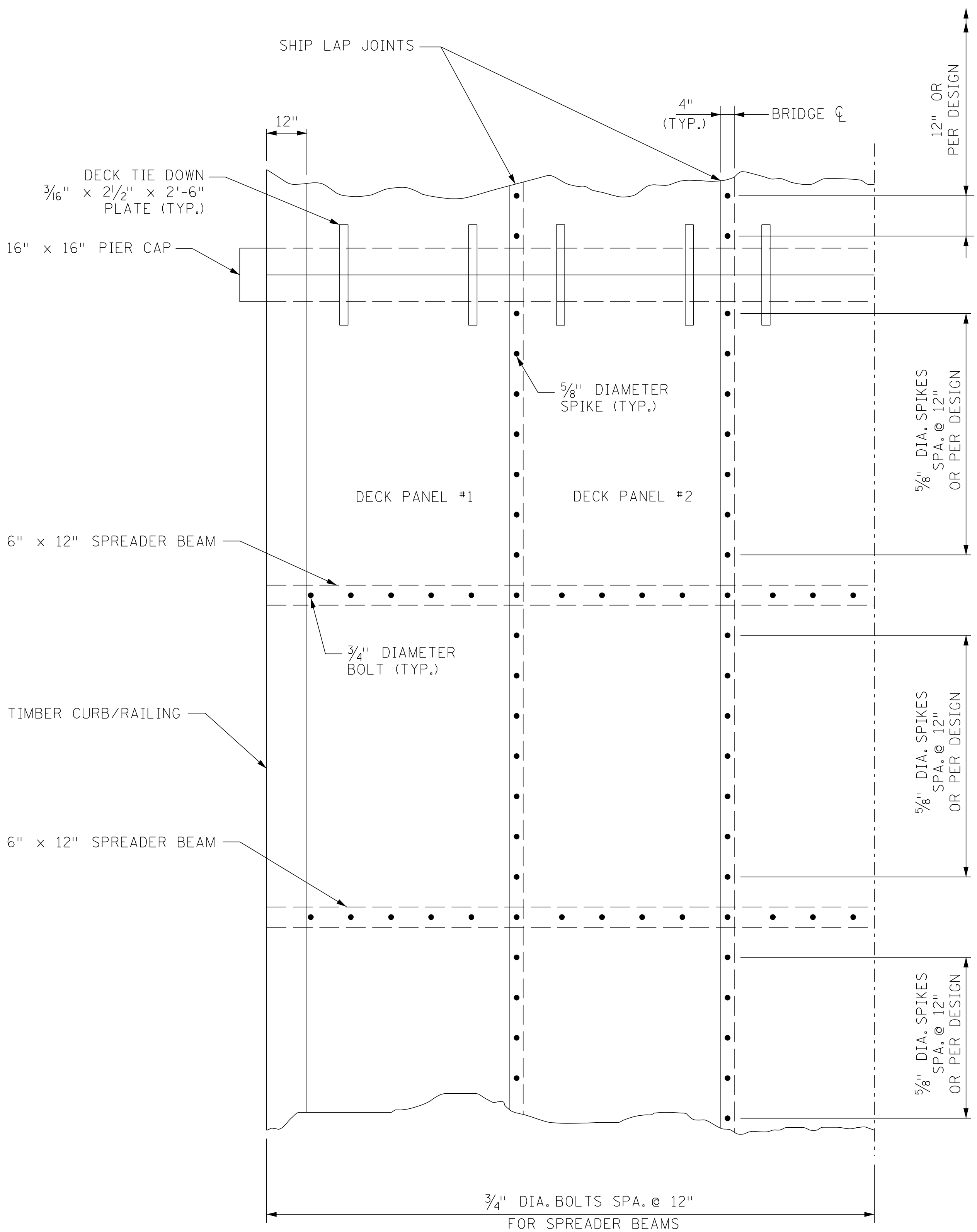


Figure 8.7.1.3 – Longitudinal Timber Deck Partial Plan View

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8.7.2 Timber Pile Cap Design Example

This example demonstrates the design of a typical timber pile cap, which accompanies the Longitudinal Spike Laminated Timber Deck design example in Article 8.7.1. The caps provide bearing support of the longitudinal deck for an intermediate bridge span as previously designed. The bridge contains no longitudinal girders; the dead and live loads are distributed loads along the pile cap. These types of bridges are usually reserved for secondary roads with low truck traffic volumes.

A. Material and Design Parameters

[Figure 8.3-1] The dimension annotations used throughout this design example are as follows. The vertical dimension of a member is considered its depth. The transverse and longitudinal dimensions of a member are considered its width and length, respectively. These dimension annotations are consistent with Figure 8.3-1 of the *2014 AASHTO LRFD Bridge Design Specifications*, except for sawn lumber descriptive names. The letter notations will be used in this example (b, d, etc.).

[8.4.1.1] **1. Pile Cap**

Initial timber pile cap width = b_{cap} = 16 in = 1.33 ft

Initial timber pile cap depth = d_{cap} = 16 in = 1.33 ft

[8.4.1.1.2] The largest size commonly available for visually-graded Posts and Timbers sawn lumber is 16 in X 16 in. Availability of lengths over 36 feet can possibly be limited, and may require a splice. This example does not require a splice. As stated earlier in Article 8.7.1, the dimensions for the rough sawn caps are used as actual.

[9.9.8] **2. Wearing Course**

Depth of wearing course = d_{ws} = 4 in, which is the average depth taken from the Longitudinal Spike Laminated Timber Deck design example in Article 8.7.1.

3. Curb and Railing (TL-4 Glulam Timber Rail with Curb)

Curb and railing components are itemized in the Longitudinal Spike Laminated Timber Deck design example.

The timber barrier design is not a part of this design example.

The maximum spacing for the timber rail posts is 6.25 ft.

[8.4.1.1]
[9.9.2] **4. Deck Laminates**

Depth of timber deck panel laminates = d_{lam} = 14 in = 1.167 ft

Width of timber deck panel laminates = b_{lam} = 4 in = 0.333 ft

[8.4.1.1.2] Deck panel lumber is designed in Article 8.7.1.

5. Piles

Diameter of circular steel shell piles = d_{pile} = 16 in
Number of piles = n_{piles} = 5

It is standard MnDOT practice to use equally spaced 16 inch diameter piles for the pile bent piers. Refer to Article 8.4 of this manual for further description.

6. Cap Span Lengths

Overall transverse length of pile caps = L_{trans} = 36 ft
Transverse combined width of deck panels = b_{bridge} = 34.0 ft
Longitudinal distance between pile cap centerlines = L = 22 ft
Transverse distance between centerlines of piles = L_{cap} = 8.17 ft
Transverse clear distance between adjacent piles = L_{clr} = 6.83 ft

The pile cap is not spliced for this design example. When a pile cap is spliced, the splice should be over an interior pile. Refer to Figure 8.7.2.1 below for pile locations. Adjacent spans are L = 22 ft for this example.

7. Unit Weights and Moisture Content

Type of pile cap wood material = Douglas Fir-Larch Posts and Timbers (No. 1)

[Table 3.5.1-1] Unit weight of soft wood (Douglas Fir-Larch) = γ_{DFL} = 0.050 kcf
[MnDOT Table 3.3.1] Unit weight of bituminous wearing course = γ_{ws} = 0.150 kcf
[MnDOT 3.3] Standard MnDOT practice is to apply a future wearing course of 20 psf.

[8.4.1.1.3] Moisture content of timber (MC) at the time of installation shall not exceed 19.0%. MnDOT designs for in service wet-use only, which is a MC of greater than 19% for sawn timber.

8. Douglas Fir-Larch Posts and Timbers (No. 1) Strength Properties

[Table 8.4.1.1.4-1] Reference Design Value of wood in flexure = F_{bo} = 1.20 ksi
Reference Design Value of wood in horizontal shear = F_{vo} = 0.17 ksi
Reference Design Value of wood in compression perpendicular to grain = F_{cpo} = 0.625 ksi

Modulus of elasticity = E_o = 1600 ksi

Select the Basic Configuration

The bridge deck consists of 5 interconnected longitudinal deck panels. The deck panels are supported by timber pile caps, which extend the width of the bridge at the piers. See the timber deck example in Article 8.7.1 for details regarding the deck design and connection configurations.

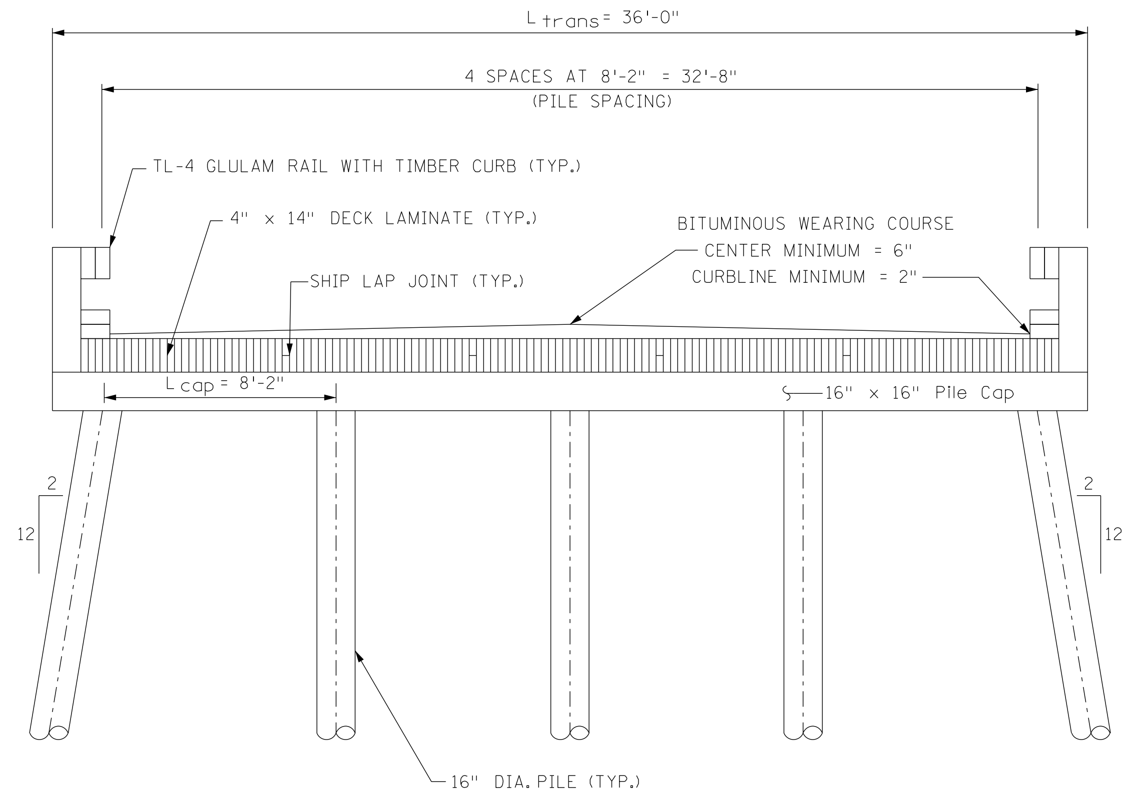


Figure 8.7.2.1 – Longitudinal Timber Deck on Pier Timber Cap

Determine Dead and Live Load Reactions, Shear Forces, and Bending Moments

A. Determine Dead Loads

Dead load units are given in kips per linear foot along the pile cap.

Area of pile cap cross section = A_{cap}

$$A_{cap} = d_{cap} \cdot b_{cap} = 16 \cdot 16 = 256 \text{ in}^2 = 1.78 \text{ ft}^2$$

Linear weight of timber pile cap = w_{cap}

$$w_{cap} = \gamma_{DFL} \cdot A_{cap} = 0.050 \cdot 1.78 = 0.089 \text{ kips/ft}$$

Linear weight of deck panels = w_{deck}

$$w_{deck} = \gamma_{DFL} \cdot d_{plank} \cdot L = 0.050 \cdot 1.167 \cdot 22 = 1.283 \text{ kips/ft}$$

Area of spreader beam = A_{spdr}

$$A_{spdr} = d_{spdr} \cdot b_{spdr} = 12 \cdot 6 = 72 \text{ in}^2 = 0.5 \text{ ft}^2$$

Linear weight of spreader beams = w_{spdr}

$$w_{spdr} = 2 \cdot A_{spdr} \cdot \gamma_{DFL} = 2 \cdot 0.5 \cdot 0.050 = 0.050 \text{ kips/ft}$$

Volume of curb/railing components per longitudinal foot of bridge length = $v_{barrier}$ = 1.955 ft^3/ft (from previous example)

Weight of timber barrier per longitudinal foot of bridge length = $w_{barrier}$

$$w_{barrier} = \left[\frac{(2 \cdot \gamma_{DFL} \cdot \upsilon_{barrier})}{b_{bridge}}\right] \cdot L = \left[\frac{(2 \cdot 0.050 \cdot 1.955}{34.0}\right] \cdot 22 = 0.127 \ \frac{\text{kips}}{\text{ft}}$$

This linear load assumes that the barrier weight acts uniformly over the entire deck width.

Linear weight of bituminous wearing course = w_{ws}

$$w_{ws} = \gamma_{ws} \cdot d_{ws} \cdot L = 0.150 \cdot 4.0 \cdot (1/12) \cdot 22 = 1.100 \text{ kips/ft}$$

Linear weight of future wearing course = w_{FWC}

$$w_{FWC} = 0.020 \cdot L = 0.020 \cdot 22 = 0.440 \text{ kips/ft}$$

Total linear dead load of components acting along the pile cap = w_{dc}

$$w_{dc} = w_{cap} + w_{deck} + w_{spdr} + w_{barrier}$$

$$w_{dc} = 0.089 + 1.283 + 0.050 + 0.127 = 1.549 \ \frac{\text{kips}}{\text{ft}}$$

Linear dead load of wearing course acting along the pile cap = w_{dw}

$$w_{dw} = w_{ws} + w_{FWC}$$

$$w_{dw} = 1.100 + 0.440 = 1.540 \ \frac{\text{kips}}{\text{ft}}$$

B. Cap Spans and Structural Analysis Models

The pile cap is made up of a four span continuous beam. For simplification, conservative modeling assumptions can be made.

1. Analysis Models

In determining the maximum member forces, MnDOT uses a variation of beam models as follows:

1) The maximum shear forces and reactions are determined by modeling the pile cap as a continuous beam on pinned supports. Moving live loads are then placed at various locations along the span, to produce the maximum shear and reactions. This method of analysis allows the effects of adjacent spans to be investigated.
2) The maximum positive bending moments (tension on pile cap bottom) are determined by considering the pile cap as a single simply-supported span between piles.

3) The maximum negative bending moments (tension on pile cap top) are determined by considering the pile cap as a single fixed-fixed span between piles, with fixed ends.

The dead and live load shear, reactions, and bending moment results can be determined using a basic structural analysis computer program, or using the standard beam formulas found in AISC 14th Edition LRFD Manual. The results are summarized in Table 8.7.2.1. The HL-93 reactions for the longitudinal deck are based on Table 3.4.1.1 of this manual in Section 3, for simplicity (except for the lane load). However, for longer spans, the adjacent spans need to be considered in figuring the truck reaction because the third axle will have an increased load effect.

[3.6.1.3]

Both the design lanes and 10.0 ft loaded width in each lane shall be positioned to produce extreme force effects. For this timber slab span, the live load is distributed over the equivalent strip widths for a single lane case or multiple lanes case that were calculated in Article 8.7.1. Only one span on the cap and approximately one third of the adjacent span for the single lane case is loaded and so will not control the design of the cap.

For the two lane case the design lanes are side by side, one on each side of the center pile. The loaded width in both design lanes is placed adjacent to the inside of the design lane above the center pile. This position of the design lanes and loaded width will create the largest force effects in the cap. To simplify the calculations of the maximum reactions and shears, it is conservatively assumed that only the two adjacent cap spans are loaded with the distributed live load.

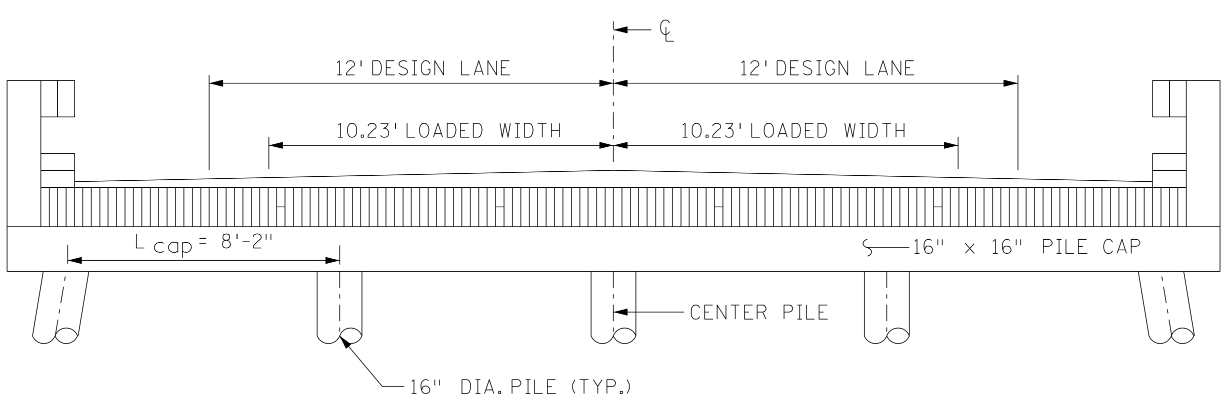


Figure 8.7.2.2 – Live Load Position for Cap Analysis

C. Summary of Maximum Shear Force, Reaction and Bending Moment Results

Table 8.7.2.1

Unfactored Load Case	Maximum Positive Bending Moment (kip·ft)	Maximum Negative Bending Moment (kip·ft)	Maximum Shear Force (kips)	Maximum Support Reaction (kips)
Component Dead Load (DC)	12.92	8.62	7.91	15.82
Wearing Course Dead Load (DW)	12.85	8.57	7.86	15.73
Multiple Lanes Loaded				
Design Truck	35.59	23.73	21.78	43.57
Design Tandem	37.07	24.71	22.69	45.37
Design Lane	11.48	7.66	7.03	14.06

D. Factored Bending Moment in Cap

1. Load Modifiers

Basis for the load modifiers is similar to example 8.7.1.

[1.3.2] Importance, redundancy, and ductility factors = η = 1.0

2. Strength I Limit State Load Factors

Use the Strength I Limit State to determine the required flexural resistance of the pile cap.

[3.6.2.3] Impact factor need not be applied to wood components.

[4.6.2.3] Skew factor (bridge is not skewed) = r = 1.0

Specific Strength I Limit State Load Factors are found in AASHTO Tables 3.4.1-1 and 3.4.1-2.

The above results (Table 8.7.2.1) indicate that multiple lanes loaded with the design tandem and lane loads control for flexure.

3. Strength I Limit State Positive Moment

Positive (tension on pile cap bottom) factored bending moment due to multiple lanes loaded case = $M_{u(m)}$

[Tables 3.4.1-1 and 3.4.1-2]

$$M_{u(m)} = \eta \cdot [1.25 \cdot M_{dc} + 1.50 \cdot M_{dw} + 1.75 \cdot r \cdot (M_{tandem(m)} + M_{lane(m)})]$$

$$M_{u(m)} = 1.0 \cdot [1.25 \cdot (12.92) + 1.50 \cdot (12.85) + 1.75 \cdot 1.0 \cdot (37.07 + 11.48)]$$

$$M_{u(m)} = 120.39 \ \text{kip} - \text{ft}$$

Check Flexural Resistance of Cap

A. Factored Flexural Resistance

The factored bending moment ($M_{u(m)}$) is the required flexural resistance of the cap that needs to be compared with the actual factored flexural resistance of the cap (M_r).

[8.6.2]

For a rectangular wood section $M_r = \phi_f \cdot F_b \cdot S_{prov} \cdot C_L = M_{r(prov)}$

Because caps are supplied in standard sizes and the dimensions are known, M_r is calculated as $M_{r(prov)}$.

[8.5.2.2]

1. Resistance Factor

Flexural resistance factor = ϕ_f = 0.85

2. Section Modulus

The section modulus is dependent on the cap size. The provided section modulus for the initial cap size is:

Provided pile cap section modulus = $S_{prov} = \dfrac{b_{cap} \cdot d_{cap}^{\ 2}}{6}$

$$S_{prov} = \frac{16 \cdot 16^2}{6} = 682.67 \ \text{in}^3$$

3. Stability Factor

[8.6.2]

Stability factor for rectangular lumber in flexure = C_L

For flexural components where depth does not exceed the width of the component, C_L = 1.0.

4. Adjustment Factors for Reference Design Values

[8.4.4.4]

Size effect factor for sawn beam lumber in flexure = C_F.

For d_{cap} > 12.0 in

[Eqn. 8-4.4.4-2]

$C_F = (12/d_{cap})^{1/9} = 0.97$

[8.4.4.2]

Format conversion factor = C_{KF}

$C_{KF} = 2.5/\phi = 2.5/0.85 = 2.94$

[8.4.4.3]

Wet Service factor for Posts and Timbers sawn lumber = C_M

For nominal thickness greater than 4.0 in, C_M = 1.0.

[8.4.4.9]
[Table 8.4.4.9-1]

Time Effect Factor = C_λ

$C_\lambda = 0.80$

[Eqn. 8.4.4.1-1]

Adjusted design value = $F_b = F_{bo} \cdot C_{KF} \cdot C_M \cdot C_F \cdot C_\lambda$

$F_b = 1.20 \cdot 2.94 \cdot 1.00 \cdot 0.97 \cdot 0.80 = 2.74$ ksi

B. Pile Cap Flexural Check

Required pile cap flexural resistance = $M_{u(m)}$

For the cap to meet Strength I Limit State, $M_{r(prov)}$ must equal or exceed $M_{u(m)}$. As determined previously, $M_{u(m)} = 120.39$ kip-ft

Provided pile cap factored flexural resistance:

[Eqn. 8.6.1-1]

$M_{r(prov)} = \phi_f \cdot F_b \cdot S_{prov} \cdot C_L = 0.85 \cdot 2.74 \cdot 682.67 \cdot 1.0$

$= 1589.94$ kip·in $= 132.49$ kip·ft

$M_{r(prov)} = 132.49$ kip·ft $\geq M_{u(m)} = 120.39$ kip·ft <u>OK</u>

Investigate Shear Resistance Requirements [8.7]

A. Critical Shear Force Location

Horizontal shear must be checked for wood components. The term "horizontal" shear is typically used in wood design, because a shear failure initiates along the grain. This shear failure is typically along the horizontal axis. The shear stress is equal in magnitude in the vertical direction, but inherent vertical resistance is greater, and so typically does not need to be designed for. AASHTO LRFD C8.7 provides commentary on this.

For components under shear, shear shall be investigated at a distance away from the face of the support equal to the depth of the component. When calculating the maximum design shear, the live load shall be placed so as to produce the maximum shear at a distance from the support equal to the lesser of either three times the depth of the component (d_{cap}) or one-quarter of the span (L_{cap}). This placement of the live load is more applicable when it is applied as axle point loads on longitudinal members, rather than the transverse distributed loads used in this example.

Location to check for shear = $(d_{cap} + {}^1/_2 \cdot d_{pile}) / L_{cap}$

$= (1.33 \text{ ft} + {}^1/_2 \cdot 1.33 \text{ ft}) / 8.17 \text{ ft}$

Check for shear at about 24% of span length away from the support centerlines, or 2.00 ft

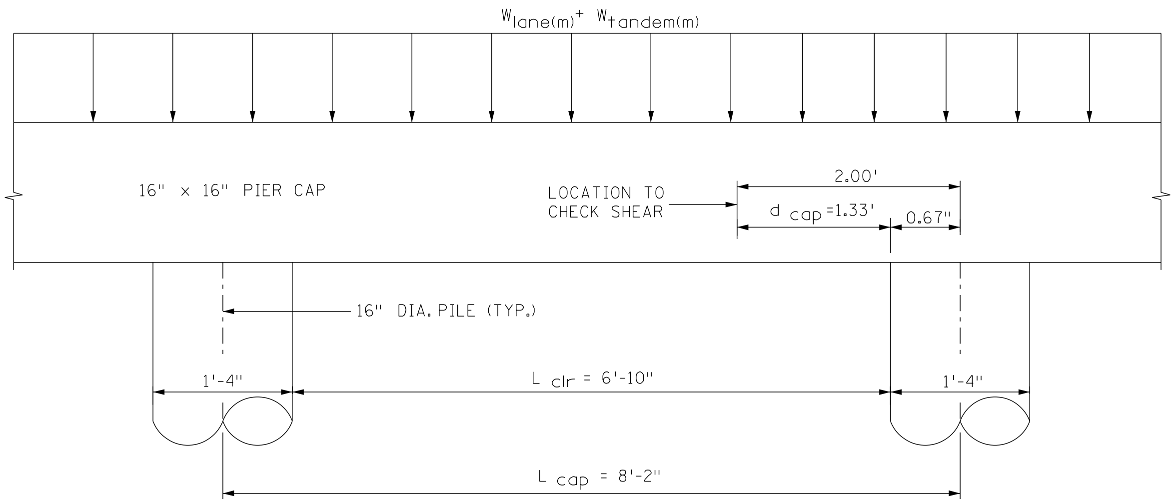


Figure 8.7.2.3 – Cap Shear Check Location

B. Unfactored Shear Forces Acting on Pile Cap

These shear forces are less than the maximums listed in Table 8.7.2.1. The results given below are not the maximum shear forces on the pile cap. Rather, they are the values taken at the appropriate distance "d_{cap}" from the critical support face.

1. Dead Load Shear Force

Component dead load shear force at a distance "d_{cap}" away from the support face = V_{dc} = 4.81 kips

Wear course dead load shear force at a distance "d_{cap}" away from the support face = V_{dw} = 4.78 kips

2. Live Load Shear Forces (Multiple Lanes Loaded)

Only the design tandem and lane loads, for the multiple lanes loaded case, are shown below. From the earlier results, this is the load case that produces the maximum shear force effect on the pier cap being analyzed.

a. Design Tandem Axle Loads

Design tandem shear forces at a distance "d_{cap}" away from the support = $V_{tandem(m)}$ = 13.81 kips

b. Design Lane Load

Design lane shear force at a distance "d_{cap}" from the support = $V_{lane(m)}$ = 4.28 kips

[3.4.1]

C. Factored Shear Force Acting on Pile Cap

1. Load Modifiers

Load modifiers for cap design are shown in the flexure check.

2. Strength I Limit State Load Factors

Use the Strength I Limit State to determine the required shear resistance of the pile cap.

Impact and skew applicability are the same as for the flexure check.

Specific Strength I Limit State Load Factors are found in AASHTO Tables 3.4.1-1 and 3.4.1-2.

The above results (Table 8.7.2.1) indicate that multiple lanes loaded with the design tandem and lane loads control for shear.

3. Strength I Limit State Shear Force

Strength I Limit State factored shear force, two lanes loaded = $V_{u(m)}$

[Tables 3.4.1-1 and 3.4.1-2]

$$V_{u(m)} = \eta \cdot [1.25 \cdot V_{dc} + 1.50 \cdot V_{dw} + 1.75 \cdot r \cdot (V_{tandem(m)} + V_{lane(m)})]$$

$$V_{u(m)} = 1.0 \cdot [1.25 \cdot (4.81) + 1.50 \cdot (4.78) + 1.75 \cdot 1.0 \cdot (13.81 + 4.28)] = 44.84 \text{ kips}$$

Check Shear Resistance of Cap

A. Factored Shear Resistance

The factored shear force ($V_{u(m)}$) is the required shear resistance of the cap that needs to be compared with the actual factored shear resistance of the cap (V_r).

[Eqns. 8.7-1, 8.7-2]

For a rectangular wood section $V_r = \phi_v \cdot F_v \cdot b_{cap} \cdot d_{cap}/1.5$

[8.5.2.2]

1. Resistance Factor

Shear resistance factor = ϕ_v = 0.75

2. Adjustment Factors for Reference Design Values

[8.4.4.2] Format conversion factor: $C_{KF} = 2.5/\phi = 2.5/0.75 = 3.33$

[8.4.4.3] Wet Service factor = C_M = 1.00

[8.4.4.9] Time effect factor = C_λ = 0.80

[Eqn. 8.4.4.1-2]

Adjusted design value = $F_v = F_{vo} \cdot C_{KF} \cdot C_M \cdot C_\lambda$

$F_v = 0.17 \cdot 3.33 \cdot 1.00 \cdot 0.80 = 0.453$ ksi

B. Pile Cap Shear Check

Required pile cap shear resistance = $V_{u(m)}$

For the cap to meet Strength I Limit State, $V_{r(prov)}$ must equal or exceed $V_{u(m)}$. As determined previously, $V_{u(m)} = 44.84$ kips.

[Eqn. 8.7-2]

$$V_{r(prov)} = \phi_v \cdot \frac{(F_v \cdot b_{cap} \cdot d_{cap})}{1.5} = 0.75 \cdot \frac{(0.453 \cdot 16 \cdot 16)}{1.5} = 57.98 \text{ kips}$$

$V_{u(m)} = 44.84$ kips $\leq V_{r(prov)} = 57.98$ kips <u>OK</u>

Investigate Compression Resistance Requirements

A. Unfactored Support Reactions Acting on the Pile Cap

The maximum support reactions are listed in Table 8.7.2.1.

1. Dead Load Reaction Force

Maximum component dead load reaction force = R_{dc} = 15.82 kips

Maximum wear course dead load reaction force = R_{dw} = 15.73 kips

2. Live Load Reaction Forces (Multiple Lanes Loaded)

Only the design tandem and lane load reactions, for the multiple lanes loaded case, are shown below. From the earlier results, this is the load case that produces the maximum reaction forces.

a. Design Tandem Axle Loads

Maximum design tandem reaction force = $R_{tandem(m)}$ = 45.37 kips

b. Design Lane Load

Maximum design lane reaction force = $R_{lane(m)}$ = 14.06 kips

[3.4.1]

B. Factored Support Reaction Forces Acting on Pile Cap

Strength I Limit State maximum factored support reaction due to two lanes loaded case = $P_{u(m)}$

[Tables 3.4.1-1 and 3.4.1-2]

$$P_{u(m)} = \eta \cdot [1.25 \cdot R_{dc} + 1.50 \cdot R_{dw} + 1.75 \cdot r \cdot (R_{tandem(m)} + R_{lane(m)})]$$

$$P_{u(m)} = 1.0 \cdot [1.25 \cdot (15.82) + 1.50 \cdot (15.73) + 1.75 \cdot 1.0 \cdot (45.37 + 14.06)]$$

$$= 147.37 \text{ kips}$$

Check Compression Resistance of Cap

A. Factored Bearing Resistance

The maximum factored support reaction $P_{u(m)}$ is the required compression resistance perpendicular to the grain of the cap that needs to be compared with the actual factored compression resistance perpendicular to the grain of the cap (P_r).

[Eqns. 8.8.1-1, 8.8.3-1]

$$P_r = \phi_{cperp} \cdot F_{cp} \cdot A_b \cdot C_b$$

[8.5.2.2]

1. Resistance Factor

Compression perpendicular to grain resistance factor = ϕ_{cperp} = 0.90

2. Adjustment Factors for Reference Design Values

[8.4.4.2] Format conversion factor: $C_{KF} = 2.1/\phi = 2.1/0.90 = 2.33$

[8.4.4.3] Wet Service factor = C_M = 0.67

[8.4.4.9] Time effect factor = C_λ = 0.80

[Eqn. 8.4.4.1-5] Adjusted design value = $F_{cp} = F_{cpo} \cdot C_{KF} \cdot C_M \cdot C_\lambda$

$F_{cp} = 0.625 \cdot 2.33 \cdot 0.67 \cdot 0.80 = 0.781$ ksi

3. Pile Cap Bearing Dimensions

For this calculation contribution from other steel on the top of the pile such as the leveling ring are conservatively ignored. Only the steel pile top plate thickness of 3/8 inches is added to the pile diameter for the area considered effective for bearing resistance of the cap.

Bearing length = $L_b = ½ \cdot d_{pile}$ = 8 in

Bearing width = $b_b = ½ \cdot d_{pile}$ = 8 in

Bearing Area = $A_b = [\pi \cdot (d_{pile})^2] / 4 = [\pi \cdot (16.75)^2] / 4 = 220.35$ in^2

4. Bearing Adjustment Factor

[Table 8.8.3-1] Adjustment Factor for Bearing = C_b

$L_b = 8.0$ in ≥ 6.0 in C_b =1.00

B. Pile Cap Bearing Resistance Check

Required pile cap compression resistance = $P_{u(m)}$ = 147.37 kips

For the cap to meet Strength I Limit State, provided compression resistance perpendicular to grain = $P_{r(prov)}$ must equal or exceed $P_{u(m)}$.

[Eqn. 8.8.3-1] $P_{r(prov)} = \phi_{cperp} \cdot F_{cp} \cdot A_b \cdot C_b = 0.9 \cdot 0.781 \cdot 220.35 \cdot 1.0 = 154.88$ kips

$P_{u(m)} = 147.37$ kips $\leq P_{r(prov)} = 154.88$ kips   OK

8.7.3 Glulam Beam Superstructure Design Example

This example goes through the design of glulam beams. The glulam beams are the main load carrying members for the bridge span and will have transverse timber deck panels. The last design example found in Article 8.7.4 will be for two different transverse deck types that could be used on these glulam beams to support the road surface: spike laminated deck panels, and glulam deck panels. This bridge type is also intended for use on secondary roads with low truck traffic volumes. The glulam beams being designed are intended to span from substructure to substructure.

[8.4.1.2]

The beams are required to be manufactured using wet use adhesives to join the individual laminates to attain the specified beam size, and under this condition the adhesive bond is stronger than the wood laminates. The beams are to be manufactured meeting the requirements of ANSI/AITC A190.1. Lamination widths for Western Species and for Southern Pine are shown in AASHTO LRFD, and the table of design values. A more complete list of beam sizes, as well as design values, is provided in the NDS.

A. Material and Design Parameters

[Figure 8.3-1]

The dimension annotations used throughout this design example are as follows. The vertical dimension of a member is considered its depth. The transverse and longitudinal measurements of a member are considered its width and length, respectively. These dimension annotations are consistent with Figure 8.3-1 of the *2014 AASHTO LRFD Bridge Design Specifications* for glulam beams (w_{bm} & d_{bm} used here). The letter notations shown in Figure 8.3-1 for sawn components will be used here for the sawn components (b, d, etc.).

[8.4.1.2.2]

For glulam beams, the timber dimensions stated shall be taken as the actual net dimensions.

1. End of Beam Support

The ends of the glulam beams could be supported by timber pile caps or bearing pads as part of a single span or multi span bridge superstructure. For the purposes of this example, a single span superstructure supported by bearing pads on concrete substructures will be assumed. The bearing pad design is not a part of this design example, it will be assumed that the compression in the wood governs the bearing area size.

[9.9.8]

2. Bituminous Wearing Surface

MnDOT uses a 2% cross slope whenever practicable. In this case, a minimum thickness of 2 inches at edge of roadway (face of curb) and

6 inches thickness at centerline of the road gives an average depth of wearing course = 4 in.

However, using a constant longitudinal thickness on a bridge superstructure with glulam beams will result in a roadway surface with a hump due to the beam camber. It is preferred to construct the final top of bituminous surface uniformly in the longitudinal direction on the deck.

If the glulam beam is cambered and the top of driving surface on the bituminous is uniform, or follows the grade for a road having a straight line profile grade, the bituminous thickness must vary longitudinally. It may vary more, if for example, the profile grade has a sag vertical curve that the bituminous must accommodate. The profile grade for specific bridge designs should be reviewed to make certain the proper bituminous thickness is used in the design of the glulam beams.

For this design example, an extra 0.45 inches average bituminous thickness is assumed which is conservatively based on a straight line average. This will be verified later in this Glulam Beam Superstructure Design Example after the beam camber is calculated. Therefore, the bituminous wearing surface thickness that will be used in the dead load calculations below for the glulam beams in this design example
= d_{ws} = 4.45 in.

3. Curb and Railing (TL-4 Glulam Timber Rail w/Curb on transv. deck)
Width of timber curb = b_{curb} = 12 in
Depth of timber curb = d_{curb} = 6.75 in
Width of timber rail post = b_{post} = 10.5 in
Length of timber rail post = L_{post} = 8.75 in
Depth of timber rail post = d_{post} = 37.5 in
Width of timber spacer block = b_{spacer} = 3.125 in
Length of timber spacer block = L_{spacer} = 8.75 in
Depth of timber spacer block = d_{spacer} = 10.5 in
Width of timber scupper = $b_{scupper}$ = 12 in
Length of timber scupper = $L_{scupper}$ = 54 in
Depth of timber scupper = $d_{scupper}$ = 6.75 in
Width of timber rail = b_{rail} = 8.75 in
Depth of timber rail = d_{rail} = 13.5 in
Spacing between barrier posts = s_{post} = 8.0 ft = 96 in (maximum)

The timber barrier design is not a part of this design example, but the dimensions are used for weight considerations. Refer to the resources noted earlier in Article 8.5 of this manual for the TL-4 Crash Tested Bridge Rail details.

[8.4.1.2]

4. Glulam Beams

Assumed depth of glulam timber beams = d_{bm} = 46.75 in
Assumed width of glulam timber beams = w_{bm} = 8.5 in

[8.4.1.2.2]

Glulam beams are supplied to the dimensions specified. Attention must be given to the species of wood, as laminate sizes vary based on species.

5. Span Lengths

Actual longitudinal length of the beams, which is also the deck length, or bridge length = L = 43.50 ft

MnDOT uses the effective span, or design span, as center to center of the beam bearing lengths. The assumed beam bearing length (18 in) is checked at the end of this Glulam Beam Superstructure Design Example.

Effective span length for the single span of glulam beams = L_e

$$L_e = L - 2 \cdot \frac{1}{2} \cdot L_b = 43.50 - 2 \cdot \frac{1}{2} \cdot \frac{18}{12} = 42.0 \text{ ft}$$

6. Unit Weights and Moisture Content

[Table 8.4.1.2.3-1]

Type of glulam beam wood material (outer/core laminates are the same species): Southern Pine – SP/SP (24F-V3).

[Table 3.5.1-1]

Unit weight of soft-wood = γ_{SP} = 0.050 kcf.

The deck will also be comprised of a soft-wood (Southern Pine or Douglas Fir). For this design example, "SP" is shown as the unit weight for the deck, but any softwood will have the same unit weight.

[MnDOT Table 3.3.1]
[MnDOT 3.3]

Unit weight of bituminous wearing surface = γ_{ws} = 0.150 kcf
Standard MnDOT practice is to apply a future wearing course of 20 psf.

[8.4.4.3]

MnDOT designs for in-service wet-use only which is a MC of greater than 16% for glulam.

[Table 8.4.1.2.3-1]

7. Southern Pine Structural Glulam (24F-V3) Strength Properties

Reference Design Value for flexure = F_{bxo} = 2.400 ksi
Reference Design Value for compression perpendicular to grain = F_{cpo} =0.740 ksi (end bearing is on tension face)
Reference Design Value for shear parallel to grain = F_{vxo} = .300 ksi (for checking horizontal shear)
Modulus of elasticity = Ex_o = 1800 ksi

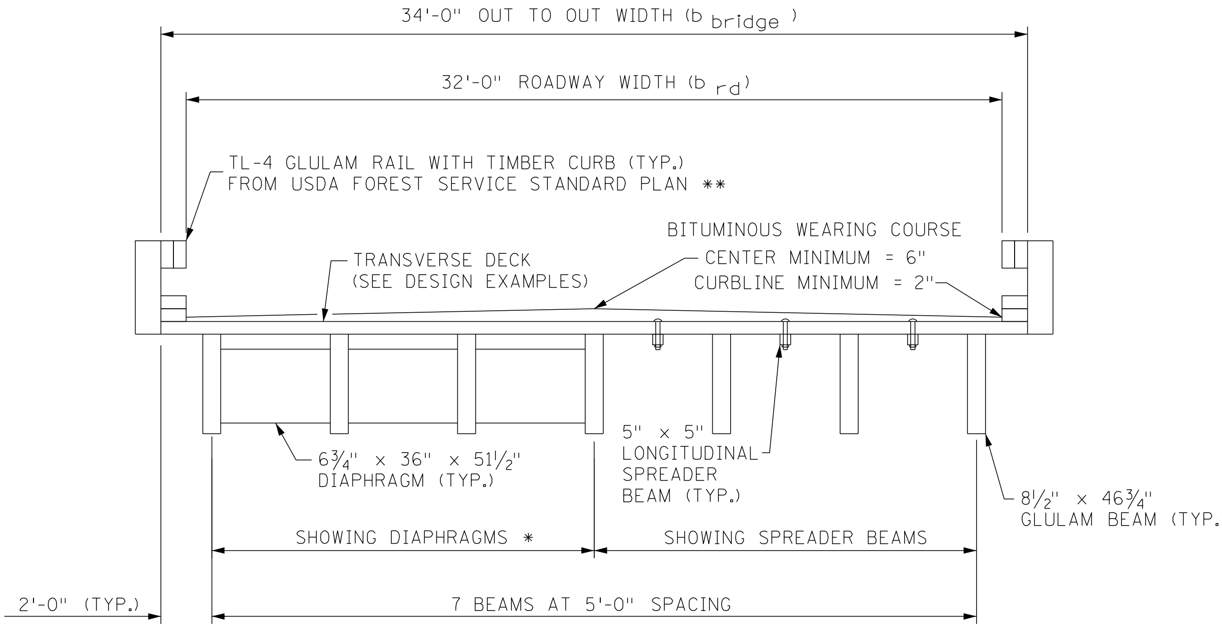


Figure 8.7.3.1 – Glulam Beams Layout

*Timber diaphragms are located near each bearing and at mid span
**Rail (barrier) posts spacing is 8.0 ft

Select the Basic Configuration

The bridge consists of 7 equally spaced glulam beams of the same size with a transverse wood deck. It is recommended to attach the deck to the beams with lag screws to stabilize the deck and prevent excess cracking in the bituminous wear course (refer to Article 8.7.4 narrative). Each glulam beam is designed as a simply supported member.

[8.11.3]

Minimal specific guidance is provided in AASHTO LRFD for bracing requirements of glulam beams. It only states that fabricated steel shapes or solid wood blocks should be used. Wood is commonly used for blocking on wood beam bridges, and generally is less cost and easier to install than steel. Also, solid wood blocks require less design effort than designing steel and associated connectors.

For deeper glulam beams, glulam diaphragms are used to attain the appropriate depth. Traditionally transverse bracing was required to be a minimum of ¾ the depth of a bending member and is currently specified in AASHTO LRFD for sawn wood beams, so that can be used as a guide on current glulam beam designs. The maximum spacing of 25.0 ft for sawn beams can also be used as a guide for standard glulam beam designs. The designer needs to check that lateral stability requirements for bending members are being met for individual designs.

[9.9.4.3]

B. Panel Dimensions and Bridge Width Deck

The transverse deck design example is found in Article 8.7.4 of this manual. It includes both a design for a spike laminated deck panel assembled from sawn lumber and a design for a deck panel that is glulam. For glulam the dimensions are taken as the actual net dimensions. The sawn lumber is typically surfaced one side and one edge, and so the nominal deck thickness dimension is used for dead load.

The spike laminated deck thickness of 6 inches is used for the deck dead load in this glulam beam design example because that has a larger dead load effect than the glulam deck. The spike laminated deck also causes the live load fraction on the beam to be larger than with a glulam deck, and so creates the worst case force effects of the two deck types for the beam design.

The transverse deck design example incorporates the use of a longitudinal stiffener beam, or spreader beam, for the deck panels to be considered interconnected in accordance with AASHTO LRFD. The dead load of the spreader beam will be included in the deck dead load for this glulam beam design example, and the size determination (5 in x 5 in) for the spreader beam is shown in the transverse deck design example.

Length of bridge deck panels = b_1 = 34.0 ft

Overall width of bridge deck = b_{bridge} = 34.0 ft

Width of each timber barrier = $b_{barrier}$ = 1.0 ft

Width of roadway = $b_{rd} = b_{bridge} - 2 \cdot b_{barrier} = 34.0 - (2 \cdot 1) = 32.0$ ft

C. Beam Spacing Dimensions

The exterior beam should generally be near enough to the outside deck edge so that the deck overhang and the exterior beam do not govern the respective designs. However, economy is gained by not placing the beam at the outside deck edge (possibly less total beams required).

[3.6.1.3] Looking at AASHTO LRFD for the application of vehicular live load, the tire on a truck axle is basically placed 1.0 ft from the face of curb or railing for deck design, and 2.0 ft for the design of all other components. Using the 1.0 ft for deck design, the tire would occur 2.0 ft from the edge deck, and so if a beam is placed here the outside deck cantilever will not govern. Typically the exterior beam then would also not govern, because applying the 2.0 ft for the design of all other components the tire on the axle would occur inside of the exterior beam. For this design example, a

2.0 ft overhang each side measured from center of the exterior beam to edge of deck will be tried.

[Table 4.6.2.2.2a-1]

The live load distribution to an interior beam is determined from the table in AASHTO LRFD. The range of applicability for this table is a maximum beam spacing of 6.0 ft. A beam spacing of 5.0 ft fits within this range, and so that will be tried for this glulam beam design example.

Determine Dead and Live Load Bending Moments

A. Dead Loads per Beam

The units for the dead load results are given in kips per foot for one beam. MnDOT assumes that the barrier load for all wood structure types acts uniformly over the bridge width. Deck and wear course are calculated based on tributary area for simplicity, as the exterior beam generally will not govern for typical designs. Exterior beam loads are shown in the design example to illustrate that the exterior beam will not govern the design.

1. Dead Loads per longitudinal foot

Weight of beam = $w_{beam} = \gamma_{SP} \cdot d_{bm} \cdot w_{bm} = 0.050 \cdot 46.75/12 \cdot 8.5/12$
$= 0.138$ klf

Weight of deck, interior beams (including spreader beam)
$= w_{deck\_int} = \gamma_{SP} \cdot d_{deck} \cdot s_{int\_bm} + \gamma_{SP} \cdot d_{spdr} \cdot b_{spdr}$
$= (0.050 \cdot 6/12 \cdot 5.0) + (0.050 \cdot 5/12 \cdot 5/12) = 0.134$ klf

Weight of deck, exterior beams (including spreader beam)
$= w_{deck\_ext} = \gamma_{SP} \cdot d_{deck} \cdot s_{ext\_bm} + \gamma_{SP} \cdot d_{spdr} \cdot b_{spdr} \cdot ½$
$= (0.050 \cdot 6/12 \cdot 4.5) + (0.050 \cdot 5/12 \cdot 5/12 \cdot 1/2) = 0.117$ klf

Weight of wearing surface, interior beams = $w_{ws\_int} = \gamma_{ws} \cdot d_{ws} \cdot s_{int\_bm}$
$= 0.150 \cdot 4.45/12 \cdot 5.0 = 0.278$ klf

Weight of wearing surface, exterior beams = $w_{ws\_ext} = \gamma_{ws} \cdot d_{ws} \cdot s_{ext\_bm}$
$= 0.150 \cdot 3.0/12 \cdot 3.5 = 0.131$ klf

Weight of future wearing course, interior beams = $w_{FWC} \cdot s_{int\_bm}$
$= 0.020 \cdot 5 = 0.100$ klf

Weight of future wearing course, exterior beams = $w_{FWC} \cdot s_{ext\_bm}$
$= 0.020 \cdot 3.5 = 0.070$ klf

2. Determine linear weight of rail system elements.

Volume of timber curb per foot of bridge length = v_{curb}

$v_{curb} = (b_{curb} \cdot d_{curb} \cdot 12 \text{ in/ft}) = (12 \cdot 6.75 \cdot 12) = 972.0 \text{ in}^3\text{/ft}$

Volume of rail post and spacer block per foot of bridge length = v_{post}

$v_{post} = (b_{post} \cdot L_{post} \cdot d_{post} + b_{spacer} \cdot L_{spacer} \cdot d_{spacer}) / s_{post}$

$v_{post} = [(10.5 \cdot 8.75 \cdot 38) + (3.125 \cdot 8.75 \cdot 10.5)] / 8$

$= 472.3\ in^3/ft$

Volume of scupper per foot of bridge length = $v_{scupper}$

$v_{scupper} = (b_{scupper} \cdot L_{scupper} \cdot d_{scupper}) / s_{post}$

$v_{scupper} = (12 \cdot 54 \cdot 6.75) / 8 = 546.75\ in^3/ft$

Volume of timber rail per foot of bridge length = v_{rail}

$v_{rail} = (b_{rail} \cdot d_{rail} \cdot 12\ in/ft) = (8.75 \cdot 13.5 \cdot 12) = 1417.5\ in^3/ft$

Volume of timber railing per longitudinal foot of bridge length = $v_{barrier}$

$v_{barrier} = v_{curb} + v_{post} + v_{scupper} + v_{rail}$

$v_{barrier} = 972.0 + 472.3 + 546.75 + 1417.5 = 3408.6\ in^3/ft$

$= 1.973\ ft^3/ft$

Total linear weight of combined timber curbs/railings = $w_{barrier}$

$$w_{barrier} = \frac{2 \cdot \gamma_{DFL} \cdot v_{barrier}}{beams_{total}} = \frac{2 \cdot 0.050 \cdot 1.973}{7} = 0.028\ klf$$

This linear weight result assumes that the curb/railing weight acts uniformly over the entire deck width.

3. Diaphragm point loads

Volume of diaphragm = $v_{diaph} = b_{diaph} \cdot L_{diaph} \cdot d_{diaph}$
$= (51.5 \cdot 6.75 \cdot 36)/1728 = 7.242\ ft^3$

Diaphragm load, interior beams = $P_{diaph\_int} = \gamma_{DFL} \cdot v_{diaph}$
$= 0.050 \cdot 7.242 = 0.362$ kips

Diaphragm load, exterior beams = $P_{diaph\_ext} = (\gamma_{DFL} \cdot v_{diaph}) / 2$
$= (0.050 \cdot 7.242) / 2 = 0.181$ kips

B. Dead Load Bending Moments per Beam

1. Moments of Individual loads

[AISC 14th p. 3-213] Maximum bending moment due to beam weight

$$M_{beam} = \frac{w_{bm} \cdot (L_e)^2}{8} = \frac{0.138 \cdot 42.0^2}{8} = 30.43\ \text{kip - ft}$$

Maximum bending moment due to deck weight, interior beams

$$M_{deck\_int} = \frac{w_{deck\_int} \cdot (L_e)^2}{8} = \frac{0.134 \cdot 42.0^2}{8} = 29.55\ \text{kip–ft}$$

Maximum bending moment due to deck weight, exterior beams

$$M_{deck\_ext} = \frac{w_{deck\_ext} \cdot (L_e)^2}{8} = \frac{0.117 \cdot 42.0^2}{8} = 25.80 \text{ kip–ft}$$

Maximum bending moment due to wearing surface, interior beams

$$M_{ws\_int} = \frac{w_{ws\_int} \cdot (L_e)^2}{8} = \frac{0.278 \cdot 42.0^2}{8} = 61.30 \text{ kip–ft}$$

Maximum bending moment due to wearing surface, exterior beams

$$M_{ws\_ext} = \frac{w_{ws\_ext} \cdot (L_e)^2}{8} = \frac{0.131 \cdot 42.0^2}{8} = 28.89 \text{ kip–ft}$$

Maximum bending moment due to future wearing course, interior beams

$$M_{FWC\_int} = \frac{w_{FWC\_int} \cdot (L_e)^2}{8} = \frac{0.100 \cdot 42.0^2}{8} = 22.05 \text{ kip–ft}$$

Maximum bending moment due to future wearing course, exterior beams

$$M_{FWC\_ext} = \frac{w_{FWC\_ext} \cdot (L_e)^2}{8} = \frac{0.070 \cdot 42.0^2}{8} = 15.44 \text{ kip–ft}$$

Maximum bending moment due to diaphragm weight, interior beams

$$M_{diaph\_int} = \frac{P_{diaph\_int} \cdot L_e}{4} = \frac{0.362 \cdot 42.0}{4} = 3.80 \text{ kip–ft}$$

Maximum bending moment due to diaphragm weight, exterior beams

$$M_{diaph\_ext} = \frac{P_{diaph\_ext} \cdot L_e}{4} = \frac{0.181 \cdot 42.0}{4} = 1.90 \text{ kip–ft}$$

Maximum bending moment due to curb/railing weight = $M_{barrier}$

$$M_{barrier} = \frac{w_{barrier} \cdot (L_e)^2}{8} = \frac{0.028 \cdot 42.0^2}{8} = 6.17 \text{ kip–ft}$$

2. Sum of Dead Load Moments per Beam

a. Interior Beam

Maximum bending moment due to bridge component dead loads, interior beam

$M_{dc\_int} = M_{beam} + M_{deck\_int} + M_{diaph\_int} + M_{barrier}$

$= 30.43 + 29.55 + 3.80 + 6.17 = 69.95$ kip·ft

Maximum bending moments due to wearing surface loads, interior beam

$M_{dw\_int} = M_{ws\_int} + M_{FWC}$

$= 61.30 + 22.05 = 83.35$ kip·ft

b. Exterior Beam

Maximum bending moment due to bridge component dead loads, exterior beam

$M_{dc\_ext} = M_{beam} + M_{deck\_ext} + M_{diaph\_ext} + M_{barrier}$
$= 30.43 + 25.80 + 1.90 + 6.17 = 64.30$ kip·ft

Maximum bending moments due to wearing surface loads, exterior beam

$M_{dw\_ext} = M_{ws\_ext} + M_{FWC}$
$= 28.89 + 15.44 = 44.33$ kip·ft

[3.6.1.2]

C. Live Load Bending Moments

The live load bending moment will be calculated per lane (12 ft) and later converted to a per beam format.

1. Design Truck Axle Loads

[3.6.1.2.2]

Point loads and spacing of the design truck axles are shown in AASHTO LRFD Figure 3.6.1.2.2-1.

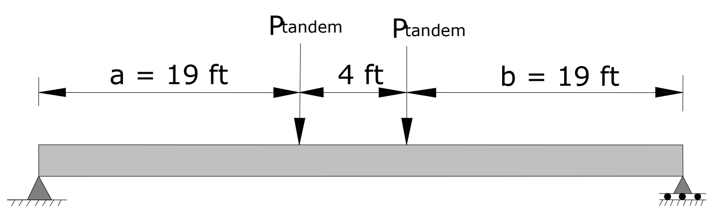
Maximum bending moment due to design truck axle load = M_{truck}. This truck moment is available in multiple reference tables (including Table 3.4.1.2 in this manual) for a 42.0 ft span.

$M_{truck} = 485.2$ kip·ft

2. Design Tandem Axle Loads

[3.6.1.2.3]

Point load of design tandem axle = P_{tandem} = 25 kips, spaced at 4 ft.



Maximum bending moment due to design tandem axle loads = M_{tandem}

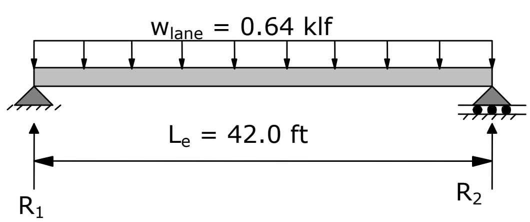
[AISC 14th p. 3-228]

$M_{tandem} = Pa = 25.0 \cdot 19.0 = 475.0$ kip·ft

[3.6.1.2.4]

3. Design Lane Loads

Uniform design lane load = w_{lane} = 0.64 klf



Maximum bending moment due to design lane load = M_{lane}

$$M_{lane} = \frac{w_{lane} \cdot (L_e)^2}{8} = \frac{0.64 \cdot 42^2}{8} = 141.1 \text{ kip-ft}$$

[4.6.2.2]

D. Live Load Distribution

The live load bending moments, calculated above, need to be distributed to a per beam basis.

The transverse deck design example next in the Chapter after this beam design example includes both a design for a spike laminated deck panel assembled from sawn lumber and a design for a deck panel that is glulam. A spike laminated deck gives a higher wheel load fraction and so that will be used for this beam design example (it is the worst case).

[3.6.1.1.1] Maximum number of traffic lanes on the deck = N_L

$$N_L = \frac{b_{rd}}{12 \frac{\text{ft}}{\text{lane}}} = \frac{32}{12} = 2.67 \cong 2 \text{ lanes}$$

[Table 4.6.2.2.2a-1] Live Load Distribution Factor (g_{int}) for interior beams is calculated using beam spacing (S), and is based on deck type and number of loaded lanes.

[3.6.1.1.2] The multiple presence factors are not intended to be applied in conjunction with the load distribution factors specified in Table 4.6.2.2.2a-1. The multiple presence factors have been accounted for in these equations.

[Table 4.6.2.2.2a-1] Two or more design lanes loaded is compared with one design lane loaded to determine the Live Load Distribution Factor to use here.

Two or more design lanes loaded:

$$g_{int} = \frac{S}{8.5} = 0.59 \text{ Design Truck, interior beam}$$

One design lane loaded:

$$g_{int} = \frac{S}{8.3} = 0.60 \text{ Design Truck, interior beam}$$

One lane loaded gives the higher live load distribution to an interior beam, and so the interior Live Load Distribution Factor = g_{int} = 0.60.

[4.6.2.2.2d] Typically the live load flexural moment for exterior beams is determined by applying the Live Load Distribution Factor (LLDF) specified for exterior beams. For this design example, the specified exterior Live Load Distribution Factor, $LLDF_{ext}$, is the lever rule.

[3.6.1.3] The design vehicle is to be placed no closer than 2.0 ft from the edge of the design lane. The most severe force effect is with the edge of design lane at the face of the timber curb. For this design example, this would place one tire (0.50 Design Trucks) 1.0 ft inside of the beam and the other inside of the next beam (which is then ignored for the lever rule applied to the exterior beam).

[C3.6.1.1.2] When using the lever rule, the multiple presence factor must be applied manually.

[Table 3.6.1.1.2-1] Similar as for the Live Load Distribution Factor for the interior beams, one lane loaded produces the largest force effect on the exterior beams, with the multiple presence factor m = 1.20 applied to the $LLDF_{ext}$.

[Table 4.6.2.2.2d-1] Exterior Live Load Distribution Factor = g_{ext} = $LLDF_{ext}$ x m.

$$g_{ext} = \frac{0.50 \text{ Design Truck} \cdot 4\text{ft}}{5\text{ft}} \cdot 1.20 = 0.48 \text{ Design Truck, exterior beam}$$

It can be seen that as originally assumed above in "Select the Basic Configuration", the interior beam will have the more severe live load force effect.

E. Live Load Moments per Beam

a. Interior Beam

Maximum moments from design truck load single lane = $M_{truck(s)}$

$$M_{truck(s)} = M_{truck} \cdot g_{int} = 485.2 \cdot 0.60 = 291.12 \text{ kip–ft}$$

Maximum moment from design tandem load single lane = $M_{tandem(s)}$

$$M_{tandem(s)} = M_{tandem} \cdot g_{int} = 475.0 \cdot 0.60 = 285.00 \text{ kip–ft}$$

Maximum moment from design lane load single lane = $M_{lane(s)}$

$$M_{lane(s)} = M_{lane} \cdot g_{int} = 141.1 \cdot 0.60 = 84.66 \text{ kip–ft}$$

b. Exterior Beam

Because $g_{ext} < g_{int}$ as checked above in Part D., exterior beam live load moments will not be calculated.

F. Summary of Unfactored Dead and Live Load Bending Moments per Beam

Table 8.7.3.1 - Applied Bending Moments

Unfactored Load Case	Maximum Positive Bending Moment (kip·ft)
Dead Loads (interior beam)	
Bridge Components (M_{dc})	69.95
Bridge Wearing Surface (M_{dw})	83.35
Dead Loads (exterior beam)	
Bridge Components (M_{dc})	64.30
Bridge Wearing Surface (M_{dw})	44.33
Live Loads (interior beam, for single lane)	
Design Truck	291.12
Design Tandem	285.00
Design Lane	84.66

G. Factored Bending Moment per Beam

1. Load Modifiers

Standard MnDOT Load Modifiers are summarized in Table 3.2.1 of this manual.

For timber bridges η_D = 1.0. MnDOT considers four or more beams to have a conventional level of redundancy and uses η_R = 1.0. This example bridge is assumed to have a design ADT of over 500 for η_I = 1.0.

[1.3.2] Therefore, importance, redundancy, and ductility factors = η = 1.0

2. Strength I Limit State Load Factors

[3.4.1] Use the Strength I Limit State to determine the required resistance for the beams.

[3.6.2.3] Impact factor need not be applied to wood components.

Specific Strength I Limit State Load Factors are found in AASHTO Tables 3.4.1-1 and 3.4.1-2.

The earlier analysis showed that the design truck load controls the bending moment of the beams. Additionally, the analysis determined that the interior beams will govern with one lane loaded. Therefore, use the design truck load with the uniform lane load in determining the critical live load bending moment acting on the interior beams.

Also, the earlier analysis calculated dead load bending moment on both the interior and exterior beams. The bending moments from dead load are larger on the interior beams. Strength checks only need to be done for the interior beams, since all beams shall be the same size.

[4.6.2.2.1]

3. Strength I Limit State Bending Moment per Beam

Factored bending moment for two lanes loaded case = $M_{u(m)}$

[Tables 3.4.1-1 and 3.4.1-2]

$$M_{u(m)} = \eta \cdot [1.25 \cdot M_{dc} + 1.50 \cdot M_{dw} + 1.75 \cdot r \cdot [M_{truck(m)} + M_{lane(m)}]]$$

$$M_{u(m)} = 1.0 \cdot [1.25 \cdot 69.95 + 1.50 \cdot 83.35 + 1.75 \cdot 1.0 \cdot [291.12 + 84.66]] = 870.08 \text{ kip-ft}$$

Check Flexural Resistance of Beams

A. Factored Flexural Resistance

The factored bending moment ($M_{u(m)}$) is the required flexural resistance of the beam that needs to be compared with the actual factored flexural resistance of the beam (M_r).

[8.6.2]

For a rectangular wood section $M_r = \phi_f \cdot F_b \cdot S_{req} \cdot C_L$.

[8.5.2.2]

1. Resistance Factors

Flexural resistance factor = ϕ_f = 0.85

Compression perpendicular to grain resistance factor = ϕ_{cperp} = 0.90

2. Provided Section Modulus

The section modulus is dependent on the beam size. The provided beam section modulus is determined from the beam dimensions assumed at the start of the design example.

The provided beam section modulus = $S_{prov} = \dfrac{w_{bm} \cdot d_{bm}^2}{6}$

$$S_{prov} = \frac{8.5 \cdot 46.75^2}{6} = 3096.21 \text{ in}^3$$

[8.6.2]

3. Stability Factor

Stability factor for the glulam beams in flexure = C_L. The stability factor shall not be applied simultaneously with the volume factor for structural glued laminated timber. In this case the beams are laterally supported and so the Stability Factor C_L = 1.0. The volume factor will be the lesser of the two values and is what will be used in the adjusted design value.

4. Adjustment Factors for Reference Design Value

[8.4.4.2]

Format conversion factor for component in flexure = C_{KF}

$C_{KF} = 2.5/\phi = 2.5/0.85 = 2.94$

[8.4.4.3]
[Table 8.4.4.3-2]

Wet Service factor for glued laminated timber in flexure = C_M

For structural glulam, wet service condition C_M = 0.80

[8.4.4.5]

Volume factor for structural glulam timber in flexure, when loads are applied to wide face of laminations = C_V (a = 0.05 for Southern Pine). The beams for this design example are not tension reinforced which represent the most commonly used beam type in Minnesota.

[Eqn. 8.4.4.5-1]

$$C_V = \left[\left(\frac{12}{d_{bm}}\right)\cdot\left(\frac{5.125}{w_{bm}}\right)\cdot\left(\frac{21}{L_e}\right)\right]^a \le 1.0$$

$$C_V = \left[\left(\frac{12}{46.75}\right)\cdot\left(\frac{5.125}{8.5}\right)\cdot\left(\frac{21}{42}\right)\right]^{0.05} = 0.88$$

[8.4.4.9]
[Table 8.4.4.9-1]

Time effect factor for Strength I Limit State = C_λ

$C_\lambda = 0.80$

[Eqn. 8.4.4.1-1]

Adjusted design value = $F_b = F_{bxo} \cdot C_{KF} \cdot C_M \cdot C_V \cdot C_\lambda$

$F_b = 2.400 \cdot 2.94 \cdot 0.80 \cdot 0.88 \cdot 0.80 = 3.97$ ksi

B. Beam Flexural Check

Required beam flexural resistance = $M_{u(m)}$

For the beam to meet Strength I Limit State, M_r must equal or exceed $M_{u(m)}$. As determined previously, $M_{u(m)} = 870.08$ kip·ft

Provided beam factored flexural resistance:

$M_{r(prov)} = \phi_f \cdot F_b \cdot S_{prov} \cdot C_L = 0.85 \cdot 3.97 \cdot 3096.21 \cdot 1.0$

$= 10{,}448.16$ kip·in $= 870.68$ kip·ft

$M_{u(m)} = 870.08$ kip·ft $\le$ $M_{r(prov)} = 870.68$ kip·ft <u>OK</u>

The required beam size indicates that the originally assumed beam size can be used, based on calculations using the worst case effect of the two deck types. Next, the beam size will be checked against deflection limits.

Investigate Deflection Requirements
[8.5.1]
[3.6.1.3.2]
[2.5.2.6.2]

A. Beam Live Load Deflection with Current Parameters

The midspan deflections are to be taken as the larger of the design truck or 25% of the design truck applied in conjunction with the design lane load.

Deflections are to be calculated using Service I Limit State.

With all design lanes loaded, it is allowed to assume all supporting components deflect equally for straight girder systems.

Then, the deflection distribution factor, DF_Δ, is determined as follows.

$$DF_\Delta = m \cdot \frac{(\#\text{ of lanes})}{(\#\text{ of beam lines})}$$

[Table 3.6.1.1.2-1]

for m = 1.0 (2 lanes loaded), $DF_\Delta = 1.0 \cdot \frac{2}{7} = 0.286$

[2.5.2.6.2]

[C2.5.2.6.2]

In the absence of other criteria, the recommended deflection limit in AASHTO LRFD for wood construction is span/425, which will be used here. The designer and owner should determine if a more restrictive criteria is justified, such as to reduce bituminous wearing course cracking and maintenance.

1. Beam Stiffness

Moment of inertia of one beam = I_{prov}

$$I_{prov} = \frac{1}{12} \cdot w_{bm} \cdot d_{bm}^3 = \frac{1}{12} \cdot 8.5 \cdot (46.75)^3 = 72{,}374 \text{ in}^4$$

[Table 8.4.4.3-2]
[Eqn. 8.4.4.1-6]

Beam modulus of elasticity with wet service included = E, (C_M =0.833)

$E = E_o \cdot C_M$ = 1800 ksi · 0.833 = 1499.4 ksi

2. Live Loads

The truck deflection can be calculated with a beam program, or alternatively there are various tables available. One method is the use of a coefficient that is divided by EI_{prov}.

Design truck load used for deflection calculations = $P_{\Delta truck}$

Coefficient for a 42.0 ft span = $P_{\Delta truck}$ = 1.468 x 10^{11}

(from reference 3 in Article 8.6 of this manual)

Design lane load used for deflection calculations = $w_{\Delta lane}$

$w_{\Delta lane}$ = 0.64 klf

3. Live Load Deflection Calculations

[3.6.1.3.2]
[AISC 14th p. 3-213]

Deflection at beam midspan due to the design truck load = Δ_{truck}

$$\Delta_{truck} = DF_\Delta \cdot \frac{P_{\Delta truck}}{E \cdot I_{prov}} = 0.286 \cdot \frac{1.468 \times 10^{11}}{1499.4 \cdot 72{,}374} = 0.387 \text{ in}$$

Deflection at beam midspan due to the design lane load = Δ_{lane}

$$\Delta_{lane} = DF_\Delta \frac{5 \cdot w_{\Delta lane} \cdot L_e^4}{384 \cdot E \cdot I_{prov}} = 0.286 \cdot \frac{5 \cdot \frac{0.64}{12} \cdot (42.0 \cdot 12)^4}{384 \cdot 1499.4 \cdot 72{,}374} = 0.118 \text{ in}$$

Deflection at beam midspan due to a combination of truck (25%) and design lane load = $\Delta_{combined}$

$\Delta_{combined} = (0.25 \cdot \Delta_{truck}) + \Delta_{lane} = (0.25 \cdot 0.387) + 0.118$

$\Delta_{combined} = 0.215 \text{ in} \leq \Delta_{truck} = 0.387 \text{ in}$

Therefore, the maximum deflection between the combination load deflection and the truck load deflection = $\Delta = \Delta_{truck} = 0.387$ in

[2.5.2.6.2]

Live load deflection limit at beam midspan = Δ_{max}

$\Delta_{max} = L_e / 425 = 42.0 / 425 = 0.0988 \text{ ft} = 1.186 \text{ in}$

$\Delta = 0.387 \text{ in} \leq \Delta_{max} = 1.186 \text{ in}$ OK

The initial beam size and grade are adequate for deflection.

Determine Camber Requirements

A. Beam Camber

Glulam beams are cambered because the spans are relatively long (compared to a longitudinal deck bridge). The dimension of the dead load deflection is larger and can present a look that the bridge is overloaded and sagging, and so camber counteracts the dead load deflection and the visual appearance of the deflection. The camber must also account for longer term deflection because wood is susceptible to creep. Glulam beams can be cambered in the shop without much difficulty.

[8.12.1]

Glued Laminated timber girders shall be cambered a minimum of two times the dead load deflection at the Service Limit State.

The deflection from the total unfactored dead load is calculated. The camber will be calculated for the interior beams, and the same camber applied to the exterior beams. FWC is included here. Some judgment can be used by the designer, but for aesthetic reasons, generally slight additional extra camber is preferred over not enough camber.

Uniform distributed Dead Load:

$w_\Delta = w_{beam} + w_{deck\_int} + w_{ws\_int} + w_{FWC\_int} + w_{barrier}$

$w_\Delta = 0.138 + 0.134 + 0.278 + 0.100 + 0.028 = 0.678 \text{ kip/ft}$

Point Dead Load: (diaphragm load): $P_\Delta = P_{dc\_int} = 0.362$ kip

$$\Delta_{DL} = \frac{5 \cdot w_\Delta \cdot L^4}{384 \cdot E \cdot I_{prov}} + \frac{P_\Delta \cdot L^3}{48 \cdot E \cdot I_{prov}}$$

$$\Delta_{DL} = \frac{5 \cdot (0.678/12) \cdot (42.0\text{x}12)^4}{384 \cdot 1499.4 \cdot 72{,}374} + \frac{0.362 \cdot (42.0\text{x}12)^3}{48 \cdot 1499.4 \cdot 72{,}374} = 0.446 \text{ in}$$

$\text{Camber} = 2\Delta_{DL} = 2 \cdot 0.446 = 0.89 \text{ in}$

The initial assumption of an additional 0.45 inches of average bituminous thickness assumed early in the example, to accommodate the beam camber, is acceptable.

Investigate Shear Resistance Requirements [8.7]

A. Critical Shear Force Location

For components under shear, shear shall be investigated at a distance away from the face of the support equal to the depth of the component.

When calculating the maximum design shear, the live load shall be placed so as to produce the maximum shear at a distance from the support equal to the lesser of either three times the depth of the component (d_{beam}) or one-quarter of the span (L_{beam}).

Horizontal shear must be checked for wood components. The term "horizontal" shear is typically used in wood design, because a shear failure initiates along the grain. This shear failure is typically along the horizontal axis. The shear stress is equal in magnitude in the vertical direction, but inherent vertical resistance is greater, and so typically does not need to be designed for. AASHTO LRFD C8.7 provides commentary on this.

Bearing has not yet been checked, but the shear calculation typically is not critical for a larger glulam beam. For the location to check shear, it will conservatively be assumed the total bearing length is 12 in.

$$\text{Location to check for shear} = [d_{beam} + {}^{1}/_{2} \cdot L_{bearing}] / L_{beam}$$
$$= [3.90 \text{ ft} + {}^{1}/_{2} \cdot 1.0 \text{ ft}] / 42.0 \text{ ft} = 0.10$$

Check for shear at 10% of the span length away from the support centerlines.

B. Unfactored Shear Forces Acting on the Beam

Dead loads and live loads are positioned at different locations for calculating shear forces in a longitudinal beam.

1. Dead Load Shear Force per Interior Beam

The maximum shear force at the support will be calculated first. As previously shown, the interior beam is the worst case for dead load and so the exterior will not be checked.

$$V_{dc\_max} = V_{beam} + V_{deck\_int} + V_{diaph\_int} + V_{barrier}$$

V_{dc_max} = 2.90 + 2.81 + 0.18 + 0.59 = 6.48 kips

$V_{dw\_max} = V_{ws} + V_{FWC}$

V_{dw_max} = 5.84 + 2.10 = 7.94 kips

Component dead load shear force at a distance "d_{beam}" away from the support face = V_{dc} = 0.80 · 6.48 = 5.18 kips

Wear course dead load shear force at a distance "d_{beam}" away from the support face = V_{dw} = 0.80 · 7.94 = 6.35 kips

2. Live Load Shear Force per Interior Beam

[Eqn. 4.6.2.2.2a-1]

The live load shear is distributed based on an average of: (0.60 of an undistributed wheel load) added to (the distribution specified in Table 4.6.2.2.2a-1). The live load is positioned as specified above.

Check position on beam: lesser of 3 · d_{beam} or L_e / 4

3 · d_{beam} = 3 · 3.90 = 11.70 ft

L_e / 4 = 42.0 / 4 = 10.50 ft

Use 10.50 ft from the centerline of bearing to position the live load.

a. Design Tandem Axle Loads

Design tandem shear forces with the live load placed at a distance away from the support of 10.50 ft = V_{tandem}

$$V_{tandem} = \frac{25 \cdot (31.5 + 27.5)}{42.0} = 35.12 \text{ kips}$$

V_{tandem} = 35.12 kips

b. Design Truck Axle Loads

Design truck shear forces with the live load placed at a distance away from the support of 10.50 ft = V_{truck}

$$V_{truck} = \frac{32 \cdot (31.5 + 17.5)}{42.0} + \frac{8 \cdot (3.5)}{42.0} = 38.00 \text{ kips}$$

V_{truck} = 38.00 kip (controls for live load)

c. Design Lane Load

Design lane load shear forces at a distance away from the support of 10.50 ft = V_{lane}

V_{lane} = 0.50 x 13.44 = 6.72 kips

d. Live Load per Interior Beam
$V_{LL} = 0.50[(0.60\ V_{LU}) + V_{LD}]$; use $g_{int} = 0.60$ from Table 4.6.2.2.2a-1

Shear live loads are multiplied by 0.50 for undistributed wheel loads, V_{LU}
$V_{LL} = 0.50[(0.60 \cdot 0.50(38.00 + 6.72) + (38.00 + 6.72)0.60]$

$V_{LL} = 20.12$ kips

[3.4.1]

C. Factored Shear Force Acting on Beam

1. Load Modifiers
Load modifiers for beam design are shown in the flexure check.

2. Strength I Limit State Load Factors
Use the Strength I Limit State to determine the required shear resistance of the beam.

Impact and skew applicability are the same as for the flexure check.

Specific Strength I Limit State Load Factors are found in AASHTO Tables 3.4.1-1 and 3.4.1-2.

The above result indicates that the design truck and lane load on an interior beam control for shear.

3. Strength I Limit State Shear Force
Strength I Limit State factored shear force, two lanes loaded = $V_{u(m)}$

[Tables 3.4.1-1 and 3.4.1-2]

$$V_{u(m)} = \eta \cdot [1.25 \cdot V_{dc} + 1.50 \cdot V_{dw} + 1.75 \cdot r \cdot [V_{truck} + V_{lane}]]$$

$$V_{u(m)} = 1.0 \cdot [1.25 \cdot (5.18) + 1.50 \cdot (6.35) + 1.75 \cdot 1.0 \cdot [20.12]] = 51.21 \text{ kips}$$

Check Shear Resistance of Beam

A. Factored Shear Resistance
The factored shear force ($V_{u(m)}$) is the required shear resistance of the beam that needs to be compared with the actual factored shear resistance of the beam (V_r).

[Eqns. 8.7-1, 8.7-2] For a rectangular wood section $V_r = \phi_v \cdot F_v \cdot w_{bm} \cdot d_{bm} / 1.5$

[8.5.2.2]

1. Resistance Factor
Shear resistance factor = $\phi_v = 0.75$

2. Adjustment Factors for Reference Design Values
[8.4.4.2] Format conversion factor: $C_{KF} = 2.5/\phi = 2.5/0.75 = 3.33$
[8.4.4.3] Wet Service factor = $C_M = 0.875$
[8.4.4.9] Time effect factor = $C_\lambda = 0.80$

[Eqn. 8.4.4.1-2]

Adjusted design value = $F_v = F_{vxo} \cdot C_{KF} \cdot C_M \cdot C_\lambda$

$F_v = 0.300 \cdot 3.33 \cdot 0.875 \cdot 0.80 = 0.699$ ksi

B. Beam Shear Check

Required beam shear resistance = $V_{u(m)}$

For the beam to meet Strength I Limit State, $V_{r(prov)}$ must equal or exceed $V_{u(m)}$. As determined previously, $V_{u(m)} = 51.21$ kips.

[Eqn. 8.7-2]

$$V_{r(prov)} = \phi_v \cdot \frac{(F_v \cdot w_{bm} \cdot d_{bm})}{1.5} = 0.75 \cdot \frac{(0.699 \cdot 8.5 \cdot 46.75)}{1.5} = 138.88 \text{ kips}$$

$V_{u(m)} = 51.21$ kips $\leq V_{r(prov)} = 138.88$ kips OK

Investigate Compression Resistance Requirements

A. Maximum Support Reactions per Beam

1. Dead Load Reaction Force

The maximum shear/reactions were calculated above in the shear force check of the beam. The calculation below adds in the end diaphragm that was ignored in the shear calculation because it would normally be located within d_{beam} (depth of the component).

$R_{dc\_max} = 2.90 + 2.81 + 0.18 + 0.59 + 0.362 = 6.84$ kips

$R_{dw\_max} = 5.84 + 2.10 = 7.94$ kips

Maximum component dead load reaction force = $R_{dc} = 6.84$ kips

Maximum wear course dead load reaction force = $R_{dw} = 7.94$ kips

2. Live Load Reactions

The maximum live load reactions can be found in Table 3.4.1.2 of this Manual (Chapter 3). R_{truck} governs over R_{tandem}.

The total reaction $R_{Total} = R_{truck} + R_{lane} = 56.0 + 13.40 = 69.4$ kips

For this example $g_{int} = 0.60$ as calculated for flexure will be used. The distribution factor for shear was less than this and so is not used here. A minimum of half a design truck should typically be used. The 0.60 for flexure is larger than half a truck (or one wheel line) on one beam and so is sufficient in this case, and most similar cases. AASHTO LRFD does not provide live load distribution factors specifically for bearing of wood beams. The designer should evaluate axle load locations on the span for individual designs to make certain that the distribution factor used in design adequately determines the reaction on the bearing.

$R_{LL} = 69.4 \cdot (0.60) = 41.64$ kips

B. Factored Support Reaction Forces Acting on Beam

[3.4.1]

Strength I Limit State maximum factored support reaction due to two lanes loaded case = $P_{u(m)}$

[Tables 3.4.1-1 and 3.4.1-2]

$$P_{u(m)} = \eta \cdot [1.25 \cdot R_{dc} + 1.50 \cdot R_{dw} + 1.75 \cdot r \cdot (R_{truck} + R_{lane})]$$

$$P_{u(m)} = 1.0 \cdot [1.25 \cdot (6.84) + 1.50 \cdot (7.94) + 1.75 \cdot 1.0 \cdot (41.64)] = 93.33 \text{ kips}$$

Check Compression Resistance of Beam

A. Factored Bearing Resistance

The maximum factored support reaction $P_{u(m)}$ is the required compression resistance perpendicular to the grain of the beam that needs to be compared with the actual factored compression resistance perpendicular to the grain of the beam (P_r).

[Eqns. 8.8.1-1, 8.8.3-1]

$P_r = \phi_{cperp} \cdot F_{cp} \cdot A_b \cdot C_b$

1. Resistance Factor

[8.5.2.2]

Compression perpendicular to grain resistance factor = ϕ_{cperp} = 0.90

2. Adjustment Factors for Reference Design Values

[8.4.4.2] Format conversion factor: $C_{KF} = 2.1/\phi = 2.1/0.90 = 2.33$

[8.4.4.3] Wet Service factor = C_M = 0.53

[8.4.4.9] Time effect factor = C_λ = 0.80

[Eqn. 8.4.4.1-5]

Adjusted design value = $F_{cp} = F_{cpo} \cdot C_{KF} \cdot C_M \cdot C_\lambda$

$F_{cp} = 0.740 \cdot 2.33 \cdot 0.53 \cdot 0.80 = 0.731$ ksi

3. Beam Bearing Dimensions

For this calculation, a bearing length, L_b, of 18 inches will be tried.

Bearing width = $b_b = w_{beam}$ = 8.5 in

Bearing Area = $A_b = L_b \times b_b = 18.0 \times 8.5 = 153.0$ in^2

4. Bearing Adjustment Factor

[Table 8.8.3-1]

Adjustment Factor for Bearing = C_b

$L_b = 18.0 \text{ in} \geq 6.0 \text{ in}$ $\quad C_b = 1.00$

B. Beam Bearing Resistance Check

Required beam compression resistance = $P_{u(m)}$ = 93.33 kips

For the beam to meet Strength I Limit State, provided compression resistance perpendicular to grain = $P_{r(prov)}$ must equal or exceed $P_{u(m)}$.

[Eqn. 8.8.3-1]

$P_{r(prov)} = \phi_{cperp} \cdot F_{cp} \cdot A_b \cdot C_b = 0.9 \cdot 0.731 \cdot 153.0 \cdot 1.0 = 100.66$ kips

$P_{u(m)} = 93.33 \text{ kips} \leq P_{r(prov)} = 100.66$ kips <u>OK</u>

As stated at the beginning of Article 8.7.3, the bearing pad design is not a part of this example, so it will be assumed that the compression in the wood governs the bearing area size.

8.7.4 Transverse Deck Design Examples

The transverse deck design examples presented here go through the design of two wood deck types that can be used on top of the glulam beams designed in Article 8.7.3. Either of these deck types, transverse spike laminated or transverse glued laminated, could be used on the glulam beams to support the road surface. The final selection is up to the owner and designer, and might be influenced by availability and cost. If cost is the main determining factor, the final decision on type can be made after a design is done for each to determine which is most economical. Both of these deck types are available and used in Minnesota.

[9.9.2]

AASHTO LRFD Section 9 covers requirements for Decks and Deck Systems, including wood decks in 9.9. The nominal thickness of wood decks other than plank decks shall not be less than 6.0 in.

[9.9.4.3.2]

AASHTO LRFD requires a wear course on wood decks, and recommends bituminous. To prevent continual cracking of the bituminous and constant maintenance, bridge decks should consist of interconnected deck panels. Various options exist for connecting panels, but for these examples the panels are attached to each other using vertical spikes through ship lap joints along with longitudinal stiffener beams also called spreader beams. The deck panel depth and spreader beam sizes are based on deflection limits as well as strength considerations. The spreader beams enable the deck to act as a single unit under deflection and to consider it designed as interconnected in accordance with AASHTO LRFD.

[9.9.4.2]

Proper deck tie downs are important for a positive connection to the support for the deck, and to prevent excessive deflections that can occur when the deck is not securely fastened to each support. In the case of the transverse decks here, the timber beams are the supports. It is recommended to attach the deck to the beams with lag screws to stabilize the deck and prevent excess cracking in the bituminous wear course. The designer should determine lag bolt spacing for specific applications, but as a guide they are commonly spaced at 2 feet in the direction of the beams. In these examples the bituminous tapers down to 2 inches minimum, and so in this case the lag screw heads should be countersunk into the deck. It is best to shop drill and countersink, so that the panel wood is treated after countersinking. The wide beams in this example provide some tolerance for assembly on the beams in the field.

[4.6.2.1.1]

The deck span under investigation is an "equivalent" strip which spans from one beam to another beam. The deck overhang outside of the exterior beam should always be investigated. The deck cantilever does not need a complete analysis in this example because the exterior glulam beams in Article 8.7.3 were positioned so that the deck overhang would not govern the deck design. Applying AASHTO LRFD 3.6.1.3.1 to this case, a wheel load along the curb will occur directly over the exterior beam, and not on the deck overhang.

Transverse Spike Laminated Deck [9.9.6]

A. Material and Design Parameters

The dimension annotations used throughout this design example are similar to a longitudinal deck. The vertical dimension of a member is considered its depth. The transverse and longitudinal measurements of a member are considered its width and length, respectively, considering the length to be in the direction transverse to the road centerline for a transverse deck. These dimension annotations are consistent with Figure 8.3-1 of the *2014 AASHTO LRFD Bridge Design Specifications*, except for sawn lumber descriptive names. The letter notations will be used in this example (b, d, etc.).

[Figure 8.3-1]

Nominal dimensions for sawn lumber are always used for dead load calculations.

1. Supporting Beams

Length of the supporting members (bearing lengths for the deck on the beams) = b_{length} = 8.5 in, determined in the previous example.

[8.4.1.2.2]

For glulam beams, the timber dimensions stated shall be taken as the actual net dimensions.

[9.9.8]

2. Bituminous Wearing Surface

MnDOT uses a 2% cross slope whenever practicable. In this case, minimum 2 inches at edge of roadway (face of curb) produces 6 inches at centerline. Because the deck spans are short, the thickness occurring within the span is used (not an average of the full deck width), and the largest force effect would be near the centerline of roadway. In addition, as described in Article 8.7.3 of this manual, the wearing surface will be thicker at the end of the deck due to beam camber. The thickness for deck design is then, d_{ws} = 6.9 in.

3. Curb and Railing [TL-4 Glulam Timber Rail with Curb]

The timber barrier design is not a part of the design examples. The dimensions were used for weight considerations in Article 8.7.3. For this example, as described above, the deck overhang does not need to be analyzed and the curb and railing do not affect the deck spanning from beam to beam.

[8.4.1.1, 9.9.2]

4. Deck Laminates

Assumed depth of timber deck panel laminates = d_{lam} = 5.75 in
Assumed width of timber deck panel laminates = b_{lam} = 3.75 in

[8.4.1.1.2]

Visually-graded transverse deck panel lumber is supplied rough sawn and typically surfaced on one side and one edge (S1S1E) to fabricate transverse deck panels to the specified dimensions. For nominal 4 in x 6 in lumber S1S1E reduces both the depth and width of an individual laminate by about 1/4 in. Nominal dimensions are used for dead loads, and surfaced dimensions are used in the section properties for strength.

[4.6.2.1.6]

5. Span Lengths

In this case, MnDOT uses the effective span, or design span, as center to center of the deck bearing length on each beam, which is also center to center of beams, as stated in AASHTO LRFD.

Effective design span length for the deck panels = L_e = 5.0 ft

6. Unit Weights and Moisture Content

Type of deck panel wood material = Douglas Fir-Larch (No.2)

[Table 3.5.1-1]
[MnDOT Table 3.3.1]
[MnDOT 3.3]

Unit weight of soft-wood = γ_{DFL} = 0.050 kcf
Unit weight of bituminous wearing surface = γ_{ws} = 0.150 kcf
Standard MnDOT practice is to apply a future wearing course of 20 psf.

[8.4.1.1.3]

Moisture content (MC) of timber at the time of installation shall not exceed 19.0%

MnDOT designs for in-service wet-use only which is a MC of greater than 19% for sawn lumber.

7. Douglas Fir-Larch Deck (No. 2) Strength Properties

[Table 8.4.1.1.4-1]

Reference Design Value for flexure = F_{bo} = 0.90 ksi
Reference Design Value for compression perpendicular to grain = F_{cpo} = 0.625 ksi
Reference Design Value for shear parallel to grain (horizontal) = F_{vo} = 0.18 ksi
Modulus of elasticity = E_o = 1600 ksi

Select the Basic Configuration

The bridge deck consists of interconnected deck panels, which are oriented perpendicular to traffic. The laminates of each panel will be connected using horizontal spikes. The panels are attached to each other using vertical spikes through ship lap joints along with longitudinal stiffener beams (also called spreader beams). The deck panel depth and spreader beam sizes are based on deflection limits as well as strength considerations. The spreader beams enable the deck to act as a single unit under deflection, and to consider it interconnected by AASHTO LRFD. For a visual representation of the transverse deck on the glulam beams as well as the spreader beams, see Figure 8.7.3.1. The connections in the shiplap joints are similar to that shown in various figures in Article 8.7.1, except with a transverse deck the joints are also transverse as that is the direction of the panels.

A. Deck Panel Sizes

For shipping purposes, transverse deck panels are typically half the width of longitudinal panels. The dimensions of the panels at the beginning and end of deck are adjusted so that the total deck length matches the length of the beams.

The dimension lumber used in transverse decks typically needs to be spliced because of the longer lengths for the smaller cross-sectional sizes. Splices should be laid out to occur over interior beams, but splices should not occur in consecutive planks. The splices should be spaced every third or fourth plank.

B. Spreader Beam Dimensions

[9.9.4.3.2]

Interconnection of panels may be made with mechanical fasteners, splines, dowels, or stiffener beams. This example will use stiffener beams, or spreader beams, along with shiplap joints similar to the longitudinal deck in Article 8.7.1. For a transverse deck, the spreader beam is to be placed longitudinally along the bridge at the center of each deck span. The following rough sawn spreader beam dimensions will be verified.

Width of spreader beams = b_{spdr} = 5 in

Depth of spreader beams = d_{spdr} = 5 in

[9.9.4.3]

Minimum allowed rigidity of the spreader beams = EI_{min} = 80,000 kip·in^2

Required moment of inertia of spreader beams to accommodate the specified rigidity for a given species and grade of wood = I_{min}.

For Douglas Fir-Larch No. 1 Posts & Timber, E_o =1600 ksi

Adjusted spreader beam modulus of elasticity = E

[8.4.4.3]

[Table 8.4.4.3-1]

Wet Service factor for Modulus of Elasticity = C_M

For nominal thickness > 4.0 in, C_M = 1.0

[Eqn. 8.4.4.1-6]

Adjusted design value = E = E_o x C_M

E = 1600 ksi x 1.0 = 1600 ksi

$$I_{min} = \frac{80{,}000}{E} = \frac{80{,}000}{1600} = 50.0 \text{ in}^4$$

Check spreader beam dimensions.

$$I_{spdr} = \frac{1}{12} \cdot b_{spdr} \cdot d_{spdr}^3$$

$$I_{spdr} = \frac{1}{12} \cdot 5 \cdot 5^3 = 52.1 \text{ in}^4 \geq I_{min} = 50.0 \text{ in}^4 \qquad \text{(OK)}$$

Determine Dead and Live Load Reactions, Shear Forces, and Bending Moments

The dead and live load shear, reaction and bending moment results can be determined using a basic structural analysis computer program, or using the standard beam formulas found in AISC 14th Edition LRFD Manual. MnDOT uses simplified analysis models that are permitted by AASHTO LRFD.

[4.6.2.1.6]

In the calculation of force effects using equivalent strips, the axle wheel loads may be considered point loads or patch loads, and the beams considered simply supported or continuous, as appropriate.

Modelling the axle wheel loads as patch loads will not have a large effect with the given beam spacing, and so for the calculations below the wheel loads on the axles are conservatively modelled as point loads.

[3.6.1.3.3]

Per AASHTO LRFD the design load in the design of decks is always an axle load; single wheel loads should not be considered. In addition, when using the approximate strip method for spans primarily in the transverse direction, only the axles for the design truck or the axles for the design tandem (whichever results in the largest effect) shall be applied to deck in determining live load force effects.

A. Analysis Models

In determining the maximum deck forces, MnDOT uses a variation of beam models for the deck strip as follows:

1) The maximum shear forces and reactions are determined by modeling the deck as a continuous beam. Moving live loads are then placed at various locations along the span, to produce the maximum shear and reactions. This method of analysis allows the effects of adjacent spans to be investigated. A two span continuous beam is conservatively assumed for simplicity.

2) The maximum positive bending moments (tension on deck bottom) and deflections are determined by considering the deck as a single simply-supported span between beams.
3) The maximum negative bending moments (tension on deck top) are determined by considering the deck as a single fixed-fixed span between beams, with fixed ends. Looking at the beam formulas in AISC 14th Edition LRFD Manual, it can be seen that this case will not govern, and so it will not be calculated here.

B. Dead Loads per Unit Strip (1 ft)

The units for the dead load results are given in kips for a 1 ft wide transverse strip.

1. Dead Loads per foot (these units could also be given as kips per square foot).

Weight of deck = $w_{deck} = \gamma_{DFL} \cdot d_{lam} = 0.050 \cdot 6/12 = 0.025$ klf/ft

Weight of wearing course = $w_{ws} = \gamma_{ws} \cdot d_{ws}$

$\gamma_{ws} \cdot d_{ws} = 0.150 \cdot 6.9/12 = 0.086$ klf/ft

Weight of future wearing course = $w_{FWC} = 0.020$ klf/ft

2. Spreader beam point loads on 1 ft wide strip.

Area of spreader beam = $A_{spdr} = d_{spdr} \cdot b_{spdr} = (5 \cdot 5)/144 = 0.174 \text{ ft}^2$

Spreader beam load = $P_{spdr} = \gamma_{DFL} \cdot A_{spdr} = 0.050 \cdot 0.174 = 0.009$ kips/ft

[AISC 14th p. 3-213]

C. Dead Load Bending Moments per Unit Strip (1 ft)

Maximum bending moment due to deck weight = M_{deck}

$$M_{deck} = \frac{w_{deck} \cdot (L_e)^2}{8} = \frac{0.025 \cdot 5.0^2}{8} = 0.078 \ \frac{\text{kip-ft}}{\text{ft}}$$

Maximum bending moment due to wearing surface weight = M_{ws}

$$M_{ws} = \frac{w_{ws} \cdot (L_e)^2}{8} = \frac{0.086 \cdot 5.0^2}{8} = 0.269 \ \frac{\text{kip-ft}}{\text{ft}}$$

Maximum bending moment due to future wearing surface weight = M_{FWC}

$$M_{FWC} = \frac{w_{FWC} \cdot (L_e)^2}{8} = \frac{0.020 \cdot 5.0^2}{8} = 0.063 \ \frac{\text{kip-ft}}{\text{ft}}$$

Maximum bending moment due to spreader beam weight = M_{spdr}

$$M_{spdr} = \frac{P_{spdr} \cdot L_e}{4} = \frac{0.009 \cdot 5.0}{4} = 0.011 \ \frac{\text{kip-ft}}{\text{ft}}$$

Maximum bending moment due to bridge component dead loads = M_{dc}

$M_{dc} = M_{deck} + M_{spdr}$

$M_{dc} = 0.078 + 0.011 = 0.089$ kip·ft/ft

Maximum bending moments due to wearing course loads = M_{dw}

$M_{dw} = M_{ws} + M_{FWC}$

$M_{dw} = 0.269 + 0.063 = 0.332$ kip·ft/ft

[3.6.1.2]

D. Live Load Moments per Axle

The live load bending moment will be calculated per axle and later converted to a per unit strip (1 ft) format.

1. Design Truck Axle Loads

[3.6.1.2.2]

Point load on one deck span from design truck axle = P_{truck} = 16 kips

Maximum bending moment due to design truck axle load = M_{truck}

$$M_{truck} = \frac{P_{truck} \cdot L_e}{4} = \frac{16.0 \cdot 5.0}{4} = 20.000 \text{ kip-ft}$$

2. Design Tandem Axle Loads

[3.6.1.2.3]

Point load of design tandem axle, one deck span = P_{tandem} = 12.5 kips

AASHTO Table A4-1 can be used in the design of concrete decks, but includes impact so is not applicable to timber. However, the table footnotes indicate that specifically calculating the tandem is not necessary. A calculation can be done that shows the heavier single wheel load from the design truck on the smaller area of deck is the controlling case. Therefore, the tandem effect is not calculated for this example.

[4.6.2.1]

E. Modification of Live Load Bending Moment

1. Convert Live Load Bending Moment to Per Unit Strip

The live load bending moment calculated above (M_{truck}) will now be distributed over the transverse equivalent strip width, and converted to a per foot basis.

[Table 4.6.2.1.3-1]

For a structural deck thickness h= 5.75 in, the equivalent strip width = E_s = 4.0h + 40.0 = 63.0 in

$$M_{truck} = M_{truck} \cdot \frac{1}{E_s} = 20.000 \cdot \frac{12}{63.0} = 3.810 \text{ kip-ft}$$

2. Multiple Presence Factors

[3.6.1.1.2, 4.6.2.1]

The multiple presence factor is to be used in conjunction with the equivalent strip widths of 4.6.2.1.

[3.6.1.1.1] Maximum number of traffic lanes on the deck = N_L

$$N_L = \frac{b_{rd}}{12 \frac{ft}{lane}} = \frac{32}{12} = 2.67 \cong 2 \text{ lanes}$$

[Table 3.6.1.1.2-1] For one lane loaded, the multiple presence factor = m = 1.20
For two lanes loaded, the multiple presence factor = m = 1.00

[C3.6.1.1.2] This design example is for an unspecified ADTT, although AASHTO LRFD recommends limitations on the use of wood deck types based on ADTT. If these recommendations are adhered to, AASHTO LRFD also allows reduction of force effects based on ADTT because the multiple presence factors were developed on the basis of an ADTT of 5000 trucks in one direction. A reduction of 5% to 10% may be applied if the ADTT is expected to be below specified limits during the life of the bridge. If the ADTT level is confirmed, the reduction may be applied subject to the judgment of the designer and approved by the State Bridge Design Engineer.

[AISC 14th p. 3-223]

F. Shear Force and Support Reactions

As described above, shear force and reactions are calculated conservatively assuming a two span continuous beam. Axle tire loads can transversely occur at a distance as short as 4 ft apart if in two separate lanes, and if the two lanes are centered on a beam the axle tire loads are then 2 ft either side of a beam. This 2 lane case will need to be checked against the one lane case.

[3.6.1.2.1]
[3.6.1.3.1]

The axle tire placement for the one lane and two lane cases are illustrated in Figures 8.7.4.1 and 8.7.4.2.

The results are converted to a per foot basis and shown in Table 8.7.4.1. The live load force effects are shown for one and two lanes, with the appropriate multiple presence factor, m, applied.

G. Summary of Maximum Shear Force, Reaction and Bending Moment Results

Table 8.7.4.1

Unfactored Load Case	Maximum Positive Bending Moment (kip·ft/ft)	Maximum Shear Force (kips/ft)	Maximum Support Reaction (kips/ft)
Component Dead Load (DC)	0.089	0.084	0.169
Wearing Course Dead Load (DW)	0.332	0.331	0.663
Live Loads			
Design Truck (1 lane, m=1.20)	4.572	2.775	3.113
Design Truck (2 lane, m=1.00)	3.810	2.414	4.827

Flexural Check of Deck Panel

H. Factored Bending Moment per Unit Strip (1 ft)

1. Load Modifiers

Standard MnDOT Load Modifiers are summarized in Table 3.2.1 of this manual.

For timber bridges η_D = 1.0. MnDOT considers spike laminated decks to have a conventional level of redundancy and uses η_R = 1.0. This example bridge is assumed to have a design ADT of over 500 for η_I = 1.0.

[1.3.2]

2. Strength I Limit State Load Factors

[3.4.1] Use the Strength I Limit State to determine the required resistance for the deck panels.

[3.6.2.3] Impact factor need not be applied to wood components.

[4.6.2.3] Skew factor (bridge is not skewed thus 1.0) = r = 1.0

Specific Strength I Limit State Load Factors are found in AASHTO Tables 3.4.1-1 and 3.4.1-2.

The earlier analysis indicated that the truck load controls the bending moment of the deck panels. Therefore, use the truck load in determining the critical live load bending moment acting on the deck panels.

[Tables 3.4.1-1 and 3.4.1-2]

3. Strength I Limit State Bending Moment per Unit Strip (1 ft)

Factored bending moment for the one lane loaded case = $M_{u(m)}$

$$M_{u(m)} = \eta \cdot [1.25 \cdot M_{dc} + 1.50 \cdot M_{dw} + 1.75 \cdot r \cdot (M_{truck} + M_{lane})]$$

$$M_{u(m)} = 1.0 \cdot [1.25 \cdot 0.089 + 1.50 \cdot 0.332 + 1.75 \cdot 1.0 \cdot (4.572)] = 8.610 \frac{\text{kip-ft}}{\text{ft}}$$

Check Flexural Resistance of Deck Panel

A. Factored Flexural Resistance

The factored bending moment ($M_{u(m)}$) is the required flexural resistance of the deck that needs to be compared with the actual factored flexural resistance of the deck panel (M_r).

[8.6.2]

For a rectangular wood section $M_r = \phi_f \cdot F_b \cdot S_{req} \cdot C_L$.

1. Resistance Factor

[8.5.2.2]

Flexural resistance factor = ϕ_f = 0.85

2. Stability Factor

[8.6.2]

Stability factor for sawn dimension lumber in flexure = C_L

Laminated deck planks are fully braced. C_L = 1.0

[8.4.4.4]
[Table 8.4.4.4-1]

3. Adjustment Factors for Reference Design Value

Size effect factor for sawn dimension lumber in flexure = C_F

d_{lam} = 6 in

b_{lam} = 4 in

C_F = 1.30

[8.4.4.2]

Format conversion factor for component in flexure = C_{KF}

$C_{KF} = 2.5/\phi = 2.5/0.85 = 2.94$

[8.4.4.3]
[Table 8.4.4.3-1]

Wet Service factor for sawn dimension lumber in flexure = C_M

Check $F_{bo} \cdot C_F$: $0.900 \cdot 1.30 = 1.17 > 1.15$

C_M = 0.85

[8.4.4.7]

Incising Factor for dimension lumber in flexure = C_i

Douglas Fir-Larch requires incising for penetration of treatment.

[Table 8.4.4.7-1]

C_i = 0.80

[8.4.4.8]
[Table 8.4.4.8-1]

Deck factor for a spike-laminated deck in flexure = C_d

C_d = 1.15

[8.4.4.9]
[Table 8.4.4.9-1]

Time effect factor for Strength I Limit State = C_λ

C_λ = 0.80

[Eqn. 8.4.4.1-1]

Adjusted design value = $F_b = F_{bo} \cdot C_{KF} \cdot C_M \cdot C_F \cdot C_i \cdot C_d \cdot C_\lambda$

F_b = 0.900 x 2.94 x 0.85 x 1.30 x 0.80 x 1.15 x 0.80 = 2.152 ksi

4. Required Section Modulus

The section modulus is dependent on the deck panel depth. The section modulus is used in Part B to solve for the deck panel depth.

B. Required Deck Panel Depth

Required deck flexural resistance = $M_{n(req)}$

For the deck panel depth to meet Strength I Limit State, M_r must equal (or exceed) $M_{u(m)}$, where $M_r = \phi M_{n(req)}$. Therefore, set $\phi M_{n(req)} = M_{u(m)}$.

$$M_{n(req)} = \frac{M_{u(m)}}{\phi_f} = \frac{8.610}{0.85} = 10.129 \text{ kip - ft}$$

Required Section Modulus of one foot of deck width = S_{req}

Required depth of deck laminates (panel) = d_{req}

$$S_{req} = \frac{12 \text{ in} \cdot d_{req}^{\ 2}}{6}$$

$M_{n(req)} = F_b \cdot S_{req} \cdot C_L$ with $C_L = 1.0$

Substituting terms gives

$$d_{req} = \sqrt{\frac{6 \cdot M_{n(req)}}{12 \cdot F_b \cdot C_L}} = \sqrt{\frac{6 \cdot 10.129 \cdot 12}{12 \cdot 2.152 \cdot 1.0}} = 5.31 \text{ in} \leq 5.75 \text{ in}$$ <u>OK</u>

The required deck panel depth (5.31 inches) indicates that the originally assumed deck depth (5.75 inches actual) can be used based on flexure. However, it is not uncommon that a deeper section could be required to satisfy the shear requirement, so that is checked next.

Investigate Shear Resistance Requirements for Deck Panel [8.7, 9.9.3.2]

A. Critical Shear Force Location

In transverse decks, maximum shear shall be computed at a distance from the support equal to the depth of the deck (d_{lam}). The tire footprint shall be located adjacent to, and on the span side of, the point on the span where maximum force effect is sought.

Location to check for shear = $(d_{lam} + {}^1/_2 \cdot b_{length}) / L_e$

= $(0.48 \text{ ft} + {}^1/_2 \cdot 0.71 \text{ ft}) / 5.0 \text{ ft}$

Check for shear at about 17% of span length away from the center of support, or 0.83 ft.

Horizontal shear must be checked for wood components. The term "horizontal" shear is typically used in wood design, because a shear failure initiates along the grain. This shear failure is typically along the horizontal axis. The shear stress is equal in magnitude in the vertical direction, but inherent vertical resistance is greater, and so typically does not need to be designed for. AASHTO LRFD C8.7 provides commentary.

B. Unfactored Shear Acting on the Deck per Unit Strip (1 ft)

For the uniformly distributed loads, the shear forces are less than the maximums listed in Table 8.7.4.1. The results given below are not the maximum shear forces on the deck (except for the design truck). Rather, they are the values taken at the appropriate distance "d_{lam}" from the critical support face. The following shear forces were taken at the location 17% of span length from center support.

1. Dead Load Shear Force

Component dead load shear force at a distance "d_{lam}" away from the support face = V_{dc} = 0.059 kips

Wear course dead load shear force at a distance "d_{lam}" away from the support face = V_{dw} = 0.232 kips

2. Live Load Shear Forces

Only the design truck is shown below. From the earlier results, this is the load case that gives the maximum shear force. One lane loaded with the multiple presence factor applied produces the maximum live load design shear forces as explained below.

a. Design Truck Load One Lane Case

[3.6.1.2.5] Truck tire contact area consists of a 20 inch width. Placing the 20 inch width according to 9.9.3.2 results in the following on one side of a support (beam) for the one lane case.

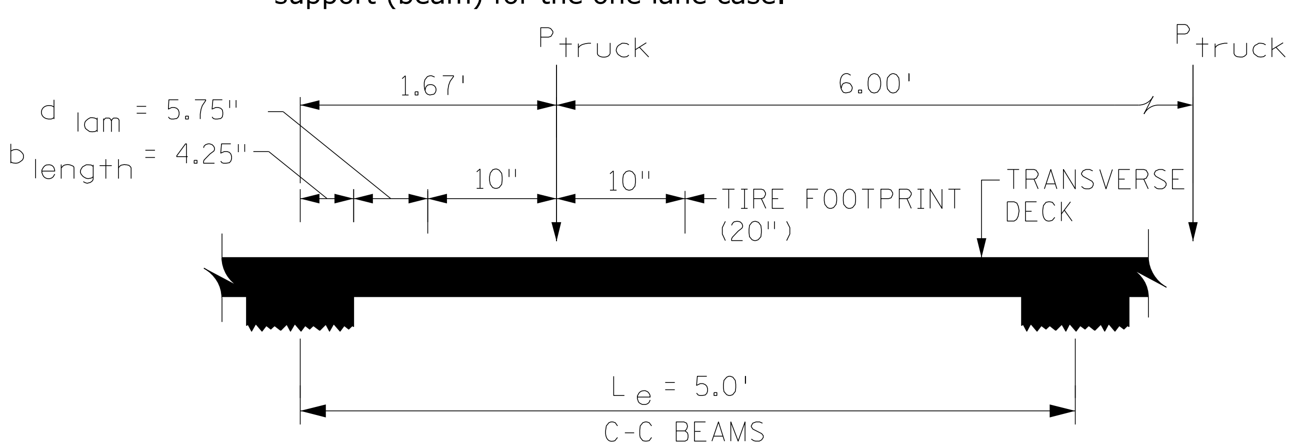


Figure 8.7.4.1

b. Design Truck Load Two Lane Case

[3.6.1.3.1]

For two adjacent loaded lanes, the closest another wheel can be placed on the opposite side of the support is 4.00 ft away, which is 2.33 ft from the support. If the minimum 4.00 ft space between wheels is centered on the support, the distance to the wheel on each side of the support is then 2.00 ft which satisfies the "d_{lam}" minimum (1.67 ft), and is what produces the maximum force effects shown in Table 8.7.4.1.

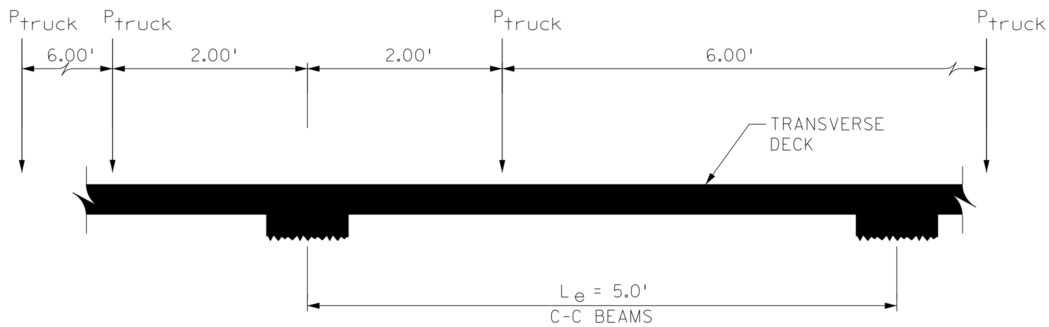


Figure 8.7.4.2

Although the maximum calculated shear forces at a distance "d_{lam}" away from the support for the design truck is governed by the case of two adjacent loaded lanes and is equal to the maximum = V_{truck} = 2.414 kips, with the multiple presence factor applied the one lane loaded case governs the design shear as shown in Table 8.7.4.1.

C. Factored Shear Acting on the Deck Panels per Unit Strip (1 ft)

1. Load Modifiers

Load modifiers for deck design are shown in the flexure check.

2. Strength I Limit State Load Factors

Use the Strength I Limit State to determine the required shear resistance of the deck.

[3.4.1]

Impact and skew applicability are the same as for the flexure check.

Specific Strength I Limit State Load Factors are found in AASHTO Tables 3.4.1-1 and 3.4.1-2.

The above results indicate that a single lane loaded with the design truck controls for shear.

3. Strength I Limit State Shear Force

Strength I Limit State factored shear force, one lane loaded = $V_{u(m)}$

[Tables 3.4.1-1 and 3.4.1-2]

$$V_{u(m)} = \eta \cdot [1.25 \cdot V_{dc} + 1.50 \cdot V_{dw} + 1.75 \cdot r \cdot [V_{truck(m)} + V_{lane(m)}]]$$

$$V_{u(m)} = 1.0 \cdot [1.25 \cdot (0.059) + 1.50 \cdot 0.232) + 1.75 \cdot 1.0 \cdot [2.775]] = 5.28 \text{ kips}$$

Check Shear Resistance of Deck Panel

A. Factored Shear Resistance

The factored shear force $V_{u(m)}$ is the required shear resistance of the deck that needs to be compared with the actual factored shear resistance of the deck (V_r).

[Eqns. 8.7-1, 8.7-2]

For a rectangular wood section $V_r = \phi_v \cdot F_v \cdot b \cdot d_{lam} /1.5$

[8.5.2.2]

1. Resistance Factor

Shear resistance factor = ϕ_v = 0.75

2. Adjustment Factors for Reference Design Values

[8.4.4.2]

Format conversion factor: $C_{KF} = 2.5/\phi = 2.5/0.75 = 3.33$

[8.4.4.3]

Wet Service factor = C_M = 0.97

[8.4.4.7]

Incising Factor for dimension lumber in flexure (F_{bo}) = C_i

Douglas Fir-Larch requires incising for penetration of treatment.

[Table 8.4.4.7-1]

C_i = 0.80

[8.4.4.9]

Time effect factor = C_λ = 0.80

[Eqn. 8.4.4.1-2]

Adjusted design value = $F_v = F_{vo} \cdot C_{KF} \cdot C_M \cdot C_i \cdot C_\lambda$

$F_v = 0.18 \cdot 3.33 \cdot 0.97 \cdot 0.80 \cdot 0.80 = 0.372$ ksi

B. Deck Panel Shear Check

Required deck shear resistance = $V_{u(m)}$

For the deck to meet Strength I Limit State, $V_{r(prov)}$ must equal or exceed $V_{u(m)}$. As determined previously, $V_{u(m)}$ = 5.28 kips.

[Eqn. 8.7-2]

$$V_{r(prov)} = \phi_v \cdot \frac{(F_v \cdot b \cdot d_{lam})}{1.5} = 0.75 \cdot \frac{(0.372 \cdot 12 \cdot 5.75)}{1.5} = 12.83 \text{ kips}$$

$V_{u(m)} = 5.28 \text{ kips} \leq V_{r(prov)} = 12.83$ kips <u>OK</u>

Check Compression Resistance

Compression, or bearing of the deck on the beams, should be computed in accordance with the provisions of AASHTO LRFD for non-standard situations that provide a very narrow bearing area for the transverse deck. For this example, compression bearing on the glued laminated beams is not close to governing the design of the deck panel and so the calculation is not shown here. It usually will not govern a transverse deck design for a bridge of standard configuration. A bearing resistance calculation check for the longitudinal deck (on the pier caps) is shown in Article 8.7.1.

Investigate Deflection Requirements [9.9.3.3]

A. Deck Live Load Deflection with Current Deck Parameters

The final check for the transverse deck design to meet AASHTO LRFD is the deformation, or deflection, calculation. The design truck will have the most severe effect, and that is used for checking the transverse deck deflection.

[3.6.1.3.3]

When using the approximate strip method for spans primarily in the transverse direction, only the axles for the design truck or the design tandem (whichever results in the largest effect) shall be applied to the deck in determining live load force effects.

Deflections are to be calculated using Service I Limit State.

Calculate deck deflections for a transverse interconnected deck using a per foot width approach. This approach can be used on a spike laminated deck with shiplap joints and a spreader beam.

[2.5.2.6.2]

[C2.5.2.6.2]

In the absence of other criteria, the recommended deflection limit in AASHTO LRFD for wood construction is span/425, which will be used here. The designer and owner should determine if a more restrictive criteria is justified, such as to reduce bituminous wearing course cracking and maintenance.

As of note, if a plank deck or a non-interconnected panel deck is being analyzed, a different approach likely is required for the live load distribution, and an additional limitation of 0.10 inches relative deflection between adjacent edges is also required.

1. Deck Stiffness

Moment of inertia of one foot width of deck panels = I_{prov}

$$I_{prov} = \frac{1}{12} \cdot b \cdot d_{lam}^3 = \frac{1}{12} \cdot 12 \cdot (5.75)^3 = 190 \ \text{in}^4$$

Adjusted deck panel modulus of elasticity = E

[8.4.4.3]

[Table 8.4.4.3-1]

Wet Service Factor for Modulus of Elasticity = C_M

$C_M = 0.90$

[Table 8.4.4.7-1]

Incising Factor for Modulus of Elasticity = C_i

$C_i = 0.95$

[Eqn. 8.4.4.1-6]

$E = E_o \cdot C_M \cdot C_i$ = 1600 ksi x 0.90 x 0.95 = 1368.0 ksi

2. Loads per Unit Strip Width (1 ft)

Design truck load on deck span used for deflection calculations = $P_{\Delta truck.}$

Similar to calculations for the maximum positive bending moments, deflections are determined by considering the deck as a single simply-supported span between beams. Therefore, the point load on one deck span from design truck axle = P_{truck} = 16 kips.

P_{truck} expressed as per foot width = $P_{\Delta truck}$:

$P_{\Delta truck} = P_{truck} \cdot 12 \text{ in} / E_S = P_{truck} \cdot 12 \text{ in} / 63 \text{ in}$

$P_{\Delta truck} = 16 \cdot 0.191 = 3.05$ kips/ft

[Table 3.6.1.1.2-1]

One lane loaded governs, the multiple presence factor = *m* = 1.20

[3.6.1.3]
[AISC 14th p. 3-215]

3. Live Load Deflection Calculations

Deflection at deck midspan due to design truck load axle load = Δ_{truck}

$$\Delta_{truck} = \frac{m \cdot P_{\Delta truck} \cdot L_e^3}{48 \cdot E \cdot I_{prov}} = \frac{1.20 \cdot 3.05 \cdot (5.00 \cdot 12)^3}{48 \cdot 1368.0 \cdot 190} = 0.06 \text{ in}$$

The maximum deflection live load deflection = Δ_{truck} = 0.06 in

[2.5.2.6.2]

Live load deflection limit at deck midspan = Δ_{max}

$\Delta_{max} = L_e / 425 = 5.0 \text{ ft} \cdot 12 \text{ in} / 425 \cdot \text{ft} = 0.14$ in

$\Delta = 0.06 \text{ in} \le \Delta_{max} = 0.14$ in OK

Deflections are also okay. Thus, the initial 6 inch nominal deck panel depth and grade are adequate for the design.

Transverse Glued Laminated Deck [9.9.4]

A. Material and Design Parameters

The dimension annotations used throughout this design example are similar to that for the transverse spike laminated deck. The vertical dimension of a member is considered its depth. The transverse and longitudinal measurements of a member are considered its width and length, respectively, considering the length to be in the direction transverse to the road centerline for a transverse deck. These dimension annotations are consistent with Figure 8.3-1 of the *2014 AASHTO LRFD Bridge Design Specifications* letter notations for sawn lumber (but not the descriptive names). The glulam definitions in Figure 8.3-1 are set up for a glulam beam, and are not applicable to a transverse glulam deck panel. The sawn lumber letter notations will be used in this example (b, d, etc.).

[Figure 8.3-1]

[8.4.1.2.2] Dimensions stated for glued laminated timber shall be taken as the actual net dimensions.

1. Supporting Beams

Length of the supporting members (bearing lengths for the deck on the beams) = b_{length} = 8.5 in, determined in the beam design example. The dimensions stated shall be taken as the actual net dimensions.

[9.9.8] **2. Bituminous Wearing Course**

MnDOT uses a 2% cross slope whenever practicable. In this case, minimum 2 in at edge of roadway (face of curb) produces 6 in at centerline. Because the deck spans are short, the thickness occurring within the span is used (not an average of the full deck width), and the largest force effect would be near the centerline of roadway. In addition, the wearing surface will be thicker at the end of the deck due to beam camber. The thickness for deck design is then, d_{ws} = 6.9 in.

3. Curb and Railing [TL-4 Glulam Timber Rail with Curb]

The timber barrier design is not a part of the design examples. The dimensions were used for weight considerations in Article 8.7.3. For the deck examples, as described above, the deck overhang does not need to be analyzed and the curb and railing do not affect the deck spanning from beam to beam.

[8.4.1.2.2, 9.9.2] **4. Glulam Deck Panels, Southern Pine**

Assumed depth of timber deck panel laminates = d_{lam} = 5.00 in
Assumed width of timber deck panel laminates = b_{lam} = 1.375 in
Attention must be given to the species of wood, as laminate widths and thicknesses vary by species. For a nominal 6 inch wide lamination in Southern Pine, a net finished width of 5 inches or 5 1/8 inches is available (which is the deck depth with the glulam placed flatwise).

Because the individual laminates in the glued laminated deck panels are not orientated horizontally as in a beam, the glulam combinations generally intended for axial loading are commonly used for transverse decks, instead of the combinations normally used for beams.

[4.6.2.1.6] **5. Span Lengths**

In this case, MnDOT uses the effective span, or design span, as center to center of the deck bearing length on each beam, which is also center to center of beams, as stated in AASHTO LRFD.

Effective design span length for the deck panels = L_e = 5.0 ft

6. Unit Weights and Moisture Content

Type of glulam panel wood material = Southern Pine (ID No. 48)

[Table 3.5.1-1] Unit weight of soft-wood = γ_{SP} = 0.050 kcf

[MnDOT Table 3.3.1] Unit weight of bituminous wearing surface = γ_{WS} = 0.150 kcf

[MnDOT 3.3] Standard MnDOT practice is to apply a future wearing course of 20 psf.

[8.4.1.1.3] Moisture content (MC) of timber at the time of installation shall not exceed 19.0%

MnDOT designs for in-service wet-use only which is a MC of greater than 16% for glulam.

7. Southern Pine Glulam Deck (ID No. 48) Strength Properties

[Table 8.4.1.2.3-2] Reference Design Value for flexure = F_{byo} = 2.000 ksi

Reference Design Value for compression perpendicular to grain
= F_{cpo} =0.740 ksi

Reference Design Value for shear parallel to grain (horizontal shear)
= F_{vyo} = .260 ksi

Modulus of elasticity = E_o = 1700 ksi

Select the Basic Configuration

The bridge deck consists of interconnected deck panels, which are oriented perpendicular to traffic. The panels are manufactured using wet use adhesives to join the individual laminates into panels. The panels are attached to each other using vertical spikes through ship lap joints along with longitudinal stiffener beams also called spreader beams. The deck panel depth and spreader beam sizes are based on deflection limits as well as strength considerations. The spreader beams enable the deck to act as a single unit under deflection and to consider it interconnected in accordance with AASHTO LRFD.

For a visual representation of the transverse deck on the glulam beams as well as the spreader beams, Figure 8.7.3.1 of this manual can be referenced. The connections in the shiplap joints are similar to that shown in various figures in Article 8.7.1, except with a transverse deck the joints are also transverse as that is the direction of the panels.

A. Deck Panel Sizes

Transverse glulam deck panels vary in width between 3.0 and 6.0 feet. The dimensions of the panels at the beginning and end of deck are adjusted so that the total deck length matches the length of the beams. The panels are to be manufactured meeting the requirements of ANSI/AITC A190.1. The panels are required to be manufactured using wet use adhesives to join the individual laminates to attain the specified panel size, and under this condition the adhesive bond is stronger than the wood laminates.

B. Spreader Beam Dimensions

[9.9.4.3.2] Interconnection of panels may be made with mechanical fasteners, splines, dowels, or stiffener beams. This example will use stiffener beams, or spreader beams, along with shiplap joints similar to the transverse spike laminated deck. For a transverse deck, the spreader beam is to be placed longitudinally along the bridge at the center of each deck span.

Glulam panels are sometimes designed with horizontal dowel connections which can be effective for transferring loads between panels under ideal conditions, but in practice can be difficult to construct properly. The shiplap joint and spreader beam eliminates the field fit up and installation problems associated with the dowel connections.

The following rough sawn spreader beam dimensions that were verified in the Transverse Spike Laminated Deck Design Example will also be used in this design example (refer to that example for the calculation).

Width of spreader beams = b_{spdr} = 5 in
Depth of spreader beams = d_{spdr} = 5 in

If preferred by the designer, a similar sized glulam spreader beam could be checked and used in this design for a transverse glulam deck, provided it meets the minimum rigidity requirements.

[9.9.4.3] The rigidity of the spreader beam shall be at least 80,000 kip·in^2.

Determine Dead and Live Load Reactions, Shear Forces, and Bending Moments

The dead and live load shear, reaction and bending moment results can be determined using a basic structural analysis computer program, or using the standard beam formulas found in AISC 14th Edition LRFD Manual. MnDOT uses simplified analysis models that are permitted by AASHTO LRFD.

[4.6.2.1.6] In the calculation of force effects using equivalent strips, the axle wheel loads may be considered point loads or patch loads, and the beams considered simply supported or continuous, as appropriate.

Modelling the axle wheel loads as patch loads will not have a large effect with the given beam spacing, and so for the calculations below the wheel loads on the axles are conservatively modelled as point loads.

[3.6.1.3.3]

Per AASHTO LRFD the design load in the design of decks is always an axle load; single wheel loads should not be considered. In addition, when using the approximate strip method for spans primarily in the transverse direction, only the axles for the design truck or the axles for the design tandem (whichever results in the largest effect) shall be applied to deck in determining live load force effects.

A. Analysis Models

In determining the maximum deck forces, MnDOT uses a variation of beam models for the deck strip as follows:

1) The maximum shear forces and reactions are determined by modeling the deck as a continuous beam. Moving live loads are then placed at various locations along the span, to produce the maximum shear and reactions. This method of analysis allows the effects of adjacent spans to be investigated. A two span continuous beam is conservatively assumed for simplicity.
2) The maximum positive bending moments (tension on deck bottom) and deflections are determined by considering the deck as a single simply-supported span between beams.
3) The maximum negative bending moments (tension on deck top) are determined by considering the deck as a single fixed-fixed span between beams, with fixed ends. Looking at the beam formulas in AISC 14th Edition LRFD Manual, it can be seen that this case will not govern, and so it will not be calculated here.

B. Dead Loads per Unit Strip (1 ft)

The units for the dead load results are given in kips for a 1 ft wide transverse strip.

1. Dead Loads per foot (these units could also be given as kips per square foot).

Weight of deck = $w_{deck} = \gamma_{SP} \cdot d_{lam} = 0.050 \cdot 5.0/12 = 0.021$ klf/ft

Weight of wear course = $w_{ws} = \gamma_{ws} \cdot d_{ws} = 0.150 \cdot 6.9/12 = 0.086$ klf/ft

Weight of future wearing course = $w_{FWC} = 0.020$ klf/ft

2. Spreader beam point loads on 1 ft wide strip.

Area of spreader beam = $A_{spdr} = d_{spdr} \cdot b_{spdr} = (5 \cdot 5)/144 = 0.174$ ft^2

Spreader beam load = $P_{spdr} = \gamma_{DFL} \cdot A_{spdr} = 0.050 \cdot 0.174 = 0.009$ kips/ft

[AISC 14th p. 3-213]

C. Dead Load Bending Moments per Unit Strip (1 ft)

Maximum bending moment due to deck weight

$$M_{deck} = \frac{w_{deck} \cdot (L_e)^2}{8} = \frac{0.021 \cdot 5.0^2}{8} = 0.066\ \frac{\text{kip–ft}}{\text{ft}}$$

Maximum bending moment due to wearing surface weight

$$M_{ws} = \frac{w_{ws} \cdot (L_e)^2}{8} = \frac{0.086 \cdot 5.0^2}{8} = 0.269\ \frac{\text{kip–ft}}{\text{ft}}$$

Maximum bending moment due to future wearing surface weight

$$M_{FWC} = \frac{w_{FWC} \cdot (L_e)^2}{8} = \frac{0.020 \cdot 5.0^2}{8} = 0.063\ \frac{\text{kip–ft}}{\text{ft}}$$

Maximum bending moment due to spreader beam weight

$$M_{spdr} = \frac{P_{spdr} \cdot L_e}{4} = \frac{0.009 \cdot 5.0}{4} = 0.011\ \frac{\text{kip–ft}}{\text{ft}}$$

Maximum bending moment due to bridge component dead loads

$M_{dc} = M_{deck} + M_{spdr}$ $\quad M_{dc} = 0.066 + 0.011 = 0.077$ kip·ft/ft

Maximum bending moments due to wearing course loads = M_{dw}

$M_{dw} = M_{ws} + M_{FWC}$ $\quad M_{dw} = 0.269 + 0.063 = 0.332$ kip·ft/ft

[3.6.1.2]

D. Live Load Moments per Axle

The live load bending moment will be calculated per wheel and later converted to a per unit strip (1 ft) format.

1. Design Truck Axle Loads

[3.6.1.2.2]

Point load on one deck span from design truck axle = P_{truck} = 16 kips

Maximum bending moment due to design truck wheel load

$$M_{truck} = \frac{P_{truck} \cdot L_e}{4} = \frac{16.0 \cdot 5.0}{4} = 20.000\ \text{kip–ft}$$

2. Design Tandem Axle Loads

[3.6.1.2.3]

Point load of design tandem wheel = P_{tandem} = 12.5 kips

AASHTO Table A4-1 can be used in the design of concrete decks, but includes impact so is not applicable to timber. However, the table footnotes indicate that specifically calculating the tandem is not necessary. A calculation can be done that shows the heavier single wheel load from the design truck on the smaller area of deck is the controlling case. Therefore, the tandem effect is not calculated for this example.

[4.6.2.1]

E. Modification of Live Load Bending Moment

1. Convert Live Load Bending Moment to Per Unit Strip

The live load bending moment calculated above (M_{truck}) will now be distributed over the transverse equivalent strip width, and converted to a per foot basis.

[Table 4.6.2.1.3-1] For a structural deck thickness h= 5.0 in, the equivalent strip width = 4.0h + 30.0 = 50.0 in

$$M_{truck} = M_{truck} \cdot \frac{1}{E_s} = 20.000 \cdot \frac{12}{50.0} = 4.800 \text{ kip–ft}$$

2. Multiple Presence Factors

[3.6.1.1.2, 4.6.2.1] The multiple presence factor is to be used in conjunction with the equivalent strip widths of 4.6.2.1.

[3.6.1.1.1] Maximum number of traffic lanes on the deck = N_L

$$N_L = \frac{b_{rd}}{12\ \frac{ft}{lane}} = \frac{32}{12} = 2.67 \cong 2 \text{ lanes}$$

[Table 3.6.1.1.2-1] For one lane loaded, the multiple presence factor = m = 1.20
For two lanes loaded, the multiple presence factor = m = 1.00

[C3.6.1.1.2] This design example is for an unspecified ADTT, although AASHTO LRFD recommends limitations on the use of wood deck types based on ADTT. If these recommendations are adhered to, AASHTO LRFD also allows reduction of force effects based on ADTT because the multiple presence factors were developed on the basis of an ADTT of 5000 trucks in one direction. A reduction of 5% to 10% may be applied if the ADTT is expected to be below specified limits during the life of the bridge. If the ADTT level is confirmed, the reduction may be applied subject to the judgment of the designer and approved by the State Bridge Design Engineer.

[AISC 14th p. 3-223] **Shear Force and Support Reactions**

As described above, shear force and reactions are calculated conservatively assuming a two span continuous beam. Axle tire loads can occur transversely at a distance as short as 4 ft apart if in two separate lanes, and if the two lanes are centered on a beam the axle tire loads are then 2 ft either side of a beam. This 2 lane case will need to be checked against the one lane case.

[3.6.1.2.1]
[3.6.1.3.1]

The axle tire placement for the one lane and two lane cases are illustrated below with diagrams, which are shown under the Chapter section "Investigate Shear Resistance Requirements for Deck Panel".

The results are converted to a per foot basis and shown in the table below. The live load force effects are shown for one and two lanes, with the appropriate multiple presence factor, m, applied.

G. Summary of Maximum Shear Force, Reaction and Bending Moment Results

Table 8.7.4.2

Unfactored Load Case	**Maximum Positive Bending Moment (kip·ft/ft)**	**Maximum Shear Force (kips/ft)**	**Maximum Support Reaction (kips/ft)**
Component Dead Load (DC)	0.077	0.071	0.143
Wearing Course Dead Load (DW)	0.332	0.331	0.663
Live Loads			
Design Truck (1 lane, *m*=1.2)	5.760	3.555	3.976
Design Truck (2 lane, *m*=1.0)	4.800	3.041	6.082

Flexural Check of Deck Panel

H. Factored Bending Moment per Unit Strip (1 ft)

1. Load Modifiers

[1.3.2] Standard MnDOT Load Modifiers are summarized in Table 3.2.1 of this manual.

For timber bridges η_D = 1.0. MnDOT considers glued laminated decks to have a conventional level of redundancy and uses η_R = 1.0. This example bridge is assumed to have a design ADT of over 500 for η_I = 1.0.

2. Strength I Limit State Load Factors

[3.4.1] Use the Strength I Limit State to determine the required resistance for the deck panels.

[3.6.2.3] Impact factor need not be applied to wood components.

[4.6.2.3] Skew factor (bridge is not skewed thus 1.0) = r = 1.0

Specific Strength I Limit State Load Factors are found in AASHTO Tables 3.4.1-1 and 3.4.1-2.

The earlier analysis indicated that the truck load controls the bending moment of the deck panels. Therefore, use the truck load in determining the critical live load bending moment acting on the deck panels.

3. Strength I Limit State Bending Moment per Unit Strip (1 ft)

[Tables 3.4.1-1 and 3.4.1-2]

Factored bending moment for the one lane loaded case = $M_{u(m)}$

$$M_{u(m)} = \eta \cdot [1.25 \cdot M_{dc} + 1.5 \cdot M_{dw} + 1.75 \cdot r \cdot [M_{truck} + M_{lane}]]$$

$$M_{u(m)} = 1.0 \cdot [1.25 \cdot 0.077 + 1.50 \cdot 0.332 + 1.75 \cdot 1.0 \cdot \ [5.76]] = 10.674 \ \frac{kip\text{–}ft}{ft}$$

Check Flexural Resistance of Deck Panel

A. Factored Flexural Resistance

The factored bending moment ($M_{u(m)}$) is the required flexural resistance of the deck that needs to be compared with the actual factored flexural resistance of the deck panel (M_r).

[8.6.2]

For a rectangular wood section $M_r = \phi_f \cdot F_b \cdot S_{req} \cdot C_L$.

1. Resistance Factor

[8.5.2.2]

Flexural resistance factor = ϕ_f = 0.85

2. Stability Factor

[8.6.2]

Stability factor for glulam lumber in flexure = C_L

Laminated deck planks are fully braced. C_L = 1.0

3. Adjustment Factors for Reference Design Value

[8.4.4.2]

Format conversion factor for component in flexure = C_{KF}

$C_{KF} = 2.5/\phi = 2.5/0.85 = 2.94$

[8.4.4.3]
[Table 8.4.4.3-2]

Wet Service factor for glued laminated timber in flexure = C_M

C_M = 0.80

Flat use factor for vertically laminated glulam timber in flexure = C_{fu}

[8.4.4.6]
[Table 8.4.4.6-2]

d_{lam} = 5.0 in

C_{fu} = 1.10

[8.4.4.9]
[Table 8.4.4.9-1]

Time effect factor for Strength I Limit State = C_λ

C_λ = 0.80

[Eqn. 8.4.4.1-1]

Adjusted design value = $F_b = F_{byo} \cdot C_{KF} \cdot C_M \cdot C_{fu} \cdot C_\lambda$

F_b = 2.00 x 2.94 x 0.80 x 1.10 x 0.80 = 4.140 ksi

4. Required Section Modulus

The section modulus is dependent on the deck panel depth. The section modulus is used in Part B to solve for the deck panel depth.

B. Required Deck Panel Depth

Required deck flexural resistance = $M_{n(req)}$

For the deck panel depth to meet Strength I Limit State, M_r must equal (or exceed) $M_{u(m)}$, where $M_r = \phi M_{n(req)}$. Therefore, set $\phi M_{n(req)} = M_{u(m)}$.

$$M_{n(req)} = \frac{M_{u(m)}}{\phi_f} = \frac{10.674}{0.85} = 12.558 \text{ kip - ft}$$

Required section modulus of one foot of deck width = S_{req}
Required depth of deck laminates (panel) = d_{req}

$$S_{req} = \frac{12 \text{ in} \cdot d_{req}^{\ 2}}{6}$$

$M_{n(req)} = F_b \cdot S_{req} \cdot C_L$, with $C_L = 1.0$, substituting terms gives

$$d_{req} = \sqrt{\frac{6 \cdot M_{n(req)}}{12\text{in} \cdot F_b \cdot C_L}} = \sqrt{\frac{6 \cdot 12.558 \cdot 12}{12\text{in} \cdot 4.140 \cdot 1.0}} = 4.27 \text{ in} \le 5.0 \text{ in} \qquad \underline{\text{OK}}$$

The required deck panel depth (4.27 inches) indicates that the originally assumed deck depth (5.0 inches) can be used based on flexure. However, it is not uncommon that a deeper section could be required to satisfy the shear requirement, so that is checked next.

Investigate Shear Resistance Requirements for Deck Panel [8.7, 9.9.3.2]

A. Critical Shear Force Location

In transverse decks, maximum shear shall be computed at a distance from the support equal to the depth of the deck (d_{lam}). The tire footprint shall be located adjacent to, and on the span side of, the point of the span where maximum force effect is sought.

Location to check for shear = $(d_{lam} + {}^1/_2 \cdot b_{length}) / L_e$
$= (0.42 \text{ ft} + {}^1/_2 \cdot 0.71 \text{ ft}) / 5.0 \text{ ft}$

Check for shear at about 16% of span length away from the center of support, or 0.78 ft.

Horizontal shear must be checked for wood components. The term "horizontal" shear is typically used in wood design, because a shear failure initiates along the grain. This shear failure is typically along the horizontal axis. The shear stress is equal in magnitude in the vertical direction, but inherent resistance is greater, and so typically does not need to be designed for. AASHTO LRFD C8.7 provides commentary on this.

B. Unfactored Shear Acting on the Deck per Unit Strip (1 ft)

For the uniformly distributed loads, the shear forces are less than the maximums listed in the earlier table (Table 8.7.4.2). The results given below are not the maximum shear forces on the deck (except for the design truck). Rather, they are the values taken at the appropriate distance "d_{lam}" from the critical support face. The following shear forces were taken at the location 16% of span length from center support.

1. Dead Load Shear Force

Component dead load shear force at a distance "d_{lam}" away from the support face = V_{dc} = 0.051 kips

Wear course dead load shear force at a distance "d_{lam}" away from the support face = V_{dw} = 0.239 kips

2. Live Load Shear Forces

Only the design truck is shown below. From the earlier results, this is the load case that gives the maximum shear force. One lane loaded with the multiple presence factor applied produces the maximum live load design shear forces as explained below.

a. Design Truck Load One Lane Case

[3.6.1.2.5] Truck tire contact area consists of a 20 inch width. Placing the 20 inch width according to 9.9.3.2 results in the following on one side of a support (beam) for the one lane case.

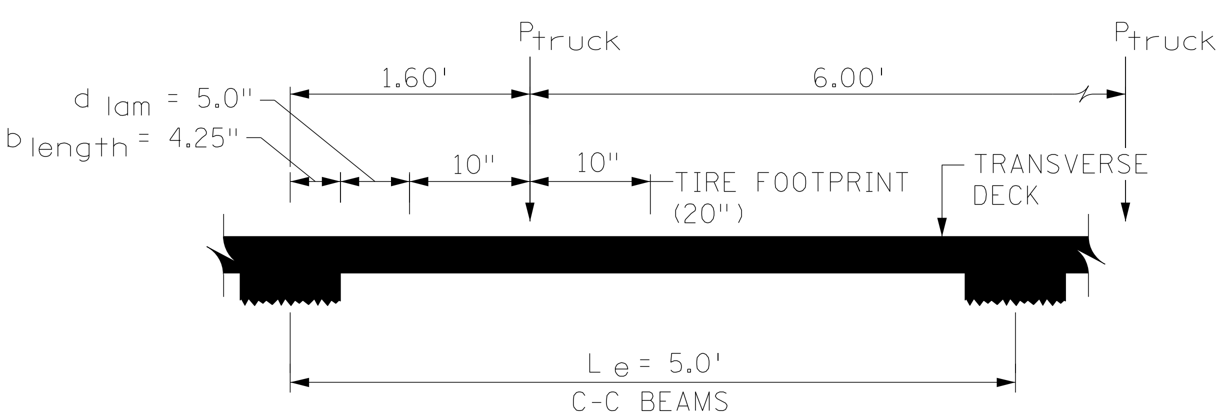


Figure 8.7.4.3

b. Design Truck Load Two Lane Case

[3.6.1.3.1]

For two adjacent loaded lanes, the closest another wheel can be placed on the opposite side of the support is 4.00 ft away, which is 2.40 ft from the support. If the minimum 4.00 ft space between wheels is centered on the support, the distance to the wheel on each side of the support is then 2.00 ft which satisfies the "d_{lam}" minimum (1.60 ft), and is what produces the maximum force effects shown in Table 8.7.4.2.

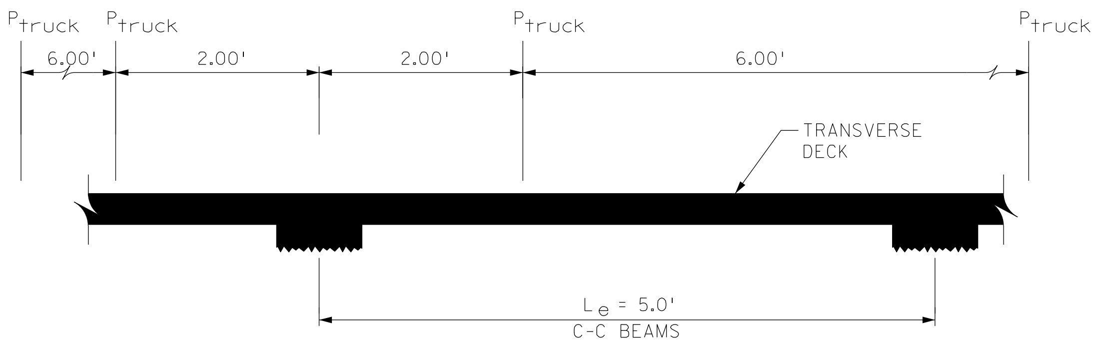


Figure 8.7.4.4

Although the maximum calculated shear forces at a distance "d_{lam}" away from the support for the design truck is governed by the case of two adjacent loaded lanes and is equal to the maximum = V_{truck} = 3.041 kips, with the multiple presence factor applied the one lane loaded case governs the design shear as shown in Table 8.7.4.2.

C. Factored Shear Acting on the Deck Panels per Unit Strip (1 ft)

1. Load Modifiers

Load modifiers for deck design are shown in the flexure check.

2. Strength I Limit State Load Factors

[3.4.1]

Use the Strength I Limit State to determine the required shear resistance of the deck.

Impact and skew applicability are the same as for the flexure check.

Specific Strength I Limit State Load Factors are found in AASHTO Tables 3.4.1-1 and 3.4.1-2.

The above results indicate that a single lane loaded with the design truck controls for shear.

3. Strength I Limit State Shear Force

Strength I Limit State factored shear force, one lane loaded = $V_{u(m)}$

[Tables 3.4.1-1 and 3.4.1-2]

$$V_{u(m)} = \eta \cdot [1.25 \cdot V_{dc} + 1.50 \cdot V_{dw} + 1.75 \cdot r \cdot [V_{truck(m)} + V_{lane(m)}]]$$

$$V_{u(m)} = 1.0 \cdot [1.25 \cdot (0.051) + 1.50 \cdot (0.239) + 1.75 \cdot 1.0 \cdot [3.555]] = 6.644 \text{ kips}$$

Check Shear Resistance of Deck Panel

A. Factored Shear Resistance

The factored shear force $V_{u(m)}$ is the required shear resistance of the deck that needs to be compared with the actual factored shear resistance of the deck (V_r).

[Eqns. 8.7-1, 8.7-2]

For a rectangular wood section $V_r = \phi_v \cdot F_v \cdot b \cdot d_{lam} /1.5$

[8.5.2.2]

1. Resistance Factor

Shear resistance factor = $\phi_v = 0.75$

2. Adjustment Factors for Reference Design Values

[8.4.4.2]

Format conversion factor: $C_{KF} = 2.5/\phi = 2.5/0.75 = 3.33$

[8.4.4.3]

Wet Service factor = $C_M = 0.875$

[8.4.4.9]

Time effect factor = $C_\lambda = 0.80$

[Eqn. 8.4.4.1-2]

Adjusted design value = $F_v = F_{vyo} \cdot C_{KF} \cdot C_M \cdot C_\lambda$

$F_v = 0.260 \cdot 3.33 \cdot 0.875 \cdot 0.80 = 0.606$ ksi

B. Deck Panel Shear Check

Required deck shear resistance = $V_{u(m)}$

For the deck to meet Strength I Limit State, $V_{r(prov)}$ must equal or exceed $V_{u(m)}$. As determined previously, $V_{u(m)} = 6.644$ kips.

[Eqn. 8.7-2]

$$V_{r(prov)} = \phi_v \frac{(F_v \cdot b \cdot d_{lam})}{1.5} = 0.75 \cdot \frac{(0.606 \cdot 12 \cdot 5.0)}{1.5} = 18.180 \text{ kips}$$

$V_{u(m)} = 6.644 \text{ kips} \leq V_{r(prov)} = 18.180 \text{ kips}$ OK

Check Compression Resistance

Compression, or bearing of the deck on the beams, should be computed in accordance with the provisions of AASHTO LRFD for non-standard situations that provide a very narrow bearing area for the transverse deck. For this example, compression bearing on the glued laminated beams is not close to governing the design of the deck panel and so the calculation is not shown here. It usually will not govern a transverse deck design for a bridge of standard configuration. A bearing resistance calculation check for the longitudinal deck (on the pier caps) is shown in 8.7.1 Longitudinal Spike Laminated Timber Deck Design Example.

Investigate Deflection Requirements [9.9.3.3]

A. Deck Live Load Deflection with Current Deck Parameters

The final check for the transverse deck design to meet AASHTO LRFD is the deformation, or deflection, calculation. The design truck will have the most severe effect, and that is used for checking the transverse deck deflection.

[3.6.1.3.3]

As stated earlier, per AASHTO LRFD, when using the approximate strip method for spans primarily in the transverse direction, only the axles for the design truck or the design tandem (whichever results in the largest effect) shall be applied to the deck in determining live load force effects.

Deflections are to be calculated using Service I Limit State.

Calculate deck deflections for a transverse interconnected deck using a per foot width approach. This approach can be used on a glulam deck with shiplap joints and a spreader beam.

[2.5.2.6.2]

[C2.5.2.6.2]

In the absence of other criteria, the recommended deflection limit in AASHTO LRFD for wood construction is span/425, which will be used here. The designer and owner should determine if a more restrictive criteria is justified, such as to reduce bituminous wearing course cracking and maintenance.

As of note, if a plank deck or a non-interconnected panel deck is being analyzed, a different approach likely is required for the live load distribution, and an additional limitation of 0.10 inches relative deflection between adjacent edges is also required.

1. Deck Stiffness

Moment of inertia of one foot width of deck panels = I_{prov}

$$I_{prov} = \frac{1}{12} \cdot b \cdot d_{lam}^{3} = \frac{1}{12} \cdot 12 \cdot (5.0)^3 = 125.0 \ \text{in}^4$$

Adjusted deck panel modulus of elasticity = E

[8.4.4.3]
[Table 8.4.4.3-2]

Wet Service Factor for Modulus of Elasticity = C_M

$C_M = 0.833$

[Eqn. 8.4.4.1-6]

$E = E_o \cdot C_M = 1700 \text{ ksi} \cdot 0.833 = 1416.1 \text{ ksi}$

2. Loads per Unit Strip Width (1 ft)

Design truck load on deck span used for deflection calculations = $P_{\Delta truck}$

Similar to calculations for the maximum positive bending moments, deflections are determined by considering the deck as a single simply-supported span between beams. Therefore, the point load on one deck span from design truck axle = P_{truck} = 16 kips.

P_{truck} expressed as per foot width = $P_{\Delta truck}$:

$P_{\Delta truck} = P_{truck} \cdot 12 \text{ in} / E_s = P_{truck} \cdot 12 \text{ in} / 50 \text{ in}$

$P_{\Delta truck} = 16 \cdot 0.240 = 3.84 \text{ kips/ft}$

[Table 3.6.1.1.2-1] One lane loaded governs, the multiple presence factor = m = 1.20

[3.6.1.3]
[AISC 14th p. 3-215]

3. Live Load Deflection Calculations

Deflection at deck midspan due to the design truck load = Δ_{truck}

$$\Delta_{truck} = \frac{m \cdot P_{\Delta truck} \cdot L_e^3}{48 \cdot E \cdot I_{Prov}} = \frac{1.20 \cdot 3.84 \cdot (5.00 \cdot 12)^3}{48 \cdot 1416.1 \cdot 125} = 0.12 \text{ in}$$

The maximum deflection = $\Delta_{max} = \Delta_{truck}$ = 0.12 in

[2.5.2.6.2] Live load deflection limit at deck midspan = Δ_{max}

$\Delta_{max} = L_e / 425 = 5.0 \text{ ft} \cdot 12 \text{ in} / 425 \cdot \text{ft} = 0.14 \text{ in}$

$\Delta = 0.12 \text{ in} \leq \Delta_{max} = 0.14 \text{ in}$ OK

Deflections are also okay. Thus, the initial 5.0 inch deck panel depth and grade are adequate for the design.

8.8 Load Rating Examples

[References to MBE Section 6]

This section demonstrates the calculation process for load rating wood bridge elements and contains several examples completed by the LRFR methodology. The Manual for Bridge Evaluation (MBE) published by AASHTO must be referenced as it governs bridge load ratings. All left hand references in this article are to the MBE.

The general load rating equation for determining the Rating Factor (RF) of a particular element, for the force effect being rated, is as follows:

[Eqn. 6A.4.2.1-1]

$$RF = \frac{C - (\gamma_{DC})(DC) - (\gamma_{DW})(DW) \pm (\gamma_P)(P)}{(\gamma_{LL})(LL + IM)}$$

[6A.1.1]

All existing, new, and rehabilitated bridges designed by LRFD must be load rated by the LRFR method. A structure properly designed and checked by the LRFD method should have the following minimum RF: RF_{Inv} = 1.0, and RF_{Oper} = 1.3

[6A.1.4]

For cases in which the MBE is silent, the current AASHTO LRFD shall govern.

The following examples load rate the superstructure elements previously designed in the design examples (Section 8.7). Usually the force effects of moment and shear are checked for typical bridge superstructures. Bearing should also be checked if based on the engineer's judgment it could control the bridge load rating. In the following examples the force effects previously designed for, will be load rated.

[Appendix A6A]
[6A.1.5.1]

Generally if the Design Load Rating, or first-level assessment, has an Inventory Rating Factor (RF) greater than or equal to 1.0, the bridge will not require posting. For simplicity of the following examples and to simply demonstrate the procedure, only the AASHTO LRFD HL-93 design vehicular live load will be load rated.

[6A.2.2.1]

The dead load effects on the structure shall be computed in accordance with the conditions existing at the time of the analysis. For a new bridge, the future wearing course used in design should not be included in the load rating calculation.

[6A.2.3.2]

One difference from design is traffic lane widths for live load application. In load ratings, roadway widths from 18 to 20 ft shall have two traffic lanes, each equal to one half the roadway width. Otherwise, live load placement is generally the same as for design.

[6A.7.4.1] Requirements specific to wood structures are shown in 6A.7. For wood structures, rating factors for the design-load rating shall be based on the Strength I load combination.

[6A.7.5] As with design, dynamic load allowance need not be applied to wood components.

8.8.1 Longitudinal Spike Laminated Timber Deck Rating Example

The variables in the general load rating equation need to be defined. Numbers from the design example for the longitudinal spike laminated timber deck will be used as applicable. The load rating will also be done on a per ft basis.

Flexure Force effect

A. Capacity for Flexure Strength Limit State

$C = \varphi_c \, \varphi_s \, \phi R_n$

[6A.4.2.3]

For a new bridge $\varphi_c = 1.00$

[6A.4.2.4]

For all timber bridges $\varphi_s = 1.00$

For flexure, $\phi R_n = \phi_f \, M_n = \phi_f \cdot F_b \cdot S \cdot C_L$

From Article 8.7.1 for this longitudinal spike laminated deck:

$\phi_f = 0.85$

$F_b = 2.16$ ksi for Douglas Fir-Larch Deck (No. 1)

$$S = \frac{12\ \text{in} \cdot d^2}{6} = \frac{12\ \text{in} \cdot 14^2}{6} = 392 \cdot \text{in}^3$$

$C_L = 1.0$

$\phi_f \, M_n = 0.85 \cdot 2.16 \cdot 392 \cdot 1.0 = 719.71$ kip · in

Therefore, $C = 1.00 \cdot 1.00 \cdot 719.71 = 719.71$ kip · in

B. Load Factors

The load factors as found in the MBE for the general load rating equation at the Inventory Rating level are:

[Table 6A.4.2.2-1]

$\gamma_{DC} = 1.25$

$\gamma_{DW} = 1.50$

$\gamma_P = 1.0$ (there are no other permanent loads and so this will be neglected in the final calculation)

$\gamma_{LL} = 1.75$

The only change to the Operating Rating level is for the live load factor:

$\gamma_{LL} = 1.35$

C. Force Effects for Flexure

The force effects for flexure (bending moments) were calculated in Article 8.7.1. The values shown here are taken from Table 8.7.1.1 (except that the FWC is removed from M_{dw}):

M_{dc} = 3.82 kip · ft = 45.84 kip · in

M_{dw} = 2.84 kip · ft = 34.08 kip · in

M_{tandem} = 21.40 kip · ft = 256.80 kip · in (for two lanes loaded, tandem governs over truck)

M_{lane} = 3.56 kip · ft = 42.72 kip · in (for two lanes loaded)

Rating Factors

A. Calculate Inventory Rating Factor for Flexure

$$RF_{Inv} = \frac{719.71 - (1.25)(45.84) - (1.50)(34.08)}{(1.75)(256.80 + 42.72)}$$

RF_{Inv} = 1.17

B. Calculate Operating Rating Factor for Flexure

$$RF_{Oper} = \frac{719.71 - (1.25)(45.84) - (1.50)(34.08)}{(1.35)(256.80 + 42.72)}$$

RF_{Oper} = 1.51

8.8.2 Glulam Beam Superstructure Rating Example

Similar to the example above, the variables in the general load rating equation need to be defined for the element (in this case beam) and force effect being rated. Numbers from the design example for the glulam beam superstructure will be used as applicable. The load rating will be done for an interior beam because that was previously shown to govern.

Flexure Force effect

[Eqn. 6A.4.2.1-2]

A. Capacity for Flexure Strength Limit State

$C = \varphi_c \, \varphi_s \, \phi R_n$

[6A.4.2.3]

For a new bridge $\varphi_c = 1.00$

[6A.4.2.4]

For all timber bridges $\varphi_s = 1.00$

For flexure, $\phi R_n = \phi_f \, M_n = \phi_f \cdot F_b \cdot S \cdot C_L$

Article 8.7.3 for the glulam beam in flexure:

$\phi_f = 0.85$

$F_b = 3.97$ ksi for SP/SP glulam beam (24F-V3)

$$S_{prov} = \frac{b \cdot d^2}{6} = \frac{8.5 \cdot 46.75^2}{6} = 3096.21 \text{ in}^3$$

$C_L = 1.0$

$\phi_f \, M_n = 0.85 \cdot 3.97 \cdot 3096.21 \cdot 1.0 = 10{,}448.16 \text{ kip} \cdot \text{in}$

Therefore, $C = 1.00 \cdot 1.00 \cdot 10{,}448.16 = 10{,}448.16 \text{ kip} \cdot \text{in}$

B. Load Factors

The load factors as found in the MBE for the general load rating equation at the Inventory Rating level are:

[Table 6A.4.2.2-1]

$\gamma_{DC} = 1.25$

$\gamma_{DW} = 1.50$

$\gamma_P = 1.0$ (there are no other permanent loads and so this will be neglected in the final calculation)

$\gamma_{LL} = 1.75$

The only change to the Operating Rating level is for the Live Load Factor:

$\gamma_{LL} = 1.35$

C. Force Effects for Flexure

The force effects for flexure (bending moments) were calculated in Article 8.7.3. The values shown here are taken from Table 8.7.3.1 (except that the FWC is removed from M_{dw}):

M_{dc} = 69.95 kip · ft = 839.40 kip · in
M_{dw} = 61.30 kip · ft = 735.60 kip · in
M_{truck} = 291.12 kip · ft = 3493.44 kip · in (truck governs over tandem)
M_{lane} = 84.66 kip · ft = 1015.92 kip · in

Rating Factors

A. Calculate Inventory Rating Factor for Flexure

$$RF_{Inv} = \frac{10{,}448.16 - (1.25)(839.40) - (1.50)(735.60)}{(1.75)(3493.44 + 1015.92)}$$

RF_{Inv} = 1.05

B. Calculate Operating Rating Factor for Flexure

$$RF_{Oper} = \frac{10{,}448.16 - (1.25)(839.40) - (1.50)(735.60)}{(1.35)(3493.44 + 1015.92)}$$

RF_{Oper} = 1.36

Shear Force effect

[Eqn. 6A.4.2.1-2]

A. Capacity for Shear Strength Limit State

$C = \varphi_c \, \varphi_s \, \phi R_n$

[6A.4.2.3]

For a new bridge φ_c = 1.00

[6A.4.2.4]

For all timber bridges φ_s = 1.00

For shear, $\phi R_n = \phi_v V_n = \phi_v \cdot F_v \cdot w_{bm} \cdot d_{bm} / 1.5$

From Article 8.7.3 for the glulam beam in shear:

ϕ_v = 0.75

F_v = 0.699 ksi for SP/SP glulam beam (24F-V3)

d_{bm} = 46.75 in
w_{bm} = 8.5 in

$\phi_v V_n$ = 0.75 · 0.699 · 8.5 · 46.75 /1.5 = 138.88 kips

Therefore, C = 1.00 · 1.00 · 138.88 = 138.88 kips

B. Load Factors

The load factors as found in the MBE for the general load rating equation at the Inventory Rating level are:

[Table 6A.4.2.2-1]

γ_{DC} = 1.25
γ_{DW} = 1.50
γ_{P} = 1.0 (there are no other permanent loads and so this will be neglected in the final calculation)
γ_{LL} = 1.75

The only change to the Operating Rating level is for the live load factor:

γ_{LL} = 1.35

C. Force Effects for Shear

The force effects for shear were calculated in Article 8.7.3. The values shown here are taken from that example at a distance "d_{beam}" away from the support (the FWC is not included in V_{dw}):

V_{dc} = 5.18 kips
V_{dw} = 4.67 kips
V_{truck} = 38.00 kips (truck governs over tandem)
V_{lane} = 6.72 kips
V_{LL} = 26.83 kips (this is the distributed LL per beam)

Rating Factors

A. Calculate Inventory Rating Factor for Shear

$$RF_{Inv} = \frac{138.88 - (1.25)(5.18) - (1.50)(4.67)}{(1.75)(26.83)}$$

RF_{Inv} = 2.67

B. Calculate Operating Rating Factor for Shear

$$RF_{Oper} = \frac{138.88 - (1.25)(5.18) - (1.50)(4.67)}{(1.35)(26.83)}$$

RF_{Oper} = 3.46

Compressive Force effect
[Eqn. 6A.4.2.1-2]

A. Capacity for Compressive Strength Limit State

$C = \varphi_c \varphi_s \phi R_n$

[6A.4.2.3]

For a new bridge φ_c = 1.00

[6A.4.2.4]

For all timber bridges $\varphi_s = 1.00$

For compression, $\phi R_n = \phi_{cperp} P_n = \phi_{cperp} \cdot F_{cp} \cdot A_b \cdot C_b$

From Article 8.7.3 for this glulam beam:

$\phi_{cperp} = 0.90$

$F_{cp} = 0.731$ ksi for SP/SP glulam beam (24F-V3)

Bearing Area $= A_b = L_b \times w_{bm} = 18.0 \times 8.5 = 153.0 \text{ in}^2$

$C_b = 1.00$

$\phi_{cperp} P_n = 0.90 \cdot 0.731 \cdot 153.0 \cdot 1.0 = 100.66$ kips

Therefore, $C = 1.00 \cdot 1.00 \cdot 100.66 = 100.66$ kips

B. Load Factors

The load factors as found in the MBE for the general load rating equation at the Inventory Rating level are:

[Table 6A.4.2.2-1]

$\gamma_{DC} = 1.25$
$\gamma_{DW} = 1.50$
$\gamma_P = 1.0$ (there are no other permanent loads and so this will be neglected in the final calculation)
$\gamma_{LL} = 1.75$

The only change to the Operating Rating level is for the Live Load Factor:

$\gamma_{LL} = 1.35$

C. Force Effects for Compression

The force effects for compression were calculated in Article 8.7.3. The values shown here are taken from that example (the FWC is not included):

$R_{dc} = 6.84$ kips
$R_{dw} = 5.84$ kips
$R_{truck} = 56.00$ kips (truck governs over tandem)
$R_{lane} = 13.40$ kips
$R_{LL} = 41.64$ kips (this is the distributed LL per beam)

Rating Factors

A. Calculate Inventory Rating Factor for Compression

$$RF_{Inv} = \frac{100.66 - (1.25)(6.84) - (1.50)(5.84)}{(1.75)(41.64)}$$

$RF_{Inv} = 1.14$

B. Calculate Operating Rating Factor for Compression

$$RF_{Oper} = \frac{100.66 - (1.25)(6.84) - (1.50)(5.84)}{(1.35)(41.64)}$$

$RF_{Oper} = 1.48$

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8.8.3 Transverse Deck Rating Examples

The variables in the general load rating equation need to be defined for the transverse decks and force effect being rated. Numbers from the design example for the transverse decks will be used as applicable. The load rating will also be done on a per ft basis.

Transverse Spike Laminated Deck

The transverse spike laminated deck will be load rated first, for the flexure and the shear force effects.

Flexure Force effect
[Eqn. 6A.4.2.1-2]

A. Capacity for Flexure Strength Limit State

$C = \varphi_c \, \varphi_s \, \phi R_n$

[6A.4.2.3] For a new bridge $\varphi_c = 1.00$

[6A.4.2.4] For all timber bridges $\varphi_s = 1.00$

For flexure, $\phi R_n = \phi_f \, M_n = \phi_f \cdot F_b \cdot S \cdot C_L$

From Article 8.7.4 for this transverse spike laminated deck in flexure:

$\phi_f = 0.85$

$F_b = 2.152$ ksi for Douglas Fir-Larch Deck (No. 2)

$$S = \frac{b \cdot d^2}{6} = \frac{12\ \text{in} \cdot 5.75^2}{6} = 66.13 \cdot \text{in}^3$$

$C_L = 1.0$

$\phi_f \, M_n = 0.85 \cdot 2.152 \cdot 66.13 \cdot 1.0 = 120.97$ kip · in

Therefore, $C = 1.00 \cdot 1.00 \cdot 120.97 = 120.97$ kip · in

B. Load Factors

The load factors as found in the MBE for the general load rating equation at the Inventory Rating level are:

[Table 6A.4.2.2-1]

$\gamma_{DC} = 1.25$
$\gamma_{DW} = 1.50$
$\gamma_P = 1.0$ (there are no other permanent loads and so this will be neglected in the final calculation)
$\gamma_{LL} = 1.75$

The only change to the Operating Rating level is for the Live Load Factor:

$\gamma_{LL} = 1.35$

C. Force Effects for Flexure

The force effects for flexure (bending moments) were calculated in Article 8.7.4 on a per ft basis. The values shown here are taken from Table 8.7.4.1 (except that the FWC is removed from M_{dw}):

M_{dc} = 0.089 kip · ft = 1.07 kip · in
M_{dw} = 0.269 kip · ft = 3.23 kip · in
M_{truck} = 4.572 kip · ft = 54.86 kip · in (truck governs over tandem)

Rating Factors

A. Calculate Inventory Rating Factor for Flexure

$$RF_{Inv} = \frac{120.97 - (1.25)(1.07) - (1.50)(3.23)}{(1.75)(54.86)}$$

RF_{Inv} = 1.20

B. Calculate Operating Rating Factor for Flexure

$$RF_{Oper} = \frac{120.97 - (1.25)(1.07) - (1.50)(3.23)}{(1.35)(54.86)}$$

RF_{Oper} = 1.55

Shear Force effect

A. Capacity for Shear Strength Limit State

[Eqn. 6A.4.2.1-2]

$C = \varphi_c \varphi_s \phi R_n$

[6A.4.2.3]

For a new bridge $\varphi_c = 1.00$

[6A.4.2.4]

For all timber bridges $\varphi_s = 1.00$

For shear, $\phi R_n = \phi_v V_n = \phi_v \cdot F_v \cdot b \cdot d_{lam} /1.5$

From Article 8.7.4 for this transverse spike laminated deck in shear:

$\phi_v = 0.75$

F_v = 0.372 ksi for Douglas Fir-Larch Deck (No. 2)

b = 12.0 in
d_{lam} = 5.75 in

$\phi_v V_n$ = 0.75 · 0.372 · 12.0 · 5.75 /1.5 = 12.83 kips

Therefore, C = 1.00 · 1.00 · 12.83 = 12.83 kips

B. Load Factors

The load factors as found in the MBE for the general load rating equation at the Inventory Rating level are:

[Table 6A.4.2.2-1]

γ_{DC} = 1.25

γ_{DW} = 1.50

γ_{P} = 1.0 (there are no other permanent loads and so this will be neglected in the final calculation)

γ_{LL} = 1.75

The only change to the Operating Rating level is for the Live Load Factor:

γ_{LL} = 1.35

C. Force Effects for Shear

The force effects for shear were calculated in Example 8.7.4 on a per ft basis. The values shown here are taken at a distance "d_{lam}" away from the support (the FWC is not included in V_{dw}):

V_{dc} = 0.059 kips

V_{dw} = 0.190 kips

V_{truck} = 2.775 kips (truck governs over tandem)

Rating Factors

A. Calculate Inventory Rating Factor for Shear

$$RF_{Inv} = \frac{12.83 - (1.25)(0.059) - (1.50)(0.190)}{(1.75)(2.775)}$$

RF_{Inv} = 2.57

B. Calculate Operating Rating Factor for Shear

$$RF_{Oper} = \frac{12.83 - (1.25)(0.059) - (1.50)(0.190)}{(1.35)(2.775)}$$

RF_{Oper} = 3.33

Transverse Glued Laminated Deck

The transverse glued laminated deck will be load rated next, for the flexure and the shear force effects.

Flexure Force effect

[Eqn. 6A.4.2.1-2]

A. Capacity for Flexure Strength Limit State

$C = \varphi_c \, \varphi_s \, \phi R_n$

[6A.4.2.3] For a new bridge $\varphi_c = 1.00$

[6A.4.2.4] For all timber bridges $\varphi_s = 1.00$

For flexure, $\phi R_n = \phi_f\, M_n = \phi_f \cdot F_b \cdot S \cdot C_L$

From Article 8.7.4 for this transverse glued laminated deck in flexure:

$\phi_f = 0.85$

$F_b = 4.140$ ksi for Southern Pine (ID No. 48)

$$S = \frac{12\ \text{in} \cdot d^2}{6} = \frac{12\ \text{in} \cdot 5.0^2}{6} = 50.0 \cdot \text{in}^3$$

$C_L = 1.0$

$\phi_f\, M_n = 0.85 \cdot 4.14 \cdot 50.0 \cdot 1.0 = 175.95$ kip · in

Therefore, C = 1.00 · 1.00 · 175.95 = 175.95 kip · in

B. Load Factors

The load factors as found in the MBE for the general load rating equation at the Inventory Rating level are:

[Table 6A.4.2.2-1]

$\gamma_{DC} = 1.25$
$\gamma_{DW} = 1.50$
$\gamma_P = 1.0$ (there are no other permanent loads and so this will be neglected in the final calculation)
$\gamma_{LL} = 1.75$

The only change to the Operating Rating level is for the Live Load Factor:

$\gamma_{LL} = 1.35$

C. Force Effects for Flexure

The force effects for flexure (bending moments) were calculated in Article 8.7.4 on a per ft basis. The values shown here are taken from Table 8.7.4.2 (except that the FWC is removed from M_{dw}):

$M_{dc} = 0.077$ kip · ft = 0.92 kip · in
$M_{dw} = 0.269$ kip · ft = 3.23 kip · in
$M_{truck} = 5.76$ kip · ft = 69.12 kip · in (truck governs over tandem)

Rating Factors

A. Calculate Inventory Rating Factor for Flexure

$$RF_{Inv} = \frac{175.95 - (1.25)(0.92) - (1.50)(3.23)}{(1.75)(69.12)}$$

$RF_{Inv} = 1.41$

B. Calculate Operating Rating Factor for Flexure

$$RF_{Oper} = \frac{175.95 - (1.25)(0.92) - (1.50)(3.23)}{(1.35)(69.12)}$$

$RF_{Oper} = 1.82$

Shear Force effect

A. Capacity for Shear Strength Limit State

[Eqn. 6A.4.2.1-2] $C = \varphi_c \, \varphi_s \, \phi R_n$

[6A.4.2.3] For a new bridge $\varphi_c = 1.00$

[6A.4.2.4] For all timber bridges $\varphi_s = 1.00$

For shear, $\phi R_n = \phi_v V_n = \phi_v \cdot F_v \cdot b \cdot d_{lam} / 1.5$

From Article 8.7.4 for this transverse glued laminated deck in shear:

$\phi_v = 0.75$

$F_v = 0.606$ ksi for Southern Pine (ID No. 48)

$b = 12.0$ in
$d_{lam} = 5.0$ in

$\phi_v V_n = 0.75 \cdot 0.606 \cdot 12.0 \cdot 5.0 / 1.5 = 18.18$ kips

Therefore, $C = 1.00 \cdot 1.00 \cdot 18.18 = 18.18$ kips

B. Load Factors

The load factors as found in the MBE for the general load rating equation at the Inventory Rating level are:

[Table 6A.4.2.2-1]

$\gamma_{DC} = 1.25$
$\gamma_{DW} = 1.50$
$\gamma_{P} = 1.0$ (there are no other permanent loads and so this will be neglected in the final calculation)
$\gamma_{LL} = 1.75$

The only change to the Operating Rating level is for the live load factor:
γ_{LL} = 1.35

C. Force Effects for Shear

The force effects for shear were calculated in Article 8.7.4 on a per ft basis. The values shown here are taken at a distance "d_{lam}" away from the support (the FWC is not included in V_{dw}):

V_{dc} = 0.051 kips
V_{dw} = 0.190 kips
V_{truck} = 3.555 kips (truck governs over tandem)

Rating Factors

A. Calculate Inventory Rating Factor for Shear

$$RF_{Inv} = \frac{18.18 - (1.25)(0.051) - (1.50)(0.190)}{(1.75)(3.555)}$$

RF_{Inv} = 2.87

B. Calculate Operating Rating Factor for Shear

$$RF_{Oper} = \frac{18.18 - (1.25)(0.051) - (1.50)(0.190)}{(1.35)(3.555)}$$

RF_{Oper} = 3.72