CHAPTER 7

DESIGN OF BEAM SUPERSTRUCTURES

7.1 INTRODUCTION

Beam superstructures consist of a series of longitudinal timber beams supporting a transverse timber deck. They are constructed of glulam or sawn lumber components and have historically been the most common and most economical type of timber bridge (Figure 7-1). For the past 20 years, beam bridges have been constructed almost exclusively from glulam because of the greater size and better performance characteristics it provides compared with sawn lumber systems. Sawn lumber bridges are still used to a limited degree on local public roads and private road systems with low traffic volumes.

This chapter addresses design considerations and requirements for beam superstructures and is divided into two parts. Part I deals with glulam systems and includes the design of glulam beams and transverse glulam deck panels. Part II covers sawn lumber systems and includes the design of lumber beams and transverse nail-laminated and plank decks. In both parts, deck design is limited to transverse and configurations only. Applications involving longitudinal decks on beam superstructures are discussed in Chapter 8. Railing systems and wearing surfaces for beam bridges are covered in Chapters 10 and 11, respectively.

7.2 DESIGN CRITERIA AND DEFINITIONS

The material presented in this chapter is based on the 1983 edition of the *AASHTO Standard Specifications for Highway Bridges* (AASHTO), including interim specifications through 1987.[1] When specific design requirements or criteria are not addressed by that specification, recommendations are based on referenced standards and specifications or commonly accepted design practice. Because AASHTO specifications are periodically revised to reflect new developments in bridge design, the designer should refer to the latest edition for the most current requirements. This chapter is not intended to serve as a substitute for current specifications.

General design criteria used in this chapter are summarized below. Additional criteria related to specific component design are given in the applicable sections.

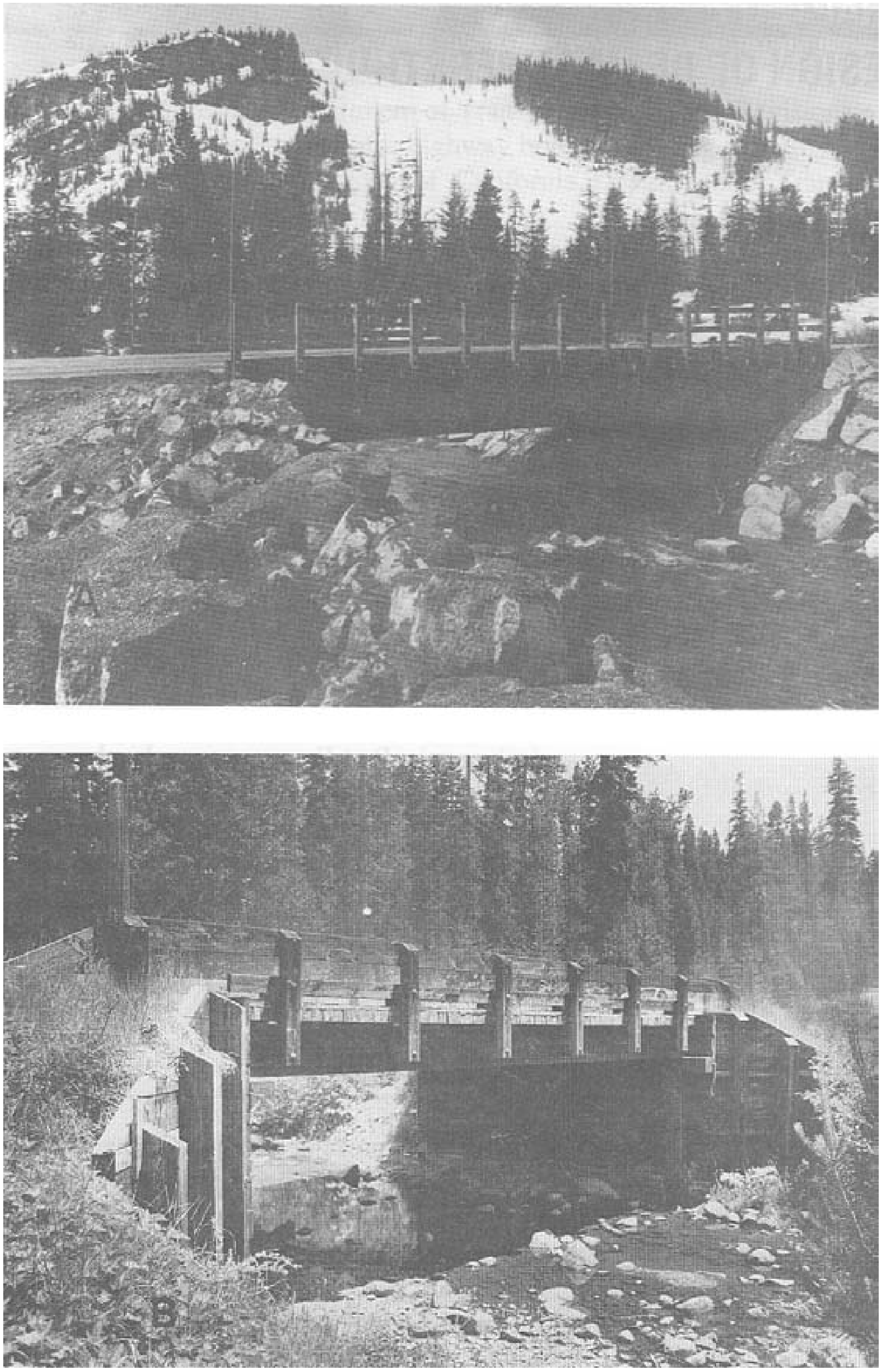


Figure 7-1. - Beam superstructures constructed of (A) glulam timber and (B) sawn lumber.

DESIGN PROCEDURES AND EXAMPLES

Sequential design procedures and examples are included in this chapter to familiarize the designer with the requirements for beam bridges. Design procedures are intended to outline basic requirements and present applicable design equations and aids. The order of the procedures is based on the most common sequence used in design and may vary for different applications. Examples are based on more specific site requirements, and criteria are noted for each example.

LOADS

Loads are based on the AASHTO load requirements discussed in Chapter 6. Beam and deck design procedures are limited to AASHTO Group I loads where design is routinely controlled by a combination of structure dead load and vehicle live load. Vehicle live loads are standard AASHTO loads consisting of H 15-44, H 20-44, HS 15-44, and HS 20-44 vehicles. Overloads are considered in the design examples in AASHTO Group IB, where allowable stresses are increased by 33 percent, as discussed in Chapter 6.

For deck design, AASHTO special provisions for HS 20-44 and H 20-44 loads apply, and a 12,000-pound wheel load is used unless otherwise noted (AASHTO Figures 3.7.6A and 3.7.7A). In most cases, deck design aids include the dead load of a 3-inch asphalt wearing surface. These aids can be used with reasonable accuracy for other common wearing surfaces since wearing-surface dead load normally has little effect on beam or deck design.

MATERIALS

Tabulated values for sawn lumber are taken from the 1986 edition of the NDS.[37,38] Species used are Douglas Fir-Larch and Southern Pine, but the principles of design apply to wood of any species group. For glulam, tabulated values are taken from the 1987 edition of *AITC 117--Design.*[5] Material specifications are given by combination symbol; however, glulam can also be specified by required design values in a format similar to that given in *AITC 117--Design.* Visually graded combination symbols are recommended, with provisions for E-rated substitution at the option of the manufacturer. *All timber components are assumed to be pressure-treated with an oil-type preservative prior to fabrication, as discussed in Chapter 4.*

LIVE LOAD DEFLECTION

AASHTO specifications do not include design criteria or guidelines for beam or deck live load deflection. The recommendations in this chapter are based on field experience and common design practice as noted for the specific component. Although it is highly recommended that these deflection guidelines be followed, deflection criteria should be based on specific design circumstances and are left to designer judgment.

CONDITIONS OF USE

Tabulated values for timber components must be adjusted for specific use conditions by all applicable modification factors discussed in Chapter 5. The following criteria have been used in this chapter.

Duration of Load. Beam and deck design for combined dead load and vehicle live load are based on a normal duration of load (that is, design stresses at the maximum allowable level do not exceed a cumulative total of 10 years). Therefore, equations for allowable design values do not include the duration of load factor, C_D.

Moisture Content. With the exception of glulam beams covered by a watertight deck, all stresses in bridge components are adjusted for wet-use conditions. Based on recommendations of the AITC,[7] covered glulam beams are designed for dry-condition stresses with the exception of compression perpendicular to grain at supports, where wet-condition stress is recommended. This is based on the assumption that a watertight deck sufficiently protects glulam beams and that superficial surface wetting does not cause significant increases in beam moisture content except at supports.

Temperature Effects and Fire-Retardant Treatment. Conditions requiring adjustments for temperature or fire-retardant treatment are rare in bridge applications. Design equations in this chapter do not include modification factors for temperature effect, C_t, or fire-retardant treatment, C_R.

PART I:
GLUED-LAMINATED TIMBER (GLULAM) SYSTEMS

7.3 GENERAL

Glued-laminated beam bridges consist of a series of transverse glulam deck panels supported on straight or slightly curved beams (Figure 7-2). They are the most practical for clear spans of 20 to 100 feet and are widely used on single-lane and multiple-lane roads and highways. Glulam has proved to be an excellent material for beam bridges because members are available in a range of sizes and grades and are easily adaptable to a modular or systems concept of design and construction. Although glulam can be custom fabricated in many shapes and sizes, the most economical structure uses standardized components in a repetitious arrangement, an approach that is particularly adaptable to bridges (Figure 7-3).

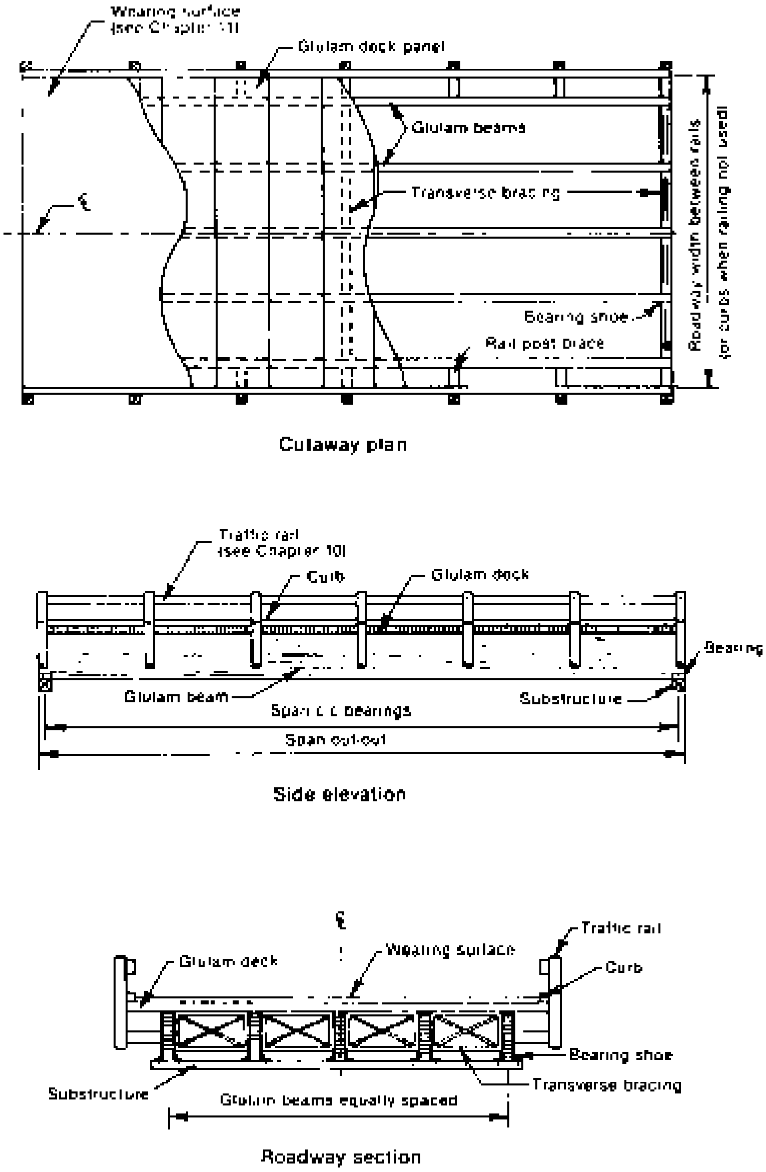


Figure 7-2. - Typical glulam beam bridge configuration.

The following three sections address design considerations, procedures, and details for glulam beam bridges. Beams and beam components are discussed first, followed by transverse glulam deck panels.

7.4 DESIGN OF BEAMS AND BEAM COMPONENTS

Beams are the principal load-carrying components of the bridge superstructure. They must be proportioned to resist applied loads and meet serviceability requirements for deflection. The total beam system consists of three primary components: beams, transverse bracing, and bearings.

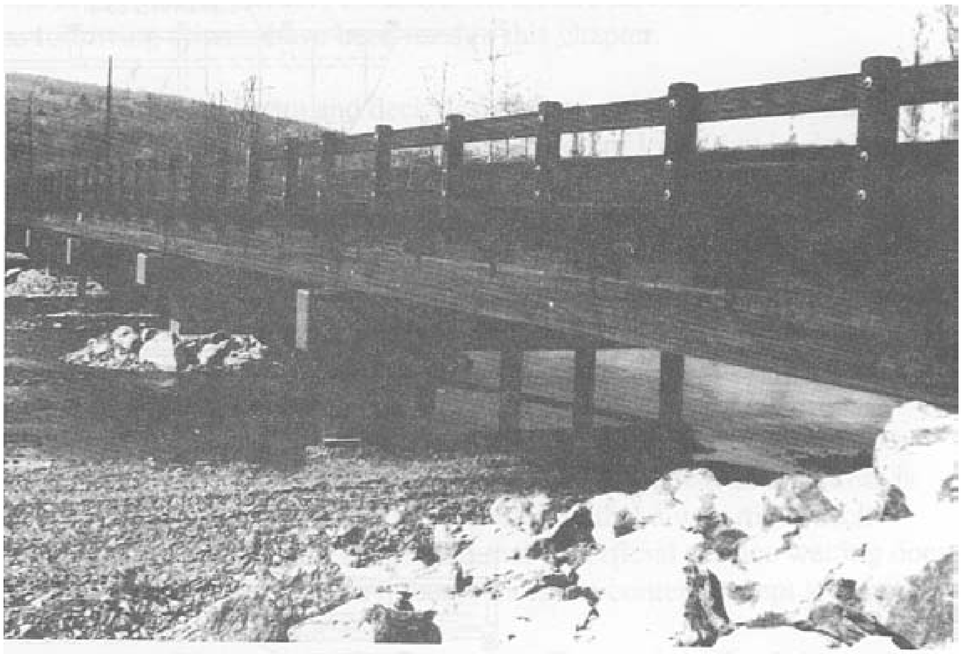


Figure 7-3.- Glulam beam bridge, 290 feet long in Tioga County, New York. This bridge was completely prefabricated in standardized components that were bolted together at the project site (photo courtesy of Weyerhaeuser Co.).

Each of these components is designed individually to perform specific functions. Together they interact to form the structural framework of the bridge.

BEAM DESIGN

Glulam bridge beams are horizontally laminated members designed from the bending combinations given in Table 1 of *AITC 117--Design.* These combinations provide the most efficient beam section where primary loading is applied perpendicular to the wide face of the laminations. The quality and strength of outer laminations are varied for different combination symbols to provide a wide range of tabulated design values in both positive and negative bending.

Glulam beams offer substantial advantages over conventional sawn lumber beams because they are manufactured in larger sizes, provide improved dimensional stability, and can be cambered to offset dead load deflection: Beams are available in standard widths ranging from 3 to 14-1/4 inches (Table 7-1) and in depth multiples of 1-1/2 inches for western species and 1-3/8 inches for Southern Pine. Beam length is usually limited by treating and transportation considerations to a practical maximum of 110 to 120 feet, but longer members may be feasible in some areas. Tables of standard glulam section properties are given in Chapter 16.

Table 7-1. - Standard glulam beam widths.

Nominal width (in.)	Net finished width (in.)	
	Western species	Southern Pine
4	3-1/8	3
6	5-1/8	5
8	6-3/4	6-3/4
10	8-3/4	8-1/2
12	10-3/4	10-1/2
14	12-1/4	—
16	14-1/4	—

Live Load Distribution

Methods for determining the maximum moment, shear, and reactions for truck and lane loads were discussed in Chapter 6. For beam superstructures, the designer must also determine the portion of the total load that is laterally distributed to each beam. The ability of a bridge to laterally distribute loads to individual beams depends on the transverse stiffness of the structure as a unit and is influenced by the type and configuration of the deck and the number, spacing, and size of beams. Load distribution may also be influenced by the type and spacing of beam bracing or diaphragms, but the effect of these components is not considered for determining load distribution.

In view of the complexity of the theoretical analysis involved in determining lateral wheel-load distribution, AASHTO specifications give empirical methods for longitudinal beam design. The fractional portion of the total vehicle load distributed to each beam is computed as a distribution factor (DF) expressed in wheel lines (WL) per beam. The magnitude of the design forces is determined by multiplying the distribution factor for each beam by the maximum force produced by one wheel line of the design vehicle (moment, shear, reaction, and so forth). The procedures for determining distribution factors for longitudinal beams depend on the type of force and are specified separately for moment, shear, and reactions.

Distribution for Moment

When computing bending moments in longitudinal beams (AASHTO 3.23.2), wheel loads are assumed to act as point loads. Lateral distribution is determined by empirical methods based on the position of the beam relative to the transverse roadway section. Different criteria are given for outside beams and for interior beams; however, AASHTO requires that the load distributed to an outside beam not be less than that distributed to an interior beam.

The distribution factor for moment in outside beams is determined by computing the reaction of the wheel lines at the beam, assuming the deck acts as a simple span between beams (Figure 7-4). Wheel lines in the outside traffic lane are positioned laterally to produce the maximum reaction at the beam, but wheel lines are not placed closer than 2 feet from the face of the traffic railing or curb (Chapter 6). The distribution factor for moment for interior beams is computed from empirical formulas based on deck thickness, beam spacing, and the number of traffic lanes (Table 7-2). For glulam decks 6 inches or more in nominal thickness, these equations are valid up to the maximum beam spacing specified in the table. When the average beam spacing exceeds the maximum, the distribution factor is the reaction of the wheel lines at the beam, assuming the flooring between beams acts as a simple span (Figure 7-5). In this case, wheel lines are laterally positioned in traffic lanes to produce the maximum beam reaction (wheel lines in adjacent traffic lanes are separated by 4 feet).

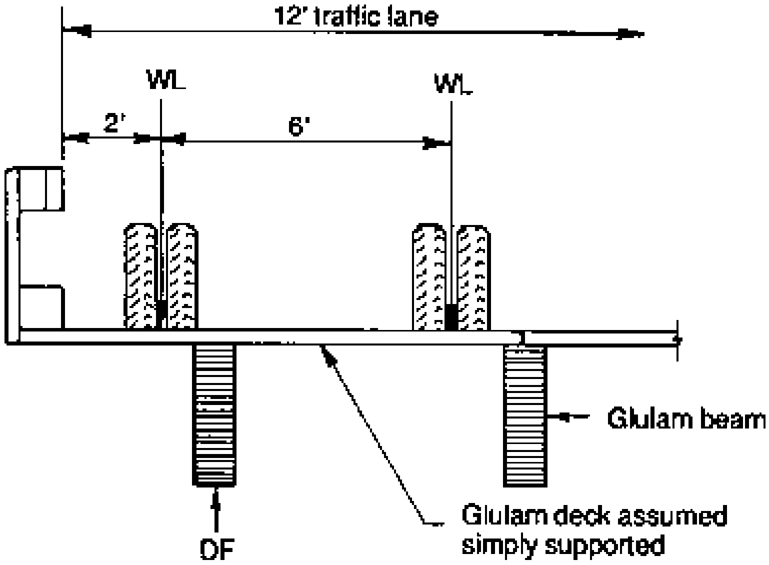


Figure 7-4. - Wheel load distribution factor to outside beams, assuming the deck acts as a simple span between supporting beams.

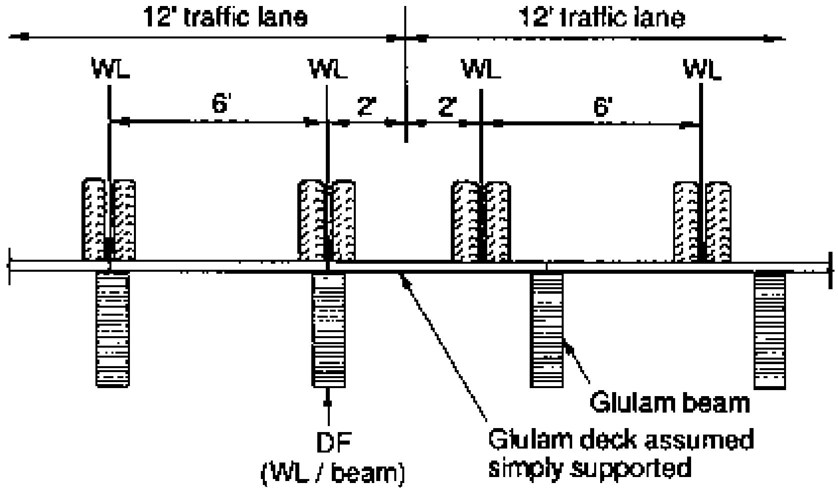
Table 7-2 - Interior beam live load distribution factors for glulam beams with transverse glulam decks.

Nominal deck thickness (in.)	DF for moment (wheel lines/beam): Bridges designed for one traffic lane	DF for moment (wheel lines/beam): Bridges designed for two or more traffic lanes
4	$S/4.5$	$S/4.0$
≥ 6	$S/6.0$	$S/5.0$
	If S exceeds 6 ft, use footnote a.	If S exceeds 7.5 ft, use footnote a.

[a] In this case, the distribution factor for each beam is the reaction of the wheel lines, assuming the deck between beams to act as a simple beam.

S = average beam spacing (ft).

From AASHTO[1] Table 3.23.1; ©1983. Used by permission.



F`igure 7-5. - Wheel load distribution factor to interior beams, assuming the deck acts as a simple span between supporting beams.

Distribution for Shear

Live-load horizontal shear in glulam beams (AASHTO 13.3.1) is computed from the maximum vertical shear occurring at a distance from the support equal to three times the beam depth (3*d)* or the span quarter point *(L*/4), whichever is less (Figure 7-6). Lateral shear distribution at this point is computed as one-half the sum of 60 percent of the shear from the undistributed wheel lines and the shear from the wheel lines distributed laterally as specified for moment. For undistributed wheel lines, one wheel line is assumed to be carried by one beam. These requirements are expressed as

$$V_{LL} = 0.5[(0.6V_{LU}) + V_{LD}] \qquad (7\text{-}1)$$

where V_{LL} = distributed live-load vertical shear used to compute horizontal shear (lb),

V_{LU} = maximum vertical shear from an undistributed wheel line (lb), and

V_{LD} = maximum vertical shear from the vehicle wheel lines distributed laterally as specified for moment (lb).

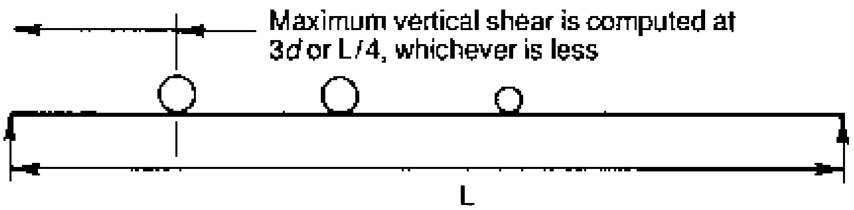


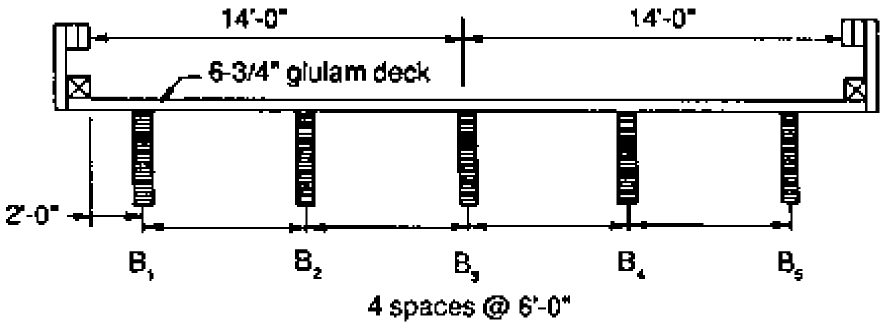
Figure 7-6. - Live load horizontal shear in timber beams is based on the maximum vertical shear occurring at a distance from the support equal to three times the beam depth (3d), or the span quarter point (L/4), whichever is less.

Distribution for Reactions

Live load distribution for end reactions (AASHTO 3.23.1) is computed assuming no longitudinal distribution of wheel loads. The DF for outside and interior beams is determined by computing the reaction of the wheel lines at the beam, assuming the deck acts as a simple span between beams (Figures 7-4 and 7-5).

Example 7-1 - Live load distribution on a multiple-lane beam bridge

A two-lane beam bridge with a 28-foot roadway width spans 52 feet. The superstructure consists of a 6-3/4-inch glulam deck supported by 5 glulam beams, symmetrically spaced at 6-feet on center. Determine the distributed live load moment, shear and reactions for an HS 20-44 design vehicle. Assume an initial beam depth of 43-1/2 inches for shear distribution.



Solution

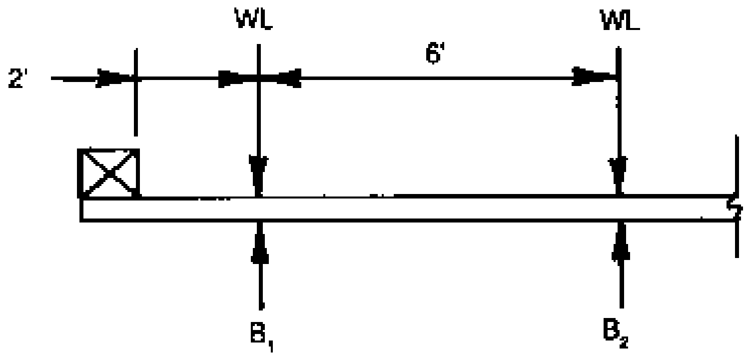
The designer must determine the distribution factors for interior and outside beams and the magnitude of the maximum forces produced by one wheel line of the design vehicle. The product of applicable DF and wheel line force provides the design value for each beam.

Distribution for Moment

The moment distribution factor for interior beams is determined from Table 7-2 based on the deck thickness, number of traffic lanes, and beam spacing. For a 6-3/4-inch glulam deck, two-lane bridge, and 6-foot beam spacing,

$$\mathrm{DF} = \frac{S}{5.0} = \frac{6.0}{5.0} = 1.20\ \mathrm{WL/beam}$$

For outside beams, the DF is computed by assuming the deck acts as a simple span between beams. The center of the wheel load is placed 2 feet from the face of the railing, and the outside beam reaction is computed, in wheel lines, by statics:



The maximum reaction results in a DF of 1.0 WL/beam; however, the DF to outside beams cannot be less than that to interior beams. Therefore, the DF to both outside and interior beams is 1.2 WL/beam.

From Table 16-8, or computations discussed in Chapter 6, the maximum moment for one wheel line of an HS 20-44 truck on a 52-foot span is 331.77 ft-k. The distributed live load moment for interior and outside beams is

$$M_{LL} = (1.20\ \text{WL/beam})(331.77\ \text{ft-k}) = 398.12\ \text{ft-k}$$

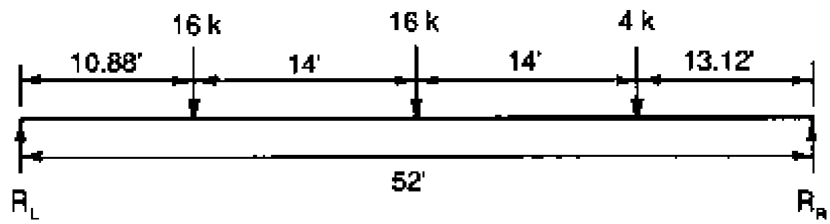
Distribution for Shear

Live load shear distribution is computed by Equation 7-1 using the same distribution factors used for moment. The first step is to compute the maximum vertical shear occurring at the lesser of *3d* or *L/4* from the support:

$$3d = \frac{(3)(43.5)}{12\ \text{in/ft}} = 10.88\ \text{ft} \qquad \frac{L}{4} = \frac{52}{4} = 13.0\ \text{ft}$$

$3d$ = 10.88 ft controls.

The maximum vertical shear for an undistributed wheel line (V_{LU}) is computed by placing the heaviest axle 10.88 feet from the support as discussed in Chapter 6:



$$V_{LU} = R_L = \frac{(13.12)(4\text{k}) + (27.12)(16\text{k}) + (41.12)(16\text{k})}{52} = 22.01\ \text{k}$$

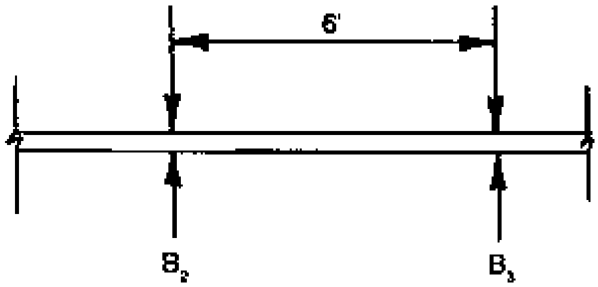
Because the moment DF is the same for interior and outside beams, the distributed shear for interior and outside beams will also be the same. By Equation 7-1,

$$V_{LD} = \text{DF}(V_{LU}) = 1.20(22.01) = 26.41 \text{ k}$$

$$V_{LL} = 0.50[(0.60V_{LU}) + V_{LD}] = 0.50[(0.60)(22.01) + (26.41)] = 19.81 \text{ k}$$

Distribution for Reactions

The reaction distribution factors to interior and outside beams are computed by assuming the deck acts as a simple span between beams. In this case, the vehicle track width of 6 feet equals the beam spacing, and the maximum DF for interior beams is 1.0:



For outside beams, the DF also equals 1.0 as initially computed for moment.

From Table 16-8, the maximum reaction for one wheel line of an HS 20-44 truck on a 52-foot span is 29.54 k. The distributed reaction for interior and outside beams is

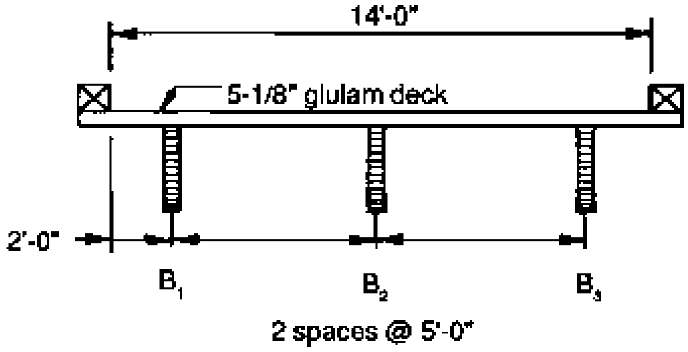
$$R_{LL} = (1.0 \text{ WL/beam})(29.54 \text{ k}) = 29.54\text{k}$$

Summary

	Interior beams	**Outside beams**
Moment	398.12 ft-k	398.12 ft-k
Shear	19.81 k	19.81 k
Reaction	29.54 k	29.54 k

Example 7-2 - Live load distribution on a single-lane beam bridge

A single-lane beam bridge with a 14-foot roadway width spans 32 feet. The superstructure consists of a 5-1/8-inch glulam deck supported by 3 glulam beams, symmetrically spaced at 5 feet on center. Determine the distributed live load moment, shear, and reactions for an HS 15-44 design vehicle. Assume an initial beam depth of 30 inches for shear distribution.



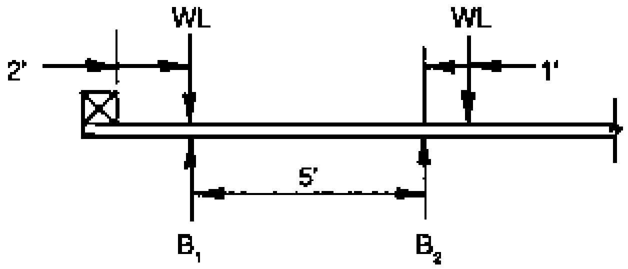
Solution

Distribution for Moment

Moment distribution to the interior beam is determined from Table 7-2:

$$DF = \frac{S}{6.0} = \frac{5.0}{6.0} = 0.83 \text{ WL/beam}$$

For outside beams, the distribution factor is computed by assuming the deck acts as a simple span between beams:



By examination, the DF to the outside beam is 1.0 WL/beam.

From Table 16-8, the maximum moment for one wheel line of an HS 15-44 truck on a 32-foot span is 117.19 ft-k. The distributed live load moments for interior and outside beams are

$$\text{Interior beam } M_{LL} = (0.83 \text{ WL/beam})(117.19 \text{ ft-k}) = 97.27 \text{ ft-k}$$

$$\text{Outside beam } M_{LL} = (1.0 \text{ WL/beam})(117.19 \text{ ft-k}) = 117.19 \text{ ft-k}$$

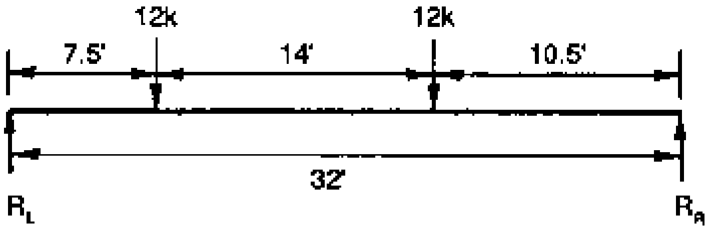
Distribution for Shear

Shear distribution is computed by Equation 7-1 based on the maximum vertical shear at the lesser of *3d* or *L/4* from the support:

$$3d = \frac{(3)(30)}{12 \text{ in/ft}} = 7.5 \text{ ft} \qquad \frac{L}{4} = \frac{32}{4} = 8.0 \text{ ft}$$

$3d = 7.5$ ft controls.

The maximum vertical shear 7.5 feet from the support is computed for one wheel line:



$$V_{LU} = R_L = \frac{(17.5\text{ ft})(24\text{ k})}{32\text{ ft}} = 13.13\text{ k}$$

By Equation 7-1,

$$\text{Interior beam } V_{LL} = 0.50\,[(0.60V_{LU}) + V_{LD}]$$

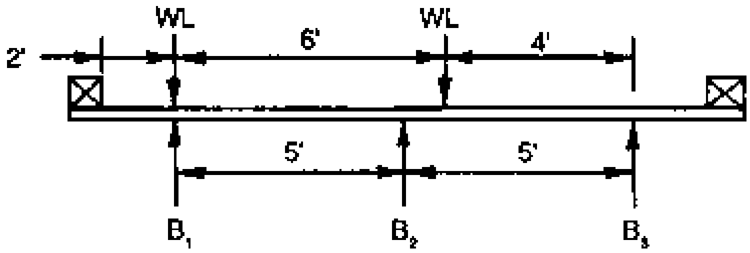
$$= 0.50\,[(0.60)(13.13) + (0.83)(13.13)] = 9.39\text{ k}$$

$$\text{Outside beam } V_{LL} = 0.50\,[(0.60V_{LU}) + V_{LD}]$$

$$= 0.50\,[(0.60)(13.13) + (1.0)(13.13)] = 10.50\text{ k}$$

Distribution for Reactions

Distribution factors for reactions are computed by assuming the deck acts as a simple span between beams. For interior beams, the wheel line is placed 2 feet from the curb face and moments for span B_2-B_3 are summed about B_3:



$$DF = \frac{(4\text{ ft})(WL)}{5\text{ ft}} = 0.80\text{ WL}$$

For outside beams, the distribution factor is the same as that obtained for moment, DF = 1.0 WL/beam.

From Table 16-8, the maximum reaction for one wheel line of an HS 15-44 truck on a 32-foot span is 19.13 k. The distributed reactions for interior and outside beams are

Interior beam R_{LL} (0.80 WL/beam)(19.13 k) = 15.30 k

Outside beam R_{LL} = (1.0 WL/beam)(19.13 k) = 19.13 k

Summary

	Interior beam	Outside beams
Moment	97.27 ft-k	117.19 ft-k
Shear	9.39 k	10.50 k
Reaction	15.30 k	19.13 k

Beam Configuration

One of the most influential factors on the overall economy and performance of a glulam bridge is the beam configuration. For a given roadway width, the number and spacing of beams can affect size and strength requirements for beam and deck elements and significantly influence the cost for material, fabrication, and construction. The number of combinations of beam size and spacing is potentially infinite, and the designer must select the most economical combination that provides the required structural capacity and meets serviceability requirements for deflection. In most situations, beam configuration is based on an economic evaluation influenced by three factors: (1) site restrictions, (2) deck thickness and performance, and (3) live load distribution to the beams.

Site Restrictions

Efficient beam design favors a relatively narrow, deep section. In some cases, the optimum beam depth may not be practical because of vertical clearance restrictions at the site. In these situations beam depth is limited, and the number of beams must be increased to achieve the same capacity provided by fewer, deeper beams. The most common configuration for such low-profile beam bridges uses a series of closely spaced beam groups (Figure 7-7). In most cases, however, the longitudinal deck designs discussed in Chapters 8 and 9 will provide a more economical design. Additional information on low-profile beam configurations is given in references listed at the end of this chapter.[7,62]

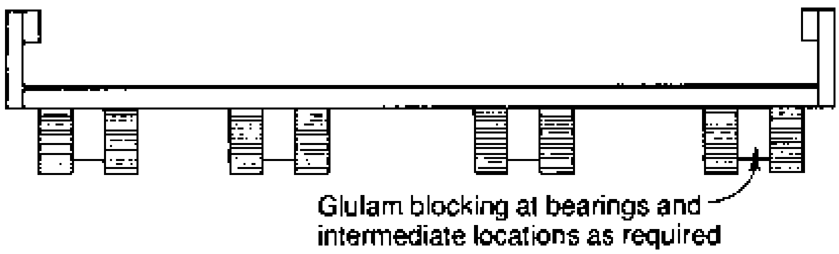


Figure 7-7. - Typical low-profile glulam beam configuration.

Deck Thickness and Performance

Deck thickness and performance vary with the spacing of supporting beams. As beam spacing increases, the stress and deflection of the deck increase, resulting in greater deck thickness, strength, or stiffness requirements. The thickness of glulam deck panels is based on standard member sizes that increase in depth in 1-1/2- to 2-inch increments. As a result, the load-carrying capacity and stiffness of a panel is adequate for a range of beam spacings. For example, a 6-3/4-inch deck panel is used when the computed deck thickness is between 5-1/8 and 6-3/4 inches. The largest effect of beam spacing on the deck occurs when the panel thickness must be increased to the next thicker panel; for example, from 6-3/4 to 8-3/4 inches. On the other hand, considerable savings may be realized when the next smaller deck thickness can be used.

In general, the most practical and most economical beam spacing for transverse glulam decks supporting highway loads is between 4.5 and 6.5 feet. The maximum recommended deck overhang, measured from the centerline of the exterior beam to the face of the curb or railing, is approximately 2.5 feet. These values are based on deck stress and deflection considerations that may vary slightly for different panel combination symbols and configurations.

Live Load Distribution

In beam design, the magnitude of the vehicle live load supported by each beam is directly related to the distribution factor computed for that beam. The higher the distribution factor, the greater the load the beam must support. Thus, the value of the DF gives a good indication of relative beam size and grade requirements for different configurations.

The relationship between the distribution factor for moment and beam spacing is illustrated for a 24-foot-wide roadway and three equally spaced beam configurations in Figure 7-8. The concepts shown for this configuration are also applicable to other roadway widths and beam configurations. The graph shows the moment distribution factor, DF, for interior and outside beams as a function of center-to-center beam spacing, *S*. Solid curves for outside beams represent the feasible range in spacing where the deck overhang is between 1 and 2.5 feet. The dashed portion of the curves identifies beam spacings where the overhang is greater than 2.5 feet. The following points should be noted:

1. The interior beam DF is a function of beam spacing and is not affected by the total number of beams.

2. When beam spacing is to the right of the intersection of interior and outside beam curves, the interior beam DF controls for all beams and outside beams must be designed for the higher interior beam DF.

3. When beam spacing is to the left of the curve intersection, the DF for outside beams is greater than for interior beams. In this case, the load supported by each beam is based on the respective DF for

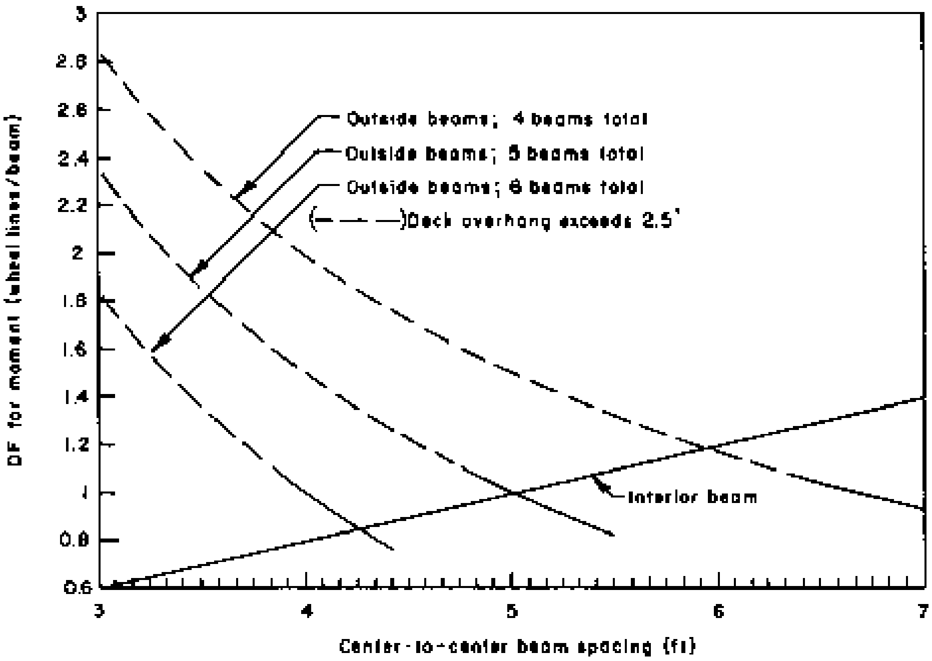


Figure 7-8. - Effects of beam configuration on the vehicle live load distribution factor (DF); roadway width of 24 feet; transverse glulam deck, 6 inches or more in nominal thickness.

that beam; that is, exterior beams support a greater portion of the load than interior beams.

4. The DF for each beam decreases as the total number of beams increases (beam spacing decreases). At the intersection of interior and outside beam curves for the five- and six-beam configurations, the distribution factors are 1.00 WL/beam and 0.85 WL/beam, respectively. For the four-beam configuration, beam spacing is limited by deck overhang restrictions, and the minimum DF of 1.27 WL/beam is controlled by interior beams.

As a general rule, beam spacing to the right of the curve intersection is the least economical because all beams must be designed for the higher DF required for interior beams. Spacing should be kept at or to the left of the intersection to achieve maximum economy. For wide bridges with many interior beams it may be beneficial to use a spacing left of the curve intersection that provides a lower DF for interior beams; however, all beams are normally designed to be the same depth, and a reduced DF for interior beams is not economical unless it allows the use of the next-lower standard beam width.

Exclusive of site restrictions, beam configuration should be based on economic and performance considerations for the deck and beam components. These considerations will vary depending on material prices, availability at the time of construction, and transportation and construction costs. The recommended beam configurations used in this chapter are given in Table 7-3.

Table 7-3.—Recommended beam spacing for glulam beams with transverse glulam decks.

Roadway width (ft)[a]	Number of beams	Beam spacing (ft)	Deck overhang (ft)[b]	Moment DF Interior beams[c]	Moment DF outside beams[d]
Single-lane bridges					
14	3	5.5	1.5	0.92	0.92
16	3	6.0	2.0	1.00	1.00
Double-lane bridges					
24	5	5.0	2.0	1.00	1.00
26	5	5.5	2.0	1.10	1.10
28	5	6.0	2.0	1.20	1.20
34	6	6.0	2.0	1.20	1.20

[a] Measured face to face of railings, or of curbs when railing is not used.

[b] Measured from centerline of outside beam to face of railing or curbs.

[c] For glulam decks 6 inches or more in nominal thickness (S/6 for single-lane; S/5 for two or more lanes).

[d] Computed assuming the deck acts as a simple span between beams, but not less than the interior beam DF.

Beam Design Procedures

Beam design is an interactive process that follows the same basic procedures discussed in Chapter 5. A combination symbol is selected and the beam is designed for bending, deflection, shear, and bearing requirements. Design is routinely controlled by a combination of dead load and vehicle live load given in AASHTO Load Groups I or IB (Chapter 6). Transverse or longitudinal loads may be significant in some cases and should also be checked.

Basic design procedures for glulam bridge beams are summarized in the following steps. The sequence assumes a typical case, where bending or deflection controls design. On short, heavily loaded spans, shear may control design, and the sequence should be modified. For clarity, design procedures are limited to one beam of a simple-span structure loaded with dead load and a standard AASHTO vehicle live load. Application of these procedures is illustrated in Examples 7-3 and 7-4, following the procedures.

1. Define basic configuration and design criteria.

Define the longitudinal and transverse bridge configuration, including the following:

a. Span length L measured center-to-center of bearings
b. Roadway width measured face-to-face of railings or curbs (AASHTO 2.1.2)
c. Number of traffic lanes (Chapter 6)
d. Number and spacing of beams
e. Deck and railing/curb configuration

Identify design vehicles (including overloads), other applicable loads, and AASHTO load combinations discussed in Chapter 6. Also note design requirements for live load deflection and any restrictions on beam depth or other design criteria.

2. Select beam combination symbol.

An initial beam combination symbol is selected from the visually graded bending combinations given in Table 1 of *AITC 117-Design.* Combination symbols that are commonly used for bridges are given in Table 7-4. Select a species and combination symbol and note tabulated values in bending (F_{bx}), compression perpendicular to grain ($F_{c\perp x}$), horizontal shear (F_{vx}), and modulus of elasticity (E_x).

Table 7-4. - Glulam bending combination symbols commonly used for bridge beams.

Beam configuration	Western species combination symbols	Southern Pine combination symbols
Single span	24F-V3 24F-V4	24F-V2 24F-V3 24F-V6
Continuous spans	24F-V8	24F-V5

3. Determine deck dead load and dead load moment.

Compute the deck dead load supported by each beam, including the weight of the deck, wearing surface, railing, and other attached components (lb/ft). Refer to Chapter 6 for procedures and material weights used for dead load calculations. When deck thickness is unknown, use an estimated thickness of 6-3/4 inches. Estimates of rail dead loads can be made from typical designs shown in Chapter 10. Minor differences between estimated and actual deck and rail dead loads normally have an insignificant effect on beam design, but should be verified and revised during the design process.

For the usual case of a uniformly distributed deck dead load, dead load moment is computed as

$$M_{DL} = \frac{w_{DL} L^2}{8} \tag{7-2}$$

where M_{DL} = dead load moment (in-lb),

w_{DL} = uniform deck dead load (lb/in), and

L = beam span (in).

When the deck dead load is not uniformly distributed, dead load moment should be computed by statics for the specific loading condition.

4. Determine live load moment.

Live load moments are computed for interior and outside beams by multiplying the maximum moment for one wheel line of the design vehicle by the applicable moment distribution factors. Tables of maximum vehicle live load moments for standard AASHTO loads and selected overloads on simple spans are given in Table 16-8.

5. Determine beam size based on bending.

Allowable bending stress in beams is controlled by the largest reduction in tabulated stress resulting from application of the size factor, C_F, or the lateral stability of beams factor, C_L (Chapter 5). The allowable bending stress in bridge beams is normally controlled by C_F, rather than C_L. Thus, initial beam size is estimated based on the deck dead load moment and vehicle live load moment, assuming the size factor controls allowable bending stress (beam dead load moment is unknown at this point). This is computed as

$$S_x C_F = \frac{M}{F_b'} \tag{7-3}$$

where $S_x C_F$ = required beam section modulus adjusted by the size factor, C_F (in^2),

M = applied dead load and live load bending moment (in-lb),

$F_b' = F_{bx} C_M$ (lb/in^2), and

C_M = moisture content factor for bending = 0.80.

An initial beam size can be selected from the $S_x C_F$ values given in glulam section property tables in Chapter 16, but it is usually more convenient to use Figure 7-9. By entering the graph with the required $S_x C_F$ value, the required beam depth for standard beam widths can be readily obtained. Beam design generally favors a relatively narrow, deep section with a depth-to-width ratio between 4:1 and 6:1.

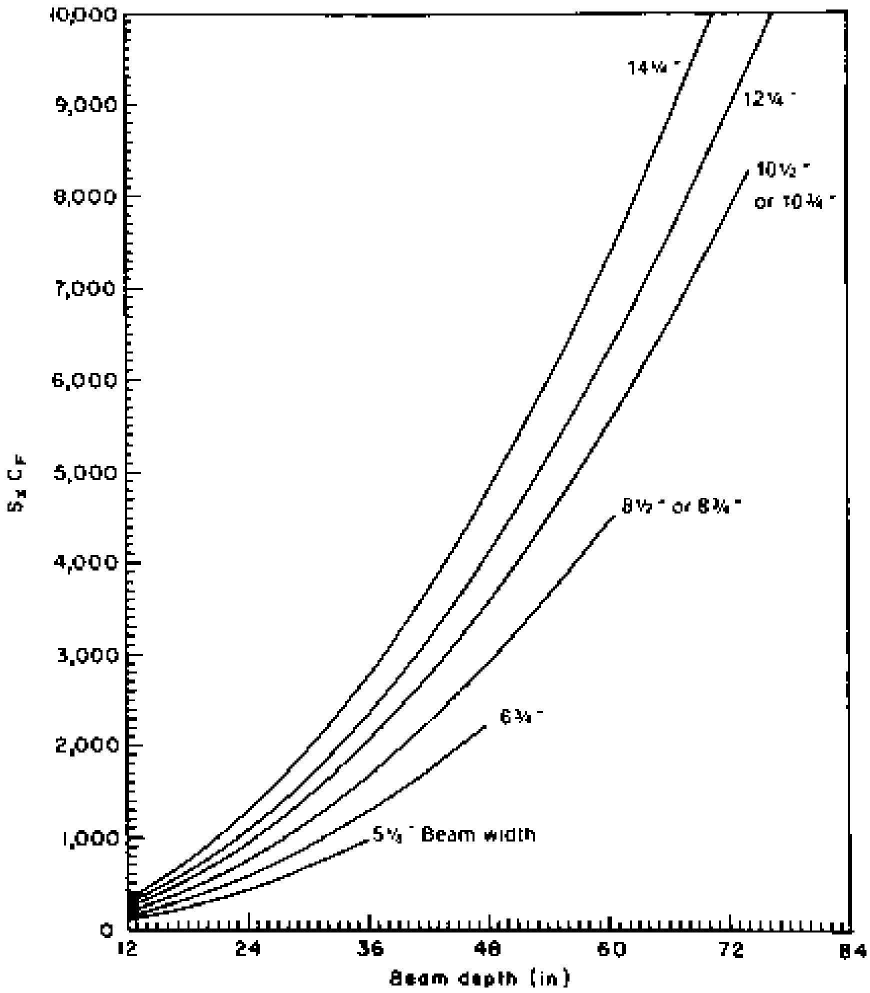


Figure 7-9. - Approximate adjusted section modulus (S_xC_F) versus beam depth for standard glulam beam widths.

After an initial beam size is selected, beam dead load moment is computed for the estimated beam size and added to the deck dead load and live load moments. A revised beam size is selected using the same procedures for initial beam selection. This interactive process is continued until a satisfactory beam size is finalized. Applied stress is then computed for the member using

$$f_b = \frac{M}{S_x} \tag{7-4}$$

This stress must not be greater than the allowable stress from

$$F_b' = F_{bx} C_M C_F \tag{7-5}$$

Allowable bending stress may be increased by a factor of 1.33 for overloads in AASHTO Load Group IB.

Beam size based on bending stress must next be checked for lateral stability. Criteria for lateral stability are based on the frequency of lateral support provided by transverse bracing between beams. Transverse bracing should be provided at each bearing for all spans and at intermediate intervals for spans greater than 20 feet. Maximum intermediate spacing is 25 feet, but bracing is generally spaced at equal intervals over the beam span (lateral bracing configurations are discussed later in this section).

Determine the spacing of transverse bracing and compute allowable bending stress based on stability from the low-variability equations given in Chapter 5. If stability controls over the size factor, it is generally most economical to reduce the unsupported beam length by adding additional bracing. When this is not practical, the beam must be redesigned for the lower stress required for stability.

6. Check live load deflection.

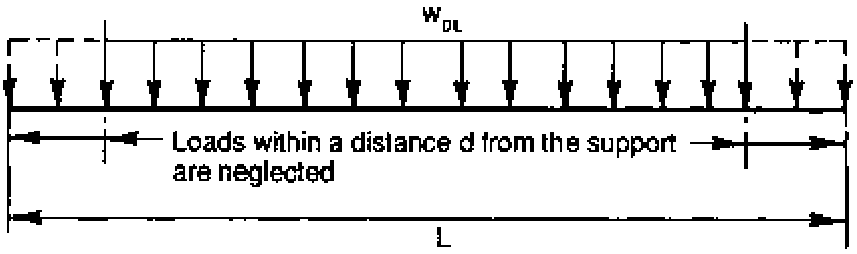
Vehicle deflections are computed from standard methods of engineering analysis. Deflection coefficients for standard AASHTO loads on simple spans are given in Table 16-8.

The distribution of deflection to bridge beams depends on the transverse deck stiffness. On single-lane bridges with glulam decks, it is generally assumed that the deflection produced by one vehicle (two wheel lines) is resisted equally by all beams. On multiple-lane structures, deflection can be distributed using the distribution factor for beam moment, or by assuming that all beams equally resist the deflection produced by the simultaneous loading of one vehicle in each traffic lane. For glulam decks, deflection in multiple-lane bridges is usually distributed using the DF for beam moment.

Compute beam live load deflection and compare it with maximum deflection criteria for the structure. When actual deflection exceeds acceptable levels, the beam moment of inertia, *I*, must be increased. Deflections are important in timber bridges and must be limited for proper performance and serviceability. Excessive deflections loosen connections and cause asphalt wearing surfaces to crack or disintegrate. Criteria for maximum deflection are based on designer judgment, but should not exceed *L/360*. When the structure supports a pedestrian walkway or will be paved with asphalt, a further reduction in deflection is desirable.

7. Check horizontal shear.

Dead load horizontal shear is based on the maximum vertical shear occurring a distance from the support equal to the beam depth, *d*. Compute the dead load vertical shear for interior and outside beams, neglecting loads acting within a distance *d* from the supports:



$$V_{DL} = w_{DL}\left(\frac{L}{2} - d\right) \qquad (7\text{-}6)$$

where V_{DL} = vertical dead load shear at a distance *d* from the support (lb) and

w_{DL} = uniform dead load supported by the beam (lb/in).

Live load vertical shear is computed at the lesser distance of *3d* or *L/4* by Equation 7-1. Applied stress in horizontal shear must not be greater than the allowable stress, as given by

$$f_v = \frac{1.5V}{A} \leq F_v' = F_{vx} C_M \qquad (7\text{-}7)$$

where $V = V_{DL} + V_{LL}$ *(lb),*

A = beam cross-sectional area (in^2), and

C_M = moisture content factor for shear = 0.875.

Allowable shear stress may be increased by a factor of 1.33 for overloads in AASHTO Load Group IB.

When $f_v \leq F_v'$, the beam is adequately proportioned for horizontal shear. If $f > F_v'$, the beam is insufficient in shear and the cross-sectional area must be increased.

8. Check lateral and longitudinal loads.

The applicability and magnitude of lateral and longitudinal loads, such as wind load, longitudinal force, and centrifugal force will vary among different structures. Loads should be computed and applied to affected members in accordance with the AASHTO load groups discussed in Chapter 6. Stresses from AASHTO loading combinations may be increased by stress adjustments for duration of load and those allowed by the specific load group, when applicable.

9. Determine bearing length and stress.

Bearing area at beam reactions must be sufficient to limit stress to an allowable level. Compute the dead load reaction, R_{DL}, at each beam (dead load of beam, deck, wearing surface, railing, and so forth). Compute the live load reaction, R_{LL}, at each beam by multiplying the maximum reaction for one wheel line by the applicable distribution factor for reactions. Maximum reactions for one wheel line of standard AASHTO loads are given in Table 16-8.

For a given beam width, the minimum bearing length must not be less than that computed by

$$\text{Required bearing length} = \frac{(R_{DL} + R_{LL})}{bF_{c\perp}'} \tag{7-8}$$

where R_{DL} = dead load reaction (lb),

R_{LL} = distributed live load reaction (lb),

b = beam width (in), and

$F_{c\perp}' = F_{c\perp x} C_M$ (lb/in²).

Minimum required bearing lengths for the usual $F_{c\perp x}$ = 650 lb/in² are given in Figure 7-10.

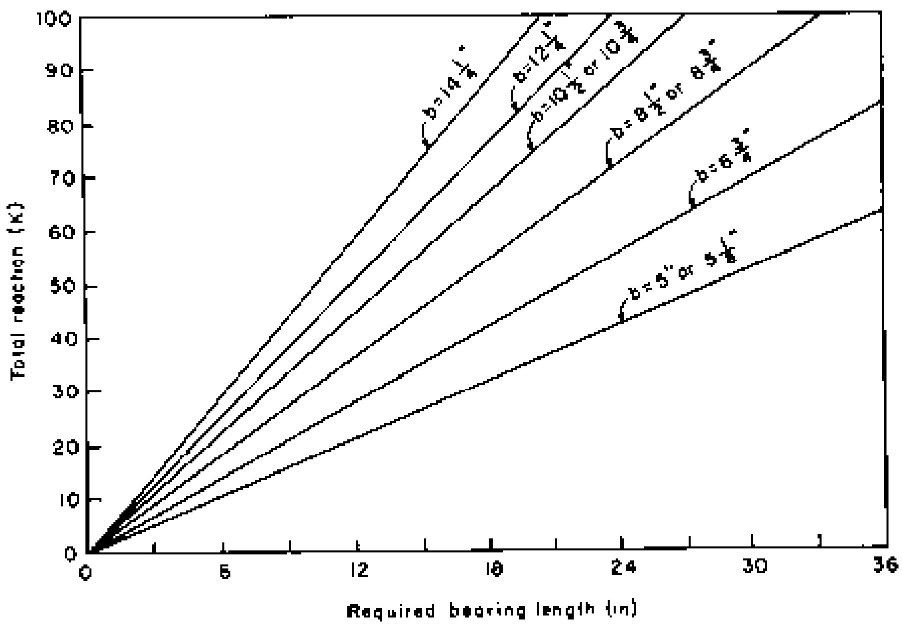


Figure 7-10.—Approximate adjusted minimum bearing length for glulam beams based on an allowable compression perpendicular to grain, $F_{c\perp}'$, of 344.5 lb/in² ($F_{c\perp}' = F_{c\perp x} C_M$ = 650 lb/in² (0.53)).

Values of $F_{c\perp x}$ in *AITC 117-Design* are based on a deformation limit of 0.04 inch and are not subject to increases for duration of load. An increase in allowable stress for overloads may result in additional nonrecoverable deformation at the bearings and is left to designer judgment.

Compute the applied stress at bearings using

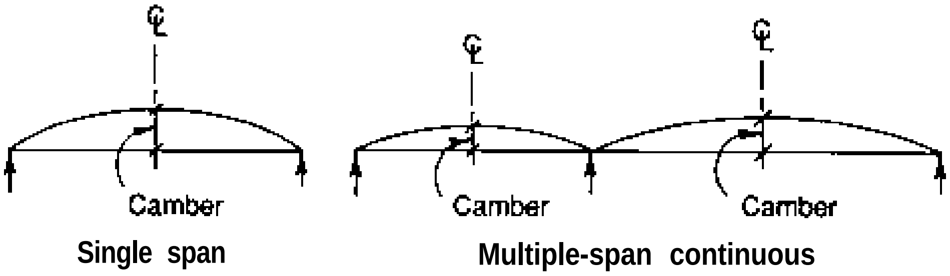
$$f_{c\perp} = \frac{R_{DL} + R_{LL}}{A} \tag{7-9}$$

where A is the bearing area in square inches. This stress must not be greater than $F_{c\perp}'$ computed for Equation 7-8. When bearing is on an inclined surface, refer to Chapter 5 for methods for computing bearing stress.

10. Determine camber.

Camber is based on the span length and configuration of beams. For beams with spans greater than 50 feet, camber is generally 1.5 to 2.0 times the computed dead load deflection (Chapter 5). For spans less than 50 feet, camber is 1.5 to 2.0 times the dead load deflection plus one-half the vehicle live load deflection. Regardless of span, camber on multiple-span beams is normally based on dead load deflections only in order to obtain acceptable riding qualities.

Camber for single-span beams is specified as a vertical offset at the beam centerline. On multiple-span continuous beams, camber may vary along the beam and should be specified at the center of each span segment.



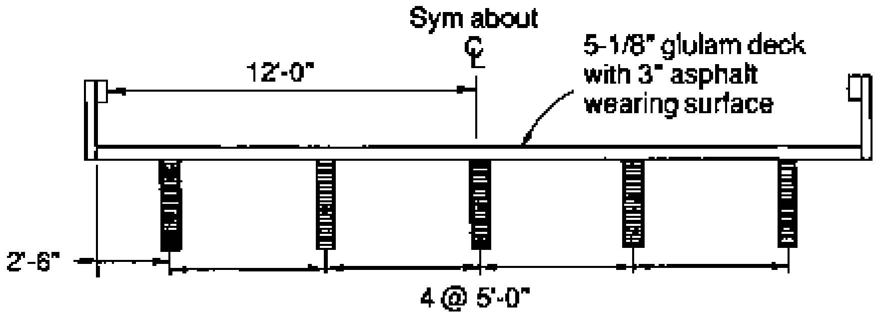
Example 7-3 - Glulam beam design; two-lane highway loading

A deteriorated bridge on a state highway is to be removed and replaced with glulam beam bridge. The new superstructure will be placed on the existing substructure where the span measured center-to-center of bearings is 94 feet. It will carry two traffic lanes and have a roadway width of 24 feet. Design the supporting beams for the structure, assuming the following:

1. A watertight glulam deck constructed of 5-1/8-inch-thick panels with a 3-inch asphalt wearing surface (including allowance for future overlay)
2. AASHTO Load Croup I loading with HS 20-44 vehicles
3. Vehicular railing with an approximate dead load of 45 lb/ft
4. Beams manufactured from visually graded western species

Solution

From the given information, a configuration of five beams spaced 5 feet on center is obtained from Table 7-3. Total deck width is increased 6 inches on each edge to account for rail width and attachment (Chapter 10).



Select a Beam Combination Symbol

A beam combination symbol 24F-V4 manufactured from visually graded western species is selected from *AITC 117--Design.* Tabulated values are as follows:

$$F_{bx} = 2{,}400 \text{ lb/in}^2$$

$$F_{c\perp x} = 650 \text{ lb/in}^2$$

$$F_{vx} = 165 \text{ lb/in}^2$$

$$E_x = 1{,}800{,}000 \text{ lb/in}^2$$

Determine Deck Dead Load and Dead Load Moment

Dead load of the deck and wearing surface is computed in lb/ft^2 based on unit weights of 50 lb/ft^3 for timber and 150 lb/ft^3 for asphalt pavement:

$$\text{DL} = \frac{(5.125 \text{ in.})(50 \text{ lb/ft}^3)}{12 \text{ in./ft}} + \frac{(3 \text{ in.})(150 \text{ lb/ft}^3)}{12 \text{ in./ft}} = 58.9 \text{ lb/ft}^2$$

The dead load applied to each beam is equal to the tributary deck width supported by the beam. In this case, interior beams support 5 feet of deck width. Exterior beams also support 5 feet of deck plus 45 lb/ft of rail dead load.

For interior beams,

$$\text{Deck } w_{DL} = (5.0 \text{ ft})(58.9 \text{ lb/ft}^2) = 294.5 \text{ lb/ft}$$

$$\text{Deck } M_{DL} = \frac{w_{DL}L^2}{8} = \frac{(294.5 \text{ lb/ft})(94 \text{ ft})^2}{8} = 325{,}275 \text{ ft-lb}$$

For outside beams,

$$\text{Deck } w_{DL} = (294.5 \text{ lb/ft}) + 45 \text{ lb/ft} = 339.5 \text{ lb/ft}$$

$$\text{Deck } M_{DL} = \frac{(339.5 \text{ lb/ft})(94 \text{ ft})^2}{8} = 374{,}978 \text{ ft-lb}$$

Determine Live Load Moment

From Table 7-3, the moment DF = 1.0 WL/beam for interior and outside beams. From Table 16-8, the maximum moment for one wheel line of an HS 20-44 truck on a 94-foot span is 708.09 ft-k.

$$M_{LL} = M(\text{DF}) = (708.09 \text{ ft-k})(1.0)(1{,}000 \text{ lb/k}) = 708{,}090 \text{ ft-lb}$$

Determine Beam Size Based on Bending

An initial beam section modulus is computed based on the deck dead load and live load moments (beam dead load is unknown). Because the deck is watertight and beams are protected from direct exposure, dry condition allowable stress is used for bending (C_M = 1.0).

$$F_b' = F_{bx} C_M C_F = (2{,}400 \text{ lb/in}^2)(1.0)(C_F)$$

For interior beams,

$$M = \text{Deck } M_{DL} + M_{LL} = 325{,}275 + 708{,}090 = 1{,}033{,}365 \text{ ft-lb}$$

By Equation 7-3,

$$S_x C_F = \frac{M}{F_b'} = \frac{(1{,}033{,}365 \text{ ft-lb})(12 \text{ in/ft})}{2{,}400 \text{ lb/in}^2} = 5{,}167 \text{ in}^3$$

For outside beams,

$$M = \text{Deck } M_{DL} + M_{LL} = 374{,}978 + 708{,}090 = 1{,}083{,}068 \text{ ft-lb}$$

$$S_x C_F = \frac{M}{F_b'} = \frac{(1{,}083{,}068 \text{ ft-lb})(12 \text{ in/ft})}{2{,}400 \text{ lb/in}^2} = 5{,}415 \text{ in}^3$$

Section modulus requirements differ slightly for interior and outside beams because of the greater load carried by the outside beams. In this case, equal beam depth is desired for even bearing, and beam design will be based on the more severe requirements for outside beams.

Entering Figure 7-9 with an outside beam value $S_x C_F$ = 5,415 in^3, an initial beam size of 12-1/4 by 57 inches is selected. From glulam section properties in Table 16-3,

$$S_x C_F = 5{,}579 \text{ in}^3$$

$$\text{Beam } w_{DL} = 242.4 \text{ lb/ft}$$

Beam dead load moment is computed and $S_x C_F$ revised:

$$\text{Beam } M_{DL} = \frac{w_{DL} L^2}{8} = \frac{242.4\ (94\ \text{ft})^2}{8} = 267{,}731\ \text{ft-lb}$$

$$\text{Revised } S_x C_F = \frac{M}{F_b'} = \frac{(1{,}083{,}068 + 267{,}731)\ (12\ \text{in/ft})}{2{,}400\ \text{lb/in}^2} = 6{,}754\ \text{in}^3$$

From Table 16-3, a revised beam size of 12-1/4 by 64-1/2 inches is selected with the following section properties:

$A = 790.1\ \text{in}^2$

$S_x C_F = 7{,}046.3\ \text{in}^3$

$S_x = 8{,}493.8\ \text{in}^3$

$C_F = 0.83$

$I_x = 273{,}927\ \text{in}^4$

Beam $w_{DL} = 274.3$ lb/ft

Applied moment is revised and bending stress is computed:

$$\text{Beam } M_{DL} = \frac{w_{DL} L^2}{8} = \frac{274.3\ (94\ \text{ft})^2}{8} = 302{,}964\ \text{ft-lb}$$

$M = 302{,}964 + 1{,}083{,}068 = 1{,}386{,}032$ ft-lb

$$f_b = \frac{M}{S_x} = \frac{1{,}386{,}032\ (12\ \text{in/ft})}{8{,}493.8\ \text{in}^3} = 1{,}958\ \text{lb/in}^2$$

$$F_b' = F_{bx} C_M C_F = 2{,}400(1.0)(0.83) = 1{,}992\ \text{lb/in}^2$$

$f_b = 1{,}958\ \text{lb/in}^2 < F_b' = 1{,}992\ \text{lb/in}^2$, so a 12-1/4 by 64-1/2-inch beam is satisfactory in bending.

Check bending stress in interior beams:

M = Beam M_{DL} + (Deck M_{DL} + M_{LL}) = 302,964 + 1,033,365 = 1,336,329 ft-lb

$$f_b = \frac{M}{S_x} = \frac{1{,}336{,}329\ (12\ \text{in/ft})}{8{,}493.8} = 1{,}888\ \text{lb/in}^2 < F_b' = 1{,}992\ \text{lb/in}^2$$

When there is a difference of 200 lb/in^2 or more between beams with the lowest bending stress and the allowable bending stress, a lower glulam combination symbol should be considered. In this case, the difference between interior beam f_b and F_b' is only 104 lb/in^2, so the 12-1/4 by 64-1/2-inch member will be used for all beams.

The beam must next be checked for lateral stability. Assuming a maximum 25-foot spacing between points of lateral support, transverse bracing will be provided at the beam ends and at the quarter points:

$$\ell_u = \frac{L}{4} = \frac{94}{4} = 23.5 \text{ ft} \qquad \text{and} \qquad \frac{\ell_u}{d} = \frac{23.5\,(12 \text{ in/ft})}{64.5} = 4.37 < 14.3$$

By Equation 5-7,

$$\ell_e = 1.63\ell_u + 3d = 1.63(23.5)(12 \text{ in/ft}) + 3(64.5) = 653.16 \text{ in}$$

By Equation 5-3,

$$C_s = \sqrt{\frac{\ell_e d}{b^2}} = \sqrt{\frac{653.16(64.5)}{(12.25)^2}} = 16.76 < 50$$

$C_s > 10$, so further stability calculations are required. As with bending stress, dry conditions of use are assumed for E, and

$$E' = E_x C_M = 1{,}800{,}000(1.0) = 1{,}800{,}000 \text{ lb/in}^2$$

By low-variability Equation 5-12,

$$F_b'' = F_{bx} C_M = 2{,}400(1.0) = 2{,}400 \text{ lb/in}^2$$

$$C_k = 0.956\sqrt{\frac{E'}{F_b''}} = 0.956\sqrt{\frac{1{,}800{,}000}{2{,}400}} = 26.18$$

$C_s = 16.76 < C_k = 26.18$, so the beam is in the intermediate slenderness range. By Equation 5-10,

$$C_L = 1 - \frac{1}{3}\left(\frac{C_s}{C_k}\right)^4 = 1 - \frac{1}{3}\left(\frac{16.76}{26.18}\right)^4 = 0.94$$

$C_L = 0.94 > C_F = 0.83$, so strength rather than stability controls allowable bending stress.

Check Live Load Deflection

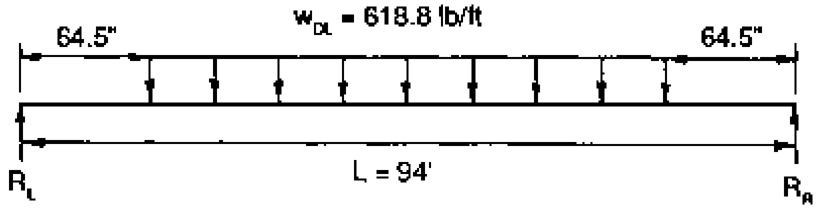
Live load deflection is checked by assuming that deflection is distributed in the same manner as bending: one beam resists the deflection produced by one wheel line. From Table 16-8, the deflection coefficient for one wheel line of an HS 20-44 truck on a 94-foot simple span is 1.02×10^{12} lb-in^3.

$$\Delta_{LL} = \frac{1.02 \times 10^{12}}{E' I_x} = \frac{1.02 \times 10^{12}}{(1{,}800{,}000)(273{,}927)} = 2.07 \text{ in.} = L/545$$

$L/545 < L/360$, so live load deflection is acceptable.

Check Horizontal Shear

From bending calculations, the total dead load for outside beams is 339.5 lb/ft for the deck and railing and 274.3 lb/ft for the beam, for a total of 613.8 lb/ft. Neglecting loads within a distance d = 64.5 inches from the supports, dead load vertical shear is computed by Equation 7-6:

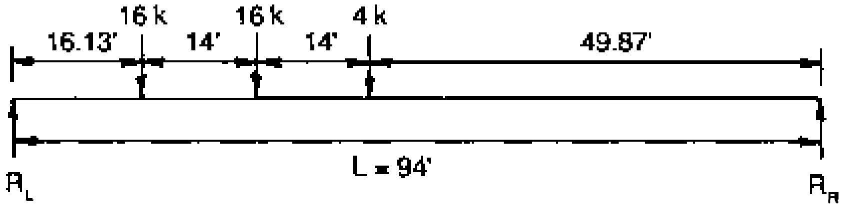


$$V_{DL} = w_{DL}\left(\frac{L}{2} - d\right) = 613.8\left(\frac{94}{2} - \frac{64.5}{12\text{ in/ft}}\right) = 25{,}549\text{ lb}$$

Live load vertical shear is computed from the maximum vertical shear occurring at the lesser of *3d* or *L/4* from the support:

$$3d = \frac{3\,(64.5)}{12\text{ in/ft}} = 16.13\text{ ft} \qquad \frac{L}{4} = \frac{(94)}{4} = 23.5\text{ ft}$$

3d = 16.13 feet controls, and maximum vertical shear is determined at that location for one wheel line of an HS 20-44 truck:



$$V_{LU} = R_L = \frac{(49.87\text{ ft})(4\text{ k}) + (63.87\text{ ft})(16\text{ k}) + (77.87\text{ ft})(16\text{ k})}{94\text{ ft}}$$

V_{LU} = 26.25 k = 26,250 lb

For a moment DF to outside beams of 1.0,

$$V_{LD} = V(\text{Moment DF}) = 26{,}250(1.0) = 26{,}250\text{ lb}$$

$$V_{LL} = 0.50\,[(0.6\,V_{LU}) + V_{LD}]$$

$$= 0.50\,[(0.6)(26{,}250) + 26{,}250] = 21{,}000\text{ lb}$$

$$\text{Total vertical shear} = V_{DL} + V_{LL} = 25{,}549 + 21{,}000 = 46{,}549\text{ lb}$$

Stress in horizontal shear is computed by Equation 7-7:

$$f_v = \frac{1.5V}{A} = \frac{1.5\,(46{,}549)}{790.1} = 88\text{ lb/in}^2$$

$F_v' = F_{vx}(C_M) = (165)(1.0) = 165$ lb/in^2

$F_v' = 165$ lb/in^2 $> f_v = 88$ lb/in^2, so the beam is satisfactory in horizontal shear.

Determine Bearing Length and Stress

Although the watertight deck is assumed to protect the beams from exposure, bearings are subject to wetting from runoff and debris accumulations that trap water. Therefore, bearings will be designed using wet-condition stress in compression perpendicular to grain.

From Table 5-7, $C_M = 0.53$, and

$$F_{c\perp}' = F_{c\perp x}(C_M) = 650(0.53) = 345 \text{ lb/in}^2$$

For a unit dead load w_{DL}, = 613.8 lb/ft to outside beams,

$$R_{DL} = \frac{w_{DL}L}{2} = \frac{(613.8)(94)}{2} = 28{,}849 \text{ lb}$$

For a 2-foot deck overhang and beam spacing of 5 feet, the reaction DF is 1.0 WL/beam for interior and outside beams. From Table 16-8, the maximum reaction for one wheel line of an HS 20-44 truck on a 94-foot span is 32.43 k = 32,430 lb:

$$R_{LL} = R(\text{DF}) = 32{,}430(1.0) = 32{,}430 \text{ lb}$$

By Equation 7-8 (or by Figure 7-10),

$$\text{Required bearing length} = \frac{(R_{DL} + R_{LL})}{b(F_{c\perp}')} = \frac{28{,}849 + 32{,}430}{12.25(345)} = 14.5 \text{ in.}$$

A bearing length of 18 inches is selected. For an out-to-out beam length of 95-1/2 feet, reactions are revised and applied stress is computed by Equation 7-9:

$$R_{DL} = \frac{613.8(95.5)}{2} = 29{,}309 \text{ lb}$$

$$f_{c\perp} = \frac{R_{DL} + R_{LL}}{A} = \frac{29{,}309 + 32{,}430}{12.25(18)}$$

$$= 280 \text{ lb/in}^2 < F_{c\perp}' = 345 \text{ lb/in}^2$$

Determine Camber

Dead load deflection is computed by Equation 5-16:

$$\Delta_{DL} = \frac{5wL^4}{384E'I_x} = \frac{5(613.8)[(94)(12 \text{ in./ft})]^4}{384(1{,}800{,}000)(273{,}927)(12 \text{ in./ft})} = 2.19 \text{ in.}$$

Using camber slightly greater than twice the dead load deflection, a minimum midspan offset of 5 inches will be specified.

Summary

The superstructure will consist of five 12-1/4-inch-wide by 64-1/2-inch-deep glulam beams, 95-1/2 feet long, with a distance center to center of bearings of 94 feet. Transverse bracing will be provided for lateral support at the bearings and at the beam quarter points. The glulam will be specified as visually graded western species conforming to combination symbol 24F-V4, or may be specified by required stresses as outlined in *AITC 117--Design.*

Stresses and deflection are as follows:

	Interior beams	**Outside beams**
f_b	1,888 lb/in^2	1,958 lb/in^2
F_b'	1,992 lb/in^2	1,992 lb/in^2
Δ_{LL}	2.07 in. = L/545	2.07 in. = L/545
f_v	< Outside beam	88 lb/in^2
F_v'	165 lb/in^2	165 lb/in^2
$f_{c\perp}$	< Outside beam	280 lb/in^2
$F_{c\perp}'$	345 lb/in^2	345 lb/in^2

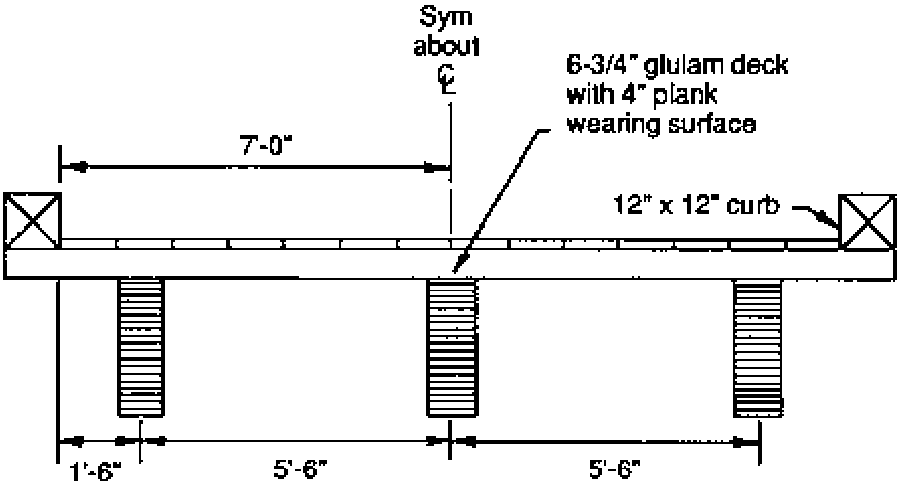
Example 7-4 - Glulam beam design; single-lane with overload

A new bridge on a local rural road will span 48 feet center-to-center of bearings. It will carry one traffic lane and have a roadway width of 14 feet. Design the supporting glulam beams for the structure, assuming

1. a nonwatertight deck constructed of 6-3/4-inch glulam panels with a 4-inch rough-sawn lumber wearing surface;
2. AASHTO Load Group I loading with an H 20-44 vehicle and AASHTO Group IB loading with a U80 overload (Figure 6-5);
3. a 12- by 12-inch rough-sawn brush curb along each deck edge; and
4. beams manufactured from visually graded Southern Pine.

Solution

A configuration of three beams spaced 5-1/2 feet on center is obtained from Table 7-3. Deck width is increased 1 foot on each edge to account for the brush curb:



Select a Beam Combination Symbol

A beam combination symbol 24F-V2 is selected from *AITC 117--Design.* Tabulated values are as follows:

$$F_{bx} = 2{,}400 \text{ lb/in}^2 \qquad C_M = 0.80$$

$$F_{c\perp x} = 650 \text{ lb/in}^2 \qquad C_M = 0.53$$

$$F_{vx} = 200 \text{ lb/in}^2 \qquad C_M = 0.875$$

$$E_x = 1{,}700{,}000 \text{ lb/in}^2 \quad C_M = 0.833$$

Compute Deck Dead Load and Dead Load Moment

Deck and wearing surface dead loads are computed as follows:

$$\text{Deck DL} = \frac{(6.75 \text{ in.})(50 \text{ lb/ft}^3)}{12 \text{ in./ft}} = 28.1 \text{ lb/ft}^2$$

$$\text{Wearing surface DL} = \frac{(4 \text{ in.})(50 \text{ lb/ft}^3)}{12 \text{ in./ft}} = 16.7 \text{ lb/ft}^2$$

The interior beam supports a 5.5-foot width of deck and wearing surface, while exterior beams support 5.25 feet of deck, 4.25 feet of wearing surface, and 50 lb/ft of curb.

For interior beams,

$$w_{DL} = (5.5)(28.1 + 16.7) = 246.4 \text{ lb/ft}$$

$$M_{DL} = \frac{w_{DL}L^2}{8} = \frac{(246.4)(48)^2}{8} = 70{,}963 \text{ ft-lb}$$

For outside beams,

$$w_{DL} = (5.25)(28.1) + (4.25\text{ ft})(16.7) + 50\text{ lb/ft} = 268.5\text{ lb/ft}$$

$$M_{DL} = \frac{(268.5)(48)^2}{8} = 77{,}328\text{ ft-lb}$$

Determine Live Load Moment

From Table 7-3, the moment DF = 0.92 WL/beam for interior and outside beams. Maximum live load moments per wheel line are obtained from Table 16-8 and are multiplied by the moment DF:

H 20-44 M_{LL} = 0.92 (212,820 ft-lb) = 195,794 ft-lb

U80 M_{LL} = 0.92 (572,590 ft-lb) = 526,783 ft-lb

Determine Beam Size Based on Bending

For a U80 overload, the tabulated bending stress can be increased 33 percent in AASHTO Load Group IB. Comparing the U80 moment to the lesser H 20-44 moment,

$$\frac{526{,}783}{1.33} = 396{,}077 > 195{,}794$$

so the U80 will control bending.

$$F_b' = F_{bx}(1.33)C_M C_F = 2{,}400(1.33)(0.80)(C_F) = 2{,}554(C_F)\text{ lb/in}^2$$

For outside beams,

$$M = \text{Deck } M_{DL} + M_{LL} = 77{,}328 + 526{,}783 = 604{,}111\text{ ft-lb}$$

$$S_x C_F = \frac{M}{F_b'} = \frac{604{,}111\text{ ft-lb}\,(12\text{ in/ft})}{2{,}554\text{ lb/in}^2} = 2{,}838\text{ in}^3$$

Entering Figure 7-9 with a value $S_x C_F$ = 2,838 in^3, an approximate beam size of 8-1/2 by 51 inches is selected. From Table 16-4,

$S_x C_F$ = 2,965.5 in^3

Beam w_{DL} = 146.1 lb/ft

$$\text{Beam } M_{DL} = \frac{w_{DL}L^2}{8} = \frac{146.1(48)^2}{8} = 42{,}077\text{ ft-lb}$$

Revising section modulus requirements,

$$S_x C_F = \frac{M}{F_b'} = \frac{(604{,}111 + 42{,}077)(12\text{ in/ft})}{2{,}554\text{ lb/in}^2} = 3{,}036\text{ in}^3$$

From Table 16-4, a revised beam size of 8-1/2 by 50-7/8 inches is chosen with the following section properties:

$A = 432.4 \text{ in}^2$

$S_x C_F = 3{,}123.0 \text{ in}^3$

$S_x = 3{,}666.7 \text{ in}^3$

$C_F = 0.85$

$I_x = 93{,}271.9 \text{ in}^4$

Beam $w_{DL} = 150.2$ lb/ft

$$\text{Beam } M_{DL} = \frac{w_{DL}L^2}{8} = \frac{150.2(48)^2}{8} = 43{,}258 \text{ ft-lb}$$

$$M = 43{,}258 + 604{,}111 = 647{,}369 \text{ ft-lb}$$

$$f_b = \frac{M}{S_x} = \frac{647{,}369(12 \text{ in/ft})}{3{,}666.7 \text{ in}^3} = 2{,}119 \text{ lb/in}^2$$

$$F_b' = F_{bx}(1.33)C_M C_F = 2{,}554(0.85) = 2{,}171 \text{ lb/in}^2$$

$f_b < F_b'$, therefore an 8-1/2 by 50-7/8-inch outside beam is sufficient in bending.

Check U80 bending stress in interior beams:

$$M = M_{DL} + M_{LL} = (43{,}258 + 70{,}963) + 526{,}783 = 641{,}004 \text{ ft-lb}$$

$$f_b = \frac{M}{S_x} = \frac{641{,}004 (12 \text{ in/ft})}{3{,}666.7 \text{ in}^3} = 2{,}098 \text{ lb/in}^2$$

The difference between interior beam f_b and F_b' is only 73 lb/in^2, so an 8-1/2- by 50-7/8-inch 24F-V2 will be used for all beams.

Check outside beam bending stresses for the H 20-44 load:

$$F_b' = F_{bx} C_M C_F = 2{,}400(0.80)(0.85) = 1{,}632 \text{ lb/in}^2$$

$$M = M_{DL} + M_{LL} = (43{,}258 + 77{,}328) + 195{,}794 = 316{,}380 \text{ ft-lb}$$

$$f_b = \frac{M}{S_x} = \frac{316{,}380 (12 \text{ in/ft})}{3{,}666.7 \text{ in}^3} = 1{,}035 \text{ lb/in}^2 < F_b' = 1{,}632 \text{ lb/in}^2$$

Check lateral stability assuming lateral support at beam ends and centerspan:

$$\ell_u = \frac{L}{2} = \frac{48}{2} = 24 \text{ ft} \qquad \frac{\ell_u}{d} = \frac{24\,(12 \text{ in/ft})}{50.88} = 5.66 < 14.3$$

$$\ell_e = 1.63\ell_u + 3d = 1.63(24)(12 \text{ in/ft}) + 3(50.88) = 622.08$$

$$C_s = \sqrt{\frac{\ell_e d}{b^2}} = \sqrt{\frac{(622.08)(50.88)}{(8.5)^2}} = 20.93 < 50$$

$C_s > 10$, so further stability calculations are required.

$$E' = E_x C_M = 1{,}700{,}000(0.833) = 1{,}416{,}100 \text{ lb/in}^2$$

By low-variability Equation 5-12,

$$F_b'' = F_{bx} C_M = 2{,}400(0.80) = 1{,}920 \text{ lb/in}^2$$

$$C_k = 0.956\sqrt{\frac{E'}{F_b''}} = 0.956\sqrt{\frac{1{,}416{,}100}{1{,}920}} = 25.96$$

$C_s = 20.93 < C_k = 25.96$, so the beam is in the intermediate slenderness range. By Equation 5-10,

$$C_L = 1 - \frac{1}{3}\left(\frac{C_s}{C_k}\right)^4 = 1 - \frac{1}{3}\left(\frac{20.93}{25.96}\right)^4 = 0.86$$

$C_L = 0.86 > C_F = 0.85$, so strength rather than stability controls the allowable bending stress and an 8-1/2- by 50-7/8-inch beam is satisfactory.

Check Live Load Deflection

Live load deflection for this single-lane configuration will be checked by assuming deflection is equally resisted by all beams. Criteria for the H 20-44 vehicle will be a maximum deflection of *L*/360. For the U80 overload, no criteria will apply, but deflection will be computed for reference.

For the H 20-44 vehicle, the deflection coefficient from Table 16-8 for one wheel line on a 48-foot simple span is 7.40×10^{10} lb-in^3. Deflection is computed by assuming that all beams equally resist the deflection produced by one truck (two wheel lines):

$$\Delta_{LL} = \frac{2\left(7.40 \times 10^{10}\right)}{E_x(C_M)(3)I_x} = \frac{1.48 \times 10^{11}}{1{,}700{,}000(0.833)(3)(93{,}272.9)} = 0.37 \text{ in.}$$

0.37 in. = $L/1{,}557 < L/360$ allowed.

For the U80 vehicle, the deflection coefficient from Table 16-8 for one wheel line is 2.35×10^{11} lb-in^3, and

$$\Delta_{LL} = \frac{2(2.35\times10^{11})}{E_s(C_M)(3)I_x} = \frac{4.70\times10^{11}}{1{,}700{,}000(0.833)(3)(93{,}272.9)} = 1.19 \text{ in.}$$

which is approximately equal to *L/484.*

Live load deflection is acceptable.

Check Horizontal Shear

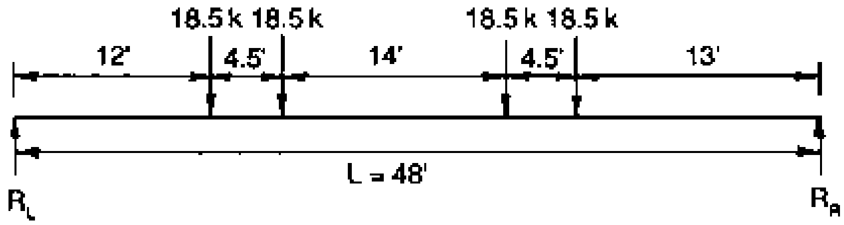
From bending calculations the total outside-beam dead load is 268.5 lb/ft for the deck and curb and 150.2 lb/ft for the beam, for a total load of 418.7 lb/ft. Neglecting loads within a distance of *d* = 50-7/8 inches from the supports, dead load vertical shear is computed by Equation 7-6:

$$V_{DL} = w_{DL}\left(\frac{L}{2}-d\right) = 418.7\left(\frac{48}{2}-\frac{50.88}{12 \text{ in/ft}}\right) = 8{,}274 \text{ lb}$$

Live load vertical shear is computed from the maximum vertical shear occurring at the lesser of *3d* or *L/4* from the support:

$$3d = \frac{3(50.88)}{12 \text{ in/ft}} = 12.72 \text{ ft} \qquad \frac{L}{4} = \frac{(48)}{4} = 12 \text{ ft}$$

L/4 = 12 feet controls, and maximum vertical shear is computed at that location for one wheel line of a U80 truck:



$$V_{LU} = R_L\frac{(13\text{ft}+4.5\text{ft}+14/2\text{ft})(74\text{k})}{48\text{ft}} = 37.77\text{k} = 37{,}770\text{lb}$$

$$V_{LD} = V(\text{DF}) = 37{,}770(0.92) = 34{,}748 \text{ lb}$$

$$V_{LL} = 0.50\,[(0.6\,V_{LU}) + V_{LD}]$$

$$= 0.50\,[(0.6)(37{,}770) + 34{,}748] = 28{,}705 \text{ lb}$$

$$V = V_{DL} + V_{LL} = 8{,}274 + 28{,}705 = 36{,}979 \text{ lb}$$

Stress in horizontal shear is computed by Equation 7-7:

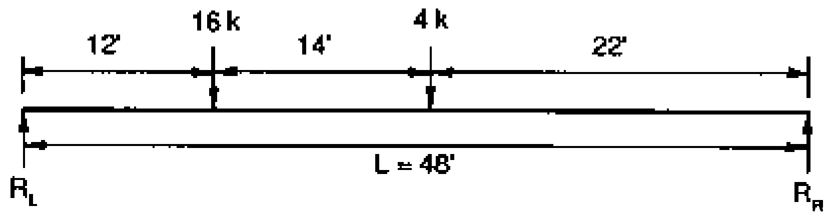
$$f_v = \frac{1.5V}{A} = \frac{1.5(36{,}979)}{432.4} = 128 \text{ lb/in}^2$$

$F_v' = F_{vx}(1.33)(C_M) = (200)(1.33)(0.875) = 233$ lb/in^2

$f_v = 128$ lb/in$^2 < F_v' = 233$ lb/in^2, so horizontal shear is acceptable.

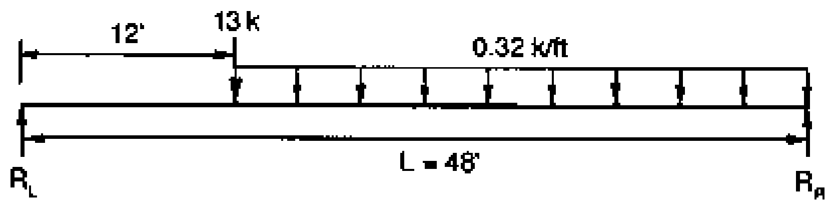
For reference, check shear for H 20-44 loading.

For truck loading,



$$V_{LU} = R_L = \frac{(22\text{ ft})(4\text{ k}) + (36\text{ ft})(16\text{ k})}{48\text{ ft}} = 13.83\text{ k} = 13{,}833\text{ lb}$$

For one-half lane loading (one wheel line),



$$V_{LU} = R_L \frac{(36\text{ ft})(13\text{ k}) + (36\text{ ft})(0.32\text{ k/ft})(18\text{ ft})}{48\text{ ft}}$$

$= 14.07$ k $= 14{,}070$ lb

H 20-44 shear stress is computed for the controlling lane load:

$V_{LD} = V(\text{DF}) = 14{,}070(0.92) = 12{,}944$ lb

$V_{LL} = 0.50\ [(0.6\ V_{LU}) + V_{LD}]$

$= 0.50\ [(0.6)(14{,}070) + 12{,}944] = 10{,}693$ lb

$$V = V_{DL} + V_{LL} = 8{,}274 + 10{,}693 = 18{,}967\text{ lb}$$

$$f_v = \frac{1.5V}{A} = \frac{1.5(18{,}967)}{432.4} = 66\text{ lb/in}^2$$

$F_v' = F_{vx}(C_M) = (200)(0.875) = 175$ lb/in$^2 > 66$ lb/in^2

The beam is satisfactory in horizontal shear.

Determine Bearing Length and Stresses

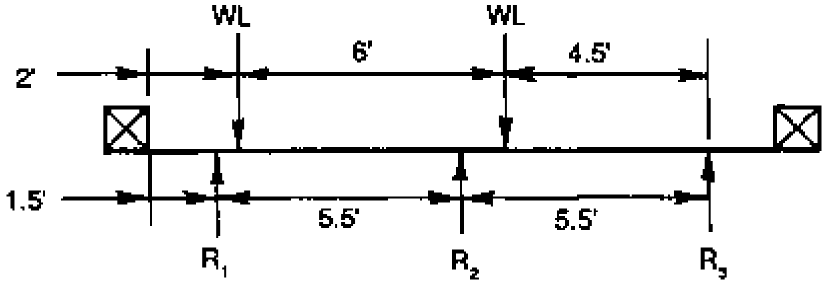
Bearing design will be based on the heavier U80 loading without the 33-percent stress increase for overloads.

$$F_{c\perp}' = F_{c\perp} (C_M) = 650(0.53) = 345 \text{ lb/in}^2$$

For a unit dead load w = 418.7 lb/ft,

$$R_{DL} = \frac{wL}{2} = \frac{(418.7)(48)}{2} = 10{,}049 \text{ lb}$$

Reaction distribution factors are computed by placing the wheel line two feet from the curb face. For this single-lane bridge, one vehicle position is used for interior and outside beam distribution factors:



Assuming that the deck acts as a simple span between supports,

$$\text{Interior beam reaction DF} = \frac{(4.5 \text{ ft})(\text{WL})}{5.5 \text{ ft}} = 0.82 \text{ WL/beam}$$

$$\text{Outside beam reaction DF} = \frac{(5 \text{ ft})(\text{WL})}{5.5 \text{ ft}} = 0.91 \text{ WL/beam}$$

From Table 16-8, the maximum reaction for one wheel line of a U80 vehicle on a 48-foot span is 57,650 pounds. For the controlling outside beams,

R_{LL} = 57,650(0.91) = 52,461 lb

$$\text{Required bearing length} = \frac{R_{DL} + R_{LL}}{b\,(F_{c\perp}')} = \frac{10{,}049 + 52{,}461}{8.5(345)} = 21.3 \text{ in.}$$

A bearing length of 24 inches will be used for an out-to-out beam length of 50 feet:

$$R_{DL} = \frac{wL}{2} = \frac{(418.7)(50)}{2} = 10{,}468 \text{ lb}$$

$$f_{c\perp} = \frac{R_{DL} + R_{LL}}{A} = \frac{10{,}468 + 52{,}461}{8.5(24)} = 308 \text{ lb/in}^2 = 345 \text{ lb/in}^2$$

Determine Camber

Dead load deflection is computed by Equation 5-16:

$$\Delta_{DL} = \frac{5wL^4}{384E'I_x} = \frac{5(418.7)[(48)(12 \text{ in./ft})]^4}{384(1,416,100)(93,272)(12 \text{ in./ft})} = 0.38 \text{ in.}$$

Camber of 1 inch will be specified at centerline, which is approximately 2-1/2 times the dead load deflection.

Summary

The superstructure will consist of three 8-1/2 by 50-7/8-inch glulam beams, 50 feet long, with a distance center to center of bearings of 48 feet. Transverse bracing will be provided for lateral support at the bearings and at midspan. The glulam will be specified as visually graded Southern Pine conforming to combination symbol 24F-V2, or may be specified by required stresses as outlined in *AITC 117--Design.* Stresses and deflection for controlling outside beams are as follows:

	H 20-44 loading	**U80 loading**
f_b	1,035 lb/in^2	2,119 lb/in^2
F_b'	1,632 lb/in^2	2,171 lb/in^2
Δ_{LL}	0.37 in. = L/1,557	1.19 in. = L/484
f_v	66 lb/in^2	128 lb/in^2
F_v'	175 lb/in^2	233 lb/in^2
$f_{c\perp}$	< U80	308 lb/in^2
$F_{c\perp}'$	345 lb/in^2	345 lb/in^2

DESIGN OF TRANSVERSE BRACING

Beams must be transversely braced to provide lateral strength and rigidity to the members. In bridge applications, beam bracing is provided to maintain the relative spacing of beams during construction and in service, laterally support the beam compression zone, and distribute lateral loads such as wind and centrifugal loads from the superstructure to the bearings. It is recommended by AASHTO that transverse bracing be provided at the bearings for all span lengths and at intermediate locations for spans longer than 20 feet. The spacing of intermediate bracing is based on requirements for lateral beam support, but should not exceed 25 feet. Although some lateral beam support and load distribution are also provided by the deck, these effects vary with the type of deck attachment and are normally neglected in design.

Bracing for glulam beams generally consists of cross frames or diaphragms placed normal to the longitudinal beam axes and stepped for skewed crossings (Figure 7-11). Cross frames are constructed of welded steel angles, a minimum of 5/16 inch thick, that are galvanized after fabrication (Figure 7-12). They are economical, lightweight, and are completely prefabricated for easy field erection. Cross frame design is based on the design requirements for structural steel given in AASHTO specifications. The size of the steel angles must be sufficient to resist applied loads and provide sufficient width for attachment bolts and hardware. Diaphragms are solid glulam blocks placed vertically between the beams (Figure 7-13). In most cases, the beams are held against the diaphragms by steel tie rods that pass through the beams on alternate sides of the diaphragm.[7] Diaphragms are more effective in laterally distributing wheel loads to beams, but diaphragms are heavier and more difficult to erect than cross frames.

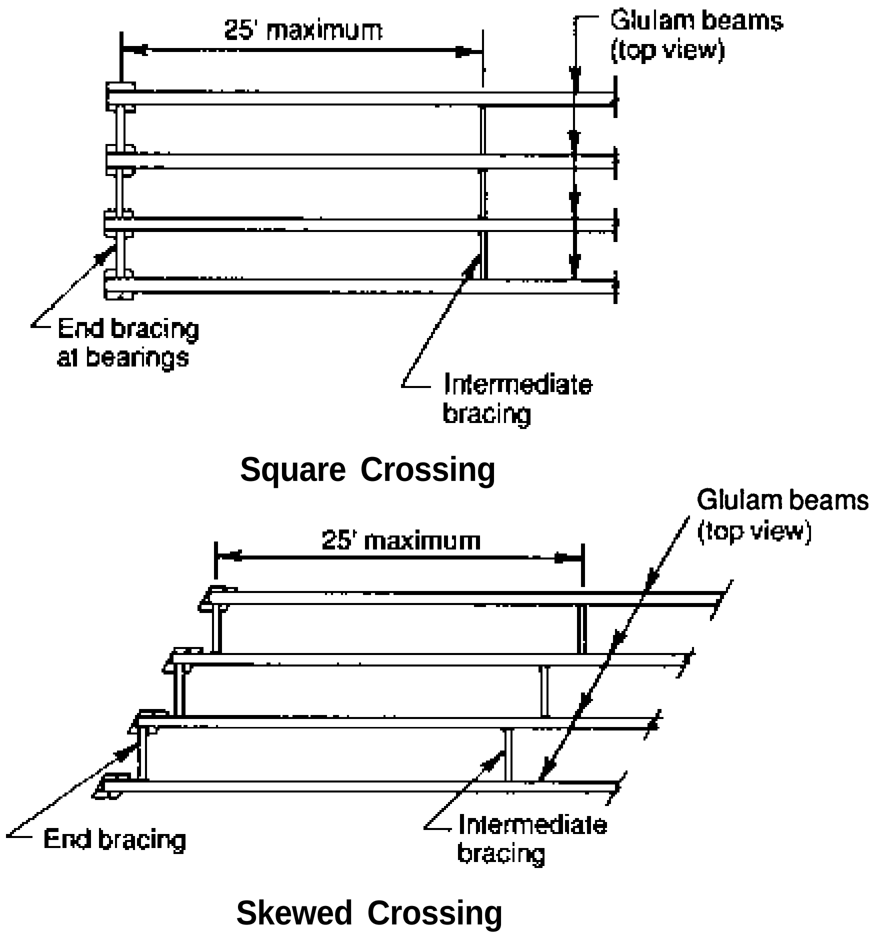


Figure 7-11. - Transverse bracing configurations for glulam beams.

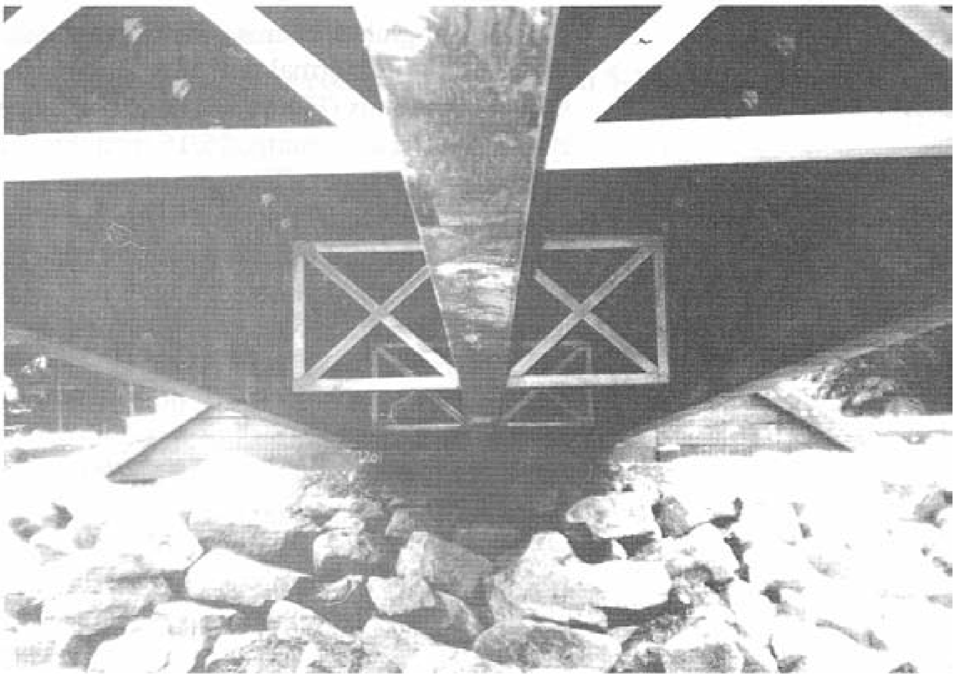


Figure 7-12. - Transverse beam bracing constructed of welded-steel cross frames (photo courtesy of Tim Chittenden, USDA Forest Service).

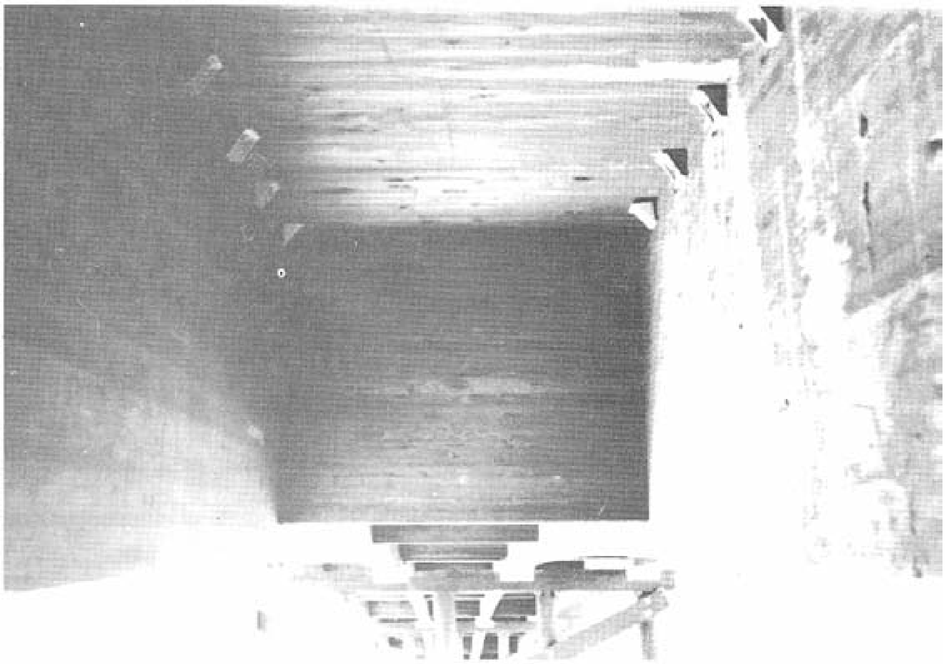


Figure 7-13. - Transverse beam bracing constructed of solid glulam diaphragms.

Cross frames and diaphragms are designed to be as deep as practical to provide support for lateral loads over the entire beam depth. They are typically designed for the most severe loading at the bearings and the same configuration is used at intermediate points, although loading at these locations may be somewhat less. The top of the bracing should be 2 to 5 inches below the deck to ensure air circulation and clearance from deck attachment hardware. The lower beam connection should be inside the outer tension zone of the beam, which is generally considered to be the lower 10 percent of the beam depth (in areas of negative bending, this applies to the beam top). Bracing at bearings should extend to the top of the bearing shoe but not conflict with bearing anchor-bolt placement. Bolted connections between the bracing and the beam should also permit minor vertical movement of the beam from variations in moisture content. Two or more bolts rigidly connecting bracing to a beam at widely spaced points can restrain vertical beam shrinkage and may cause splitting, if shrinkage occurs.

DESIGN OF BEARINGS

Bearings support the bridge beams and transmit vertical, longitudinal, and transverse loads from the superstructure to the substructure. The two general types of bearings used are fixed bearings and expansion bearings. Fixed bearings are designed to prevent beam movement in the longitudinal direction. Expansion bearings allow longitudinal movement and are used when the superstructure will expand or contract because of thermal changes or deflection. Both types of bearings prevent transverse movement but allow small beam rotations at the support. For most timber bridges, longitudinal movement is insignificant, and fixed bearings are used. Nevertheless, expansion bearings may be required for exceptionally long spans or when thermal movement of other material such as steel or concrete must be considered.

A typical bearing for timber beams consists of four components: bearing shoe, bearing pad, beam attachment bolts, and anchor bolts (Figure 7-14). Design of these components is based on the direction and magnitude of loads transmitted by the superstructure. The bearing must be capable of distributing vertical loads from dead load and vehicle live load (including uplift when applicable), and lateral loads from sources such as wind, seismic forces, centrifugal forces, and vehicle braking.

Bearing Shoe

The bearing shoe is a bracket constructed of a welded steel plate or angles that connects the beams to the substructure (Figure 7-15). The plate configuration includes a base plate and is most commonly used for spans of approximately 50 feet or more. The angle configuration may be used for longer spans but is generally most suited for spans shorter than approximately 50 feet. A base plate for the angle configuration is optional, but is commonly used when bearing is on a timber cap or sill.

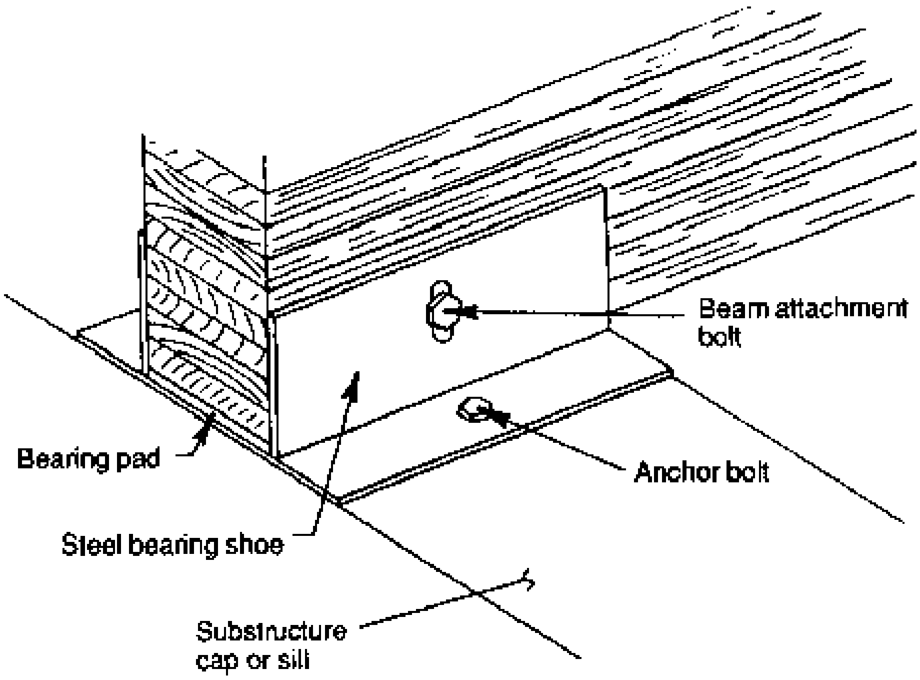


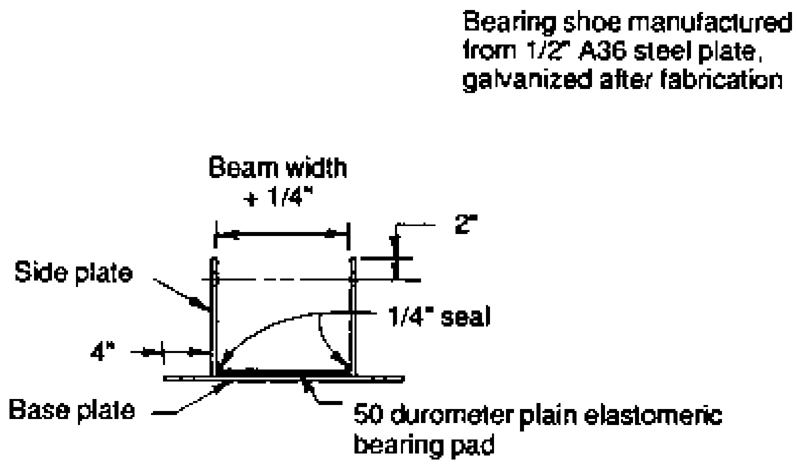
Figure 7-14. - Typical fixed-bearing configuration for glulam beams.

The size of the bearing shoe depends on the beam size and required length of bearing. Minimum length is the required beam bearing length. The width between side plates is the beam width plus 1/4 inch. The height of the side plates must be sufficient to resist transverse loads and locate the beam attachment bolt a minimum or four times, but preferably five times, the bolt diameter above the base of the beam. When the bearings are subject to uplift, the minimum height of the attachment bolt is seven times the bolt diameter.

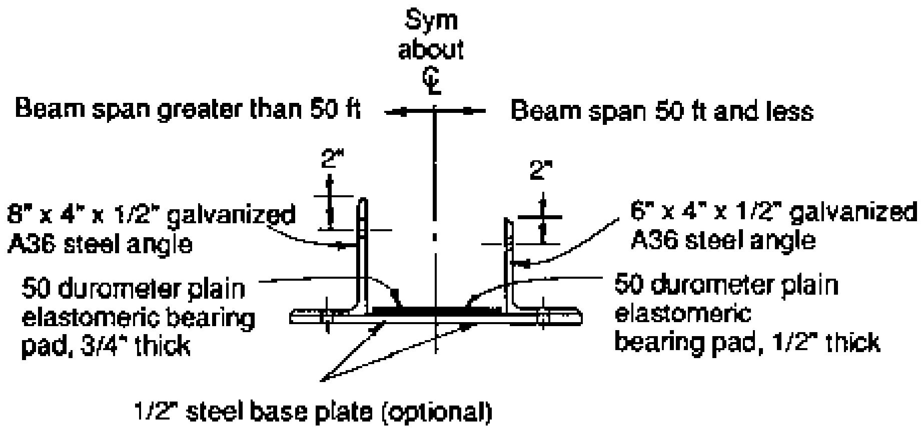
Bearing Pad

A bearing pad is a thin pad of elastomeric rubber (usually neoprene) placed between the beam and the support. For timber bridges, the purpose of the pad is to allow slight movement and rotation of the beam through deformation of the pad, provide a smooth bearing surface and compensate for irregularities in the bearing surfaces, and elevate the beam above the sill or cap where water may collect.

Bearing pad size depends on the bearing area of the beam. Pads are equal in length to the beam bearing length and are 1/4 inch narrower than the beam width. Pad thickness depends on the type of bearing, whether fixed or expansion. For fixed bearings, pads are typically 1/2 inch thick for spans shorter than 50 feet, and 3/4 to 1 inch thick for spans longer than 50 feet. For expansion bearings, pad thickness is based on the anticipated



Steel plate bearing shoe



Steel angle bearing shoe

Figure 7-15. - Typical bearing shoe details for glulam beams.

movement of the superstructure, and must be based on design criteria given in AASHTO for elastomeric bearings (AASHTO Section 14). In both cases, a pad with nominal 50 or 60 durometer hardness is recommended.

Beam Attachment Bolts

Beam attachment bolts connect the beams to the bearing shoe and transmit longitudinal and uplift forces from the superstructure. Minimum recommended bolt diameters are 3/4 inch for spans up to approximately 50 feet and 1 inch for spans longer than 50 feet. For most designs, one bolt at the center of the bearing length is adequate; however, the number and diameter of bolts should be based on the magnitude and direction of applied loads.

Beam attachment bolts are placed in round holes bored through the beam before preservative treatment. Holes in the bearing shoe are slotted or round depending on the type of bearing and direction of vertical forces. For fixed bearings without uplift, holes are generally slotted vertically to allow for construction tolerances and permit the beam to rotate slightly at the support. When fixed bearings are subjected to uplift, holes are round. For expansion bearings, holes are slotted horizontally to allow longitudinal beam movement.

Anchor Bolts

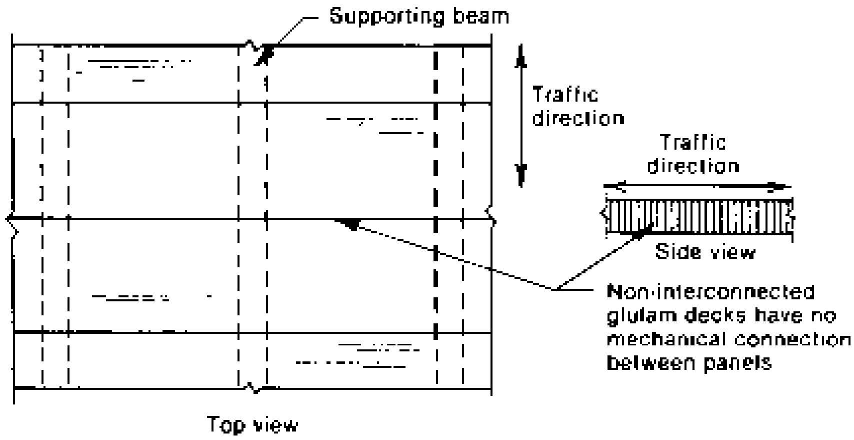
Anchor bolts transmit vertical and lateral loads from the bearing shoe to the substructure. On steel and concrete substructures, anchor bolts are normally machine bolts or studs. On timber substructures, lag screws may be used. Anchor bolts are typically placed through round holes in the bearing shoe, but slotted holes may be used at the option of the designer to allow for construction tolerances.

The number and diameter of anchor bolts depends on load magnitude and bolt capacity. As a minimum, two bolts are provided at each bearing, one on each side of the beam. Recommended minimum diameters are 3/4 inch for spans 50 feet or shorter and 1 inch for spans longer than 50 feet. Additional bolts or increased bolt diameters may be required depending on the magnitude of transmitted loads.

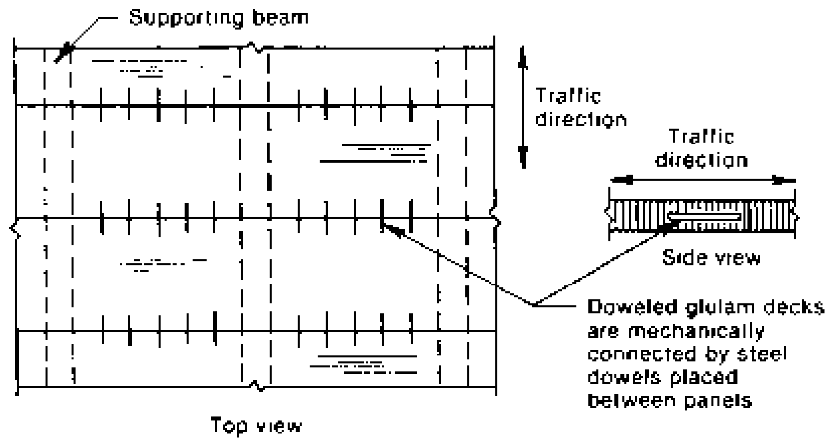
7.5 DESIGN OF GLULAM DECKS

Glulam decks are constructed of panels manufactured of vertically laminated lumber. The panels are placed transverse to the supporting beams, and loads act parallel to the wide face of the laminations. The two basic types of glulam decks are the noninterconnected deck and the doweled deck (Figure 7-16). Noninterconnected decks have no mechanical connection between adjacent panels. Doweled decks are interconnected with steel dowels to distribute loads between adjacent panels. Both deck types are stronger and stiffer than conventional nail-laminated lumber or plank decks, resulting in longer deck spans, increased spacing of supporting beams, and reduced live load deflection. Additionally, glulam panels can be placed to provide a watertight deck, protecting the structure from the deteriorating effects of rain and snow.

Glulam decks are manufactured from visually graded western species or Southern Pine sawn lumber using the same lumber grade throughout. Any of several axial combination symbols in Table 2 of *AITC 117--Design* may be used. The three most frequently used combination symbols for each species are listed in Table 7-5. Combination symbols with a tabulated bending stress of 1,800 lb/in^2 or less are the most economical and most commonly used.



Non-interconnected glulam deck



Doweled glulam deck

Figure 7-16. - Configurations for noninterconnected and doweled glulam decks.

Table 7-5 - Glulam axial combination symbols commonly used for bridge decks.

Western species			Southern Pine		
Combination symbol	Grade	F_t	Combination symbol	Grade	F_t
1	L3	1,450	46	N3M	1,450
2	L2	1,800	47	N2M	1,750
3	L2D	2,100	48	N2D	2,000

Glulam decks are generally 5-1/8 inches (5 inches for Southern Pine) or 6-3/4 inches thick. Increased thicknesses up to 14-1/4 inches are available, but are seldom required (design aids in this section are limited to decks 8-3/4 inches thick or less). Panel width is a multiple of 1-1/2 inches, the net width of the individual lumber laminations. The practical width of panels ranges from approximately 30 to 55 inches; however, the designer should check local manufacturing and treating limitations before specifying widths over 48 inches. Panels can be manufactured in any specified length to be continuous across the structure. It is common practice to vary adjacent panel lengths to provide a drainage opening under curbs (Figure 7-17).

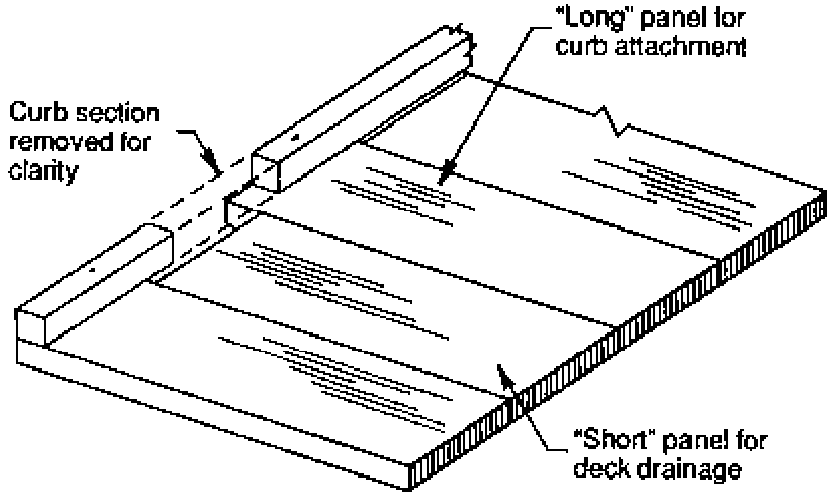


Figure 7-17. - The length of glulam deck panels may be varied between adjacent panels to provide a drainage opening under the curb.

The performance and economy of glulam deck panels can be significantly affected by the configuration and materials specified in design. The most economical design is one that uses a modular-type system with two or three standardized panels in a repetitious arrangement. Panel width and configuration are usually based on criteria for curb or railing systems (Chapter 10). When the bridge length is not evenly divisible by the selected panel width, odd-width panels are placed on the approach ends of the deck.

NONINTERCONNECTED GLULAM DECKS

Noninterconnected glulam decks are the most widely used type of glulam deck in modern timber bridge construction (Figure 7-18). They are economical, require little fabrication, and are easy to install with unskilled labor and without special equipment. Because the panels are not connected to one another, each panel acts individually to resist the stresses and deflection from applied loads.

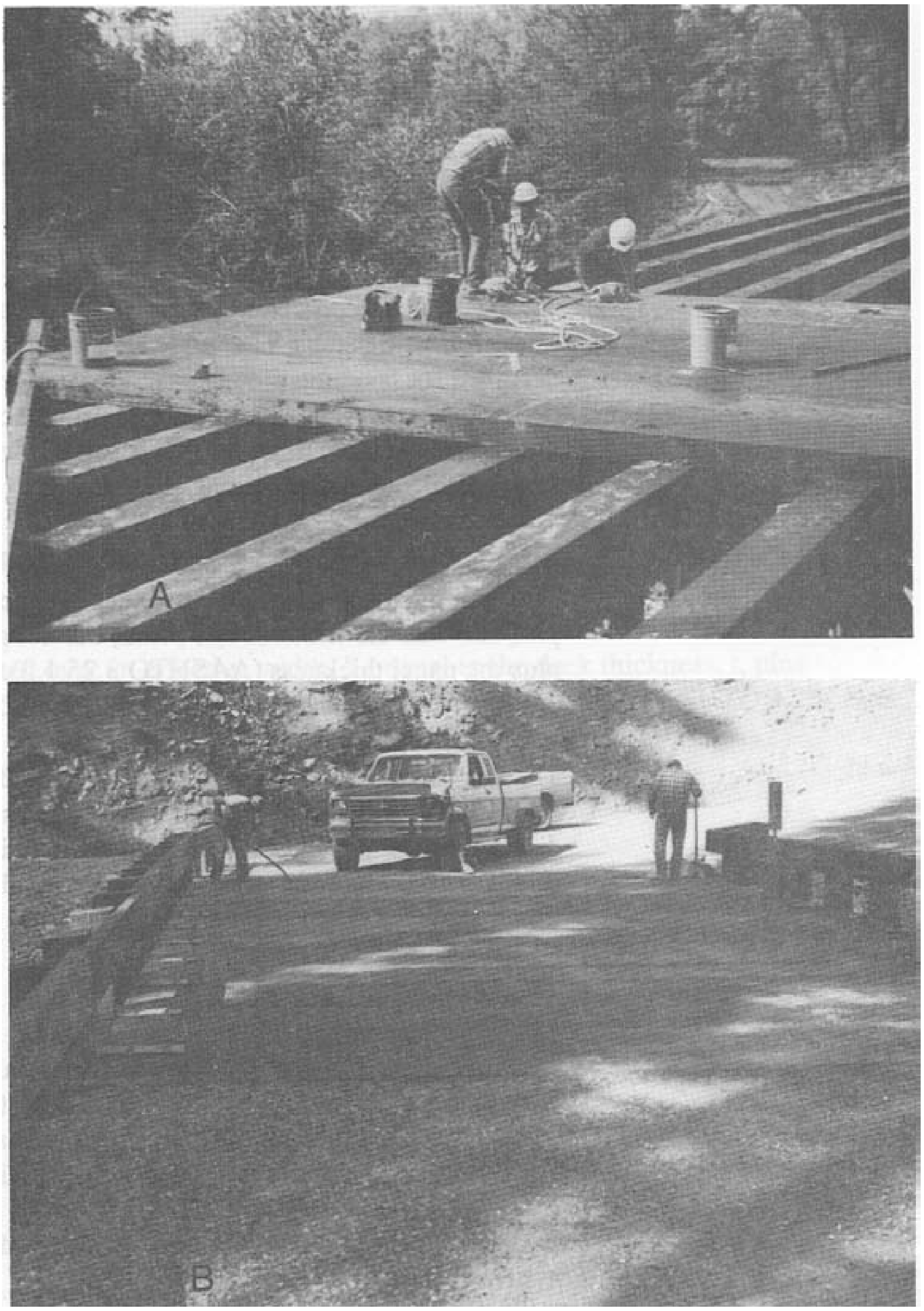


Figure 7-18. - (A) Noninterconnected glulam deck being placed (photo courtesy of LamFab Wood Structures, Inc.). (B) Completed glulam deck is prepared for paving (photo courtesy of Ron Vierra, USDA Forest Service).

Design Procedures

Noninterconnected glulam decks are designed using an interactive procedure, similar to that previously discussed for beams. The deck is assumed to act as a simple span between beams and is designed for the stresses acting in the direction of the deck span, and deflection. Stresses occurring in the direction perpendicular to the span are not critical and are not considered in design.

The basic design procedures for noninterconnected glulam decks are given in the following steps. The sequence assumes that panels are initially designed for bending, then checked for deflection and shear. Although deflection rather than bending stress usually controls in most applications, the acceptable level of deflection is established by the designer and may vary for different applications.

1. Define the deck span, design loads, and panel size.

The effective deck span, *s*, is the clear distance between supporting beams plus one-half the width of one beam, but not greater than the clear span plus the panel thickness (AASHTO 3.25.1.2). Panel width and length are based on considerations previously discussed.

The deck design load is the maximum wheel load of the design vehicle. For H 20-44 and HS 20-44 loads, AASHTO special provisions for timber decks apply, and a 12,000-pound wheel load is used instead of the standard 16,000-pound wheel load. As a result, the maximum wheel load for all standard AASHTO vehicles (H 15-44, HS 15-44, H 20-44 and HS 20-44) is 12,000 pounds.

2. Estimate deck thickness.

Deck thickness, *t*, must be estimated for initial calculations. It is generally most practical to start with a 6-3/4-inch deck (an initial estimate of deck thickness based on bending or deflection can also be made from Tables 7-8 and 7-9 presented later in this section).

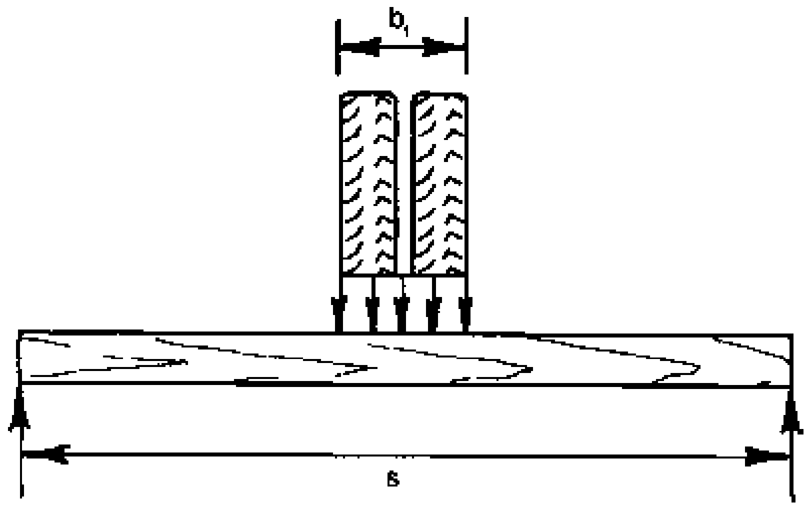
3. Determine. wheel distribution widths and effective deck section properties.

In the direction of the deck span, the wheel load is assumed to be uniformly distributed over a width, b_t (AASHTO 3.25.1.1), as computed by

$$b_t = \sqrt{0.025P} \qquad (7\text{-}10)$$

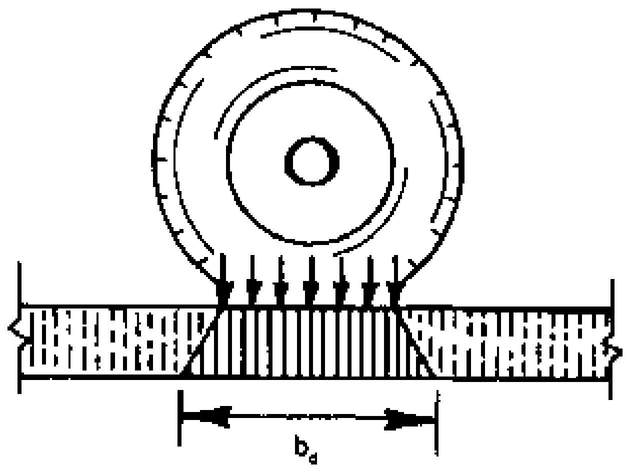
where b_t = wheel load distribution width in the direction of the deck span (in) and

P = maximum wheel load (lb).



For a 12,000-pound wheel load, b_t = 17.32 inches.

In the direction perpendicular to the deck span, the wheel load is distributed over an effective width, b_d, equal to the deck thickness, t, plus 15 inches, but not greater than the deck panel width (AASHTO 3.25.1.1):



$$b_d = t + 15 \leq \text{actual panel width} \tag{7-11}$$

where b_d = wheel load distribution width perpendicular to the deck span (in.) and

t = deck thickness (in.)

The effective deck section, defined by a deck width, b_d, and thickness, t, is designed as a beam to resist the loads and deflection produced by one wheel line of the design vehicle. Effective deck section properties are computed by

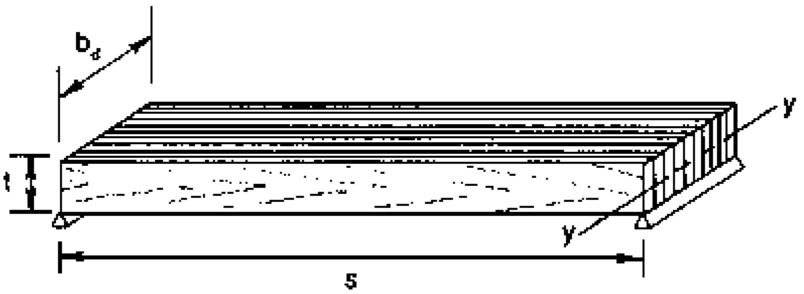
$$A = \text{effective deck area (in}^2) = b_d t \quad (7\text{-}12)$$

$$S_y = \text{effective deck section modulus (in}^3) = \frac{b_d t^2}{6} \quad (7\text{-}13)$$

$$I_y = \text{effective deck moment of inertia (in}^4) = \frac{b_d t^3}{12} \quad (7\text{-}14)$$

Effective deck section properties for common deck thicknesses are given in Table 7-6.

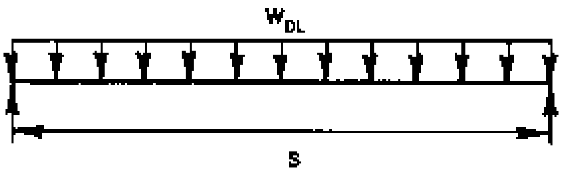
Table 7-6. - Effective deck section properties for noninterconnected glulam deck panels.



t (in.)	b_d (in.)	A (in²)	S_y (in³)	I_y (in⁴)
5	20	100.00	83.33	208.33
5-1/8	20.13	103.17	88.01	225.75
6-3/4	21.75	146.81	165.16	557.43
8-1/2	23.50	199.75	282.98	1,202.66
8-3/4	23.75	207.81	303.06	1,325.89

4. Determine dead load moment.

Uniform dead load moment for the effective deck section can be computed:



$$M_{DL} = \frac{w_{DL}\, s^2}{8} \tag{7-15}$$

where M_{DL} = deck dead load moment (in-lb),

w_{DL} = dead load of the deck and wearing surface over the wheel load distribution width, b_d(lb/in), and

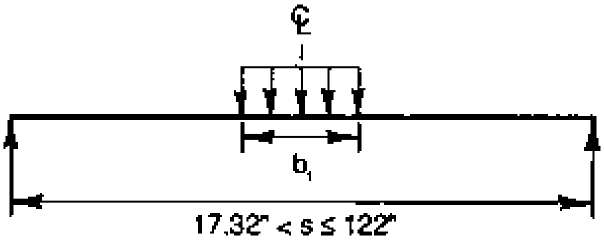
s = effective deck span (in.).

When a portion of the dead load is not uniformly distributed (as when the deck supports utility lines or other components), dead load moment from these nonuniform loads is computed by assuming the deck acts as a simple span, and the moment from the additional loading is added to M_{DL} computed by Equation 7-15.

5. Determine live load moment.

Compute the maximum vehicle live load moment by assuming that the deck acts as a simple span between beams. Wheel loads are positioned laterally on the span to produce the maximum moment using the same procedures discussed in Chapter 6 for a moving series of loads.

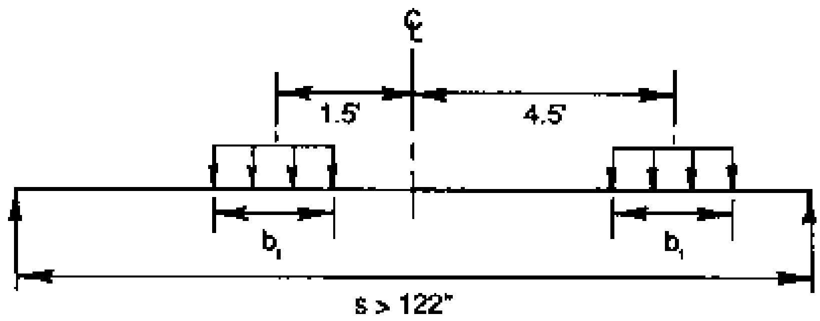
For one traffic lane, the maximum moment for a standard 12,000-pound wheel load and 6-foot-track width depends on the effective deck span, *s*. When the effective deck span is greater than 17.32 inches, but less than or equal to 122 inches $(17.32 < s \leq 122)$, maximum moment is produced when a single wheel load is positioned at the span centerline, and is computed as follows:



$$M_{LL} = 3{,}000s - 25{,}983 \tag{7-16}$$

where M_{LL} is the maximum live moment (in-lb).

When the effective deck span is greater than 122 inches ($s > 122$), the maximum moment is produced when both wheel loads are on the span. Maximum moment occurs under the wheel load closest to the span centerline when the span centerline bisects the centroid of the wheel loads and the adjacent wheel load, and is computed as follows:



$$M_{LL} = 6{,}000s + \frac{7{,}776{,}000}{s} - 457{,}983 \tag{7-17}$$

6. Compute bending stress and select a deck combination symbol.

When deck panels are continuous over two spans or less, bending stress is based on simple span moments and is computed by

$$f_b = \frac{M}{S_y} \tag{7-18}$$

where $M = M_{LL} + M_{DL}$ computed for a simple span (in-lb).

When the deck is continuous over more than two spans, the maximum bending moment is 80 percent of that computed for a simple span to account for span continuity (AASHTO 3.25.4), and is computed by

$$f_b = \frac{0.8M}{S_y} \tag{7-19}$$

Select a panel combination symbol from Table 2 of *AITC 117-Design* that provides the required bending stress. The most common combination symbols are No. 2 for western species (F_{by} = 1,800 lb/in^2) and No. 47 for Southern Pine (F_{by} = 1,750 lb/in^2). The applied bending stress, f_b, must not exceed F_b' for the selected combination symbol, computed by

$$F_b' = F_{by} C_F C_M \tag{7-20}$$

where F_{by} = tabulated bending stress from Table 2 of *AITC 117-Design* (lb/in^2) and

C_F = size factor for panels less than 12 inches thick:

t (in.)	C_F
5 or 5-1/8	1.10
6-3/4	1.07
8 or 8-3/4	1.04

F_b' computed by Equation 7-20 is given in Table 7-7 for common values of F_{by}. Allowable bending stress may be increased by a factor of 1.33 for overloads in AASHTO Load Group IB.

Table 7-7. - Values of F_b' for glulam deck panels.

Deck	Allowable bending stress, F_b' (lb/in^2)[a]				
t (in.)	F_{by}=1,450	F_{by}=1,750	F_{by}=1,800	F_{by}=2,000	F_{by}=2,100
5-1/8[b]	1,276.0	1,540.0	1,584.0	1,760.0	1,848.0
6-3/4	1,241.2	1,498.0	1,540.8	1,712.0	1,797.6
8-3/4[c]	1,206.4	1,456.0	1,497.6	1,664.0	1,747.2

[a] $F_b' = F_{by} C_M C_F$

[b] Also applies to t = 5 inches for Southern Pine.

[c] Also applies to t = 8 1/2 inches for Southern Pine.

If $f_b \leq F_b'$, the initial deck thickness and combination symbol are satisfactory in bending. When F_b is significantly lower than F_b', a thinner deck or lower grade combination symbol may be more economical; however, no changes in the panel thickness or combination symbol should be made until the live load deflection is determined.

If $f_b > F_b'$, the deck is insufficient in bending and the deck thickness or grade must be increased, or the effective deck span reduced. If deck thickness or span is changed, the design sequence must be repeated. In some cases, it may be more economical to increase deck thickness to the next higher standard size, rather than use a higher-grade combination symbol. The designer should check local availability and prices for different panel thicknesses and combination symbols before specifying panels with F_b greater than 1,800 lb/in^2 for visually graded western species or 1,750 lb/in^2 for visually graded Southern Pine.

Approximate maximum spans based on bending for noninterconnected glulam decks continuous over more than two spans are given in Table 7-8.

Table 7-8. - Approximate maximum effective span for noninterconnected transverse glulam deck panels based on bending; deck continuous over more than two spans; loading from a 12,000-pound wheel load plus the deck dead load, including a 3-inch asphalt wearing surface; b_d = 15 inches + deck thickness.

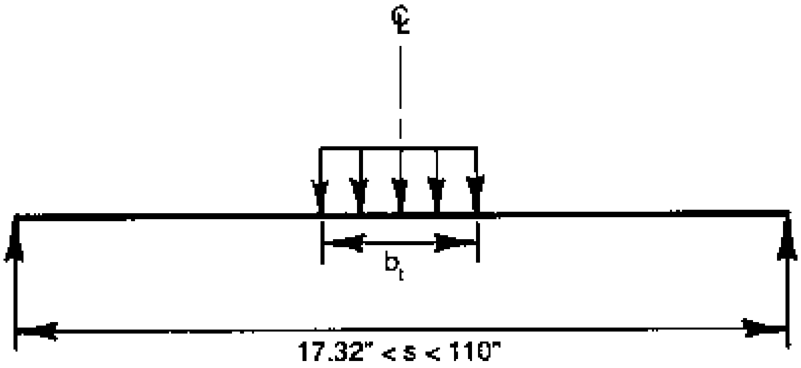
	Approximate maximum deck span (in.)		
F_{by} (lb/in^2)	t = 5 in. or t = 5-1/8 in.	t = 6-3/4 in.	t = 8-1/2 in. or t = 8-3/4 in.
1,450	52	91	>120
1,750	61	107	>120
1,800	65	109	>120
2,000	68	120	>120
2,100	74	>120	>120

$F_b' = F_{by}C_MC_F$ as given in Table 7-7.

7. Check live load deflection.

Live load deck deflection is computed by standard methods of engineering analysis, assuming the deck to be a simple span between beams. For standard AASHTO trucks, with 12,000-pound wheel loads and a 6-foot track width, equations for maximum deflection on a simple span are as follows:

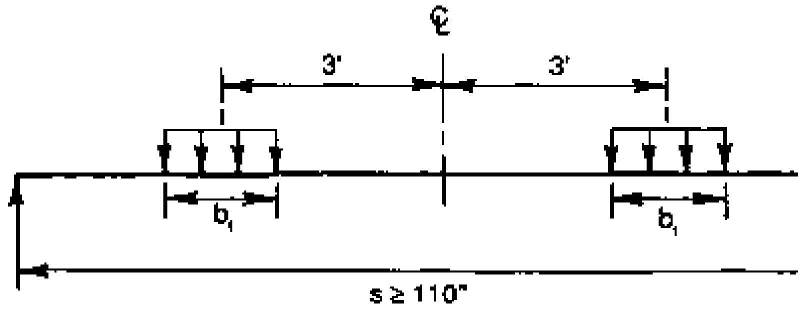
For effective spans greater than 17.32 inches, but less than or equal to 110 inches (17.32 < s < 110), maximum live load deflection occurs with one wheel load positioned at the span centerline and is computed as follows:



$$\Delta_{LL} = \frac{1.80}{E'I_y}\left(138.8s^3 - 20,780s + 90,000\right) \qquad (7\text{-}21)$$

where $E' = EC_M$ (lb/in^2).

When the effective deck span is greater than or equal to 110 inches **($s \geq 110$),** maximum live load deflection is obtained when both wheel loads are centered on the span and is computed as follows:



$$\Delta_{LL} = \frac{1}{E'I_y}\left(500s^3 + 90.5s^2 - 3{,}967{,}074s + 98{,}663{,}396\right) \tag{7-22}$$

When the deck is continuous over more than two spans, the maximum deflection is 80 percent of that computed for a simple span to account for deck continuity. In this case, values obtained from Equations 7-21 or 7-22 may be multiplied by 0.80. Deflection coefficients for standard 12,000-pound wheel load(s) on decks continuous over more than two spans are given in Figure 7-19.

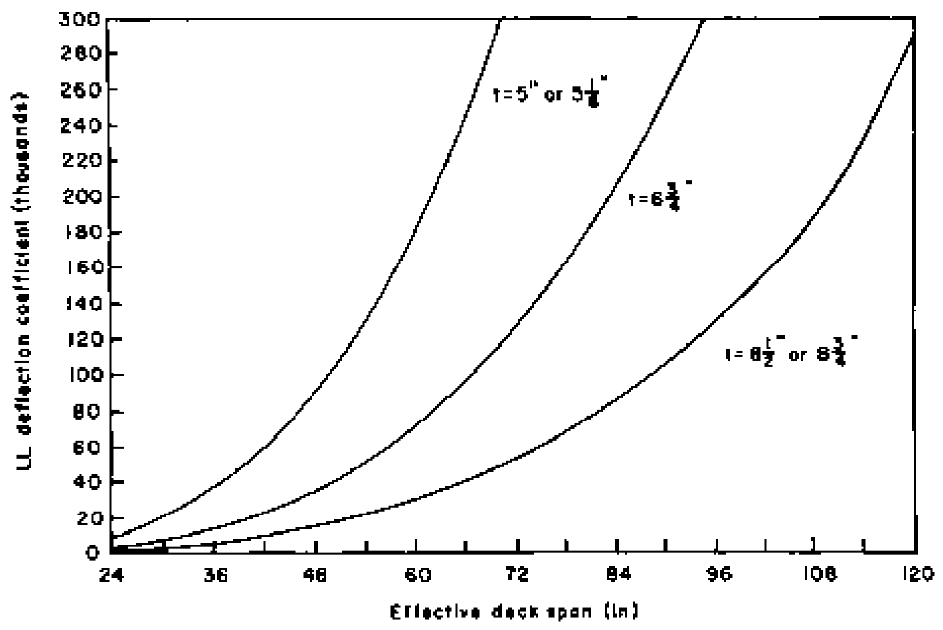


Figure 7-19. - Vehicle live load deflection coefficients for 12,000-pound wheel load(s) on a transverse, noninterconnected glulam deck that is continuous over more than two spans. Divide the deflection coefficient by E' to obtain the deck deflection in inches.

Requirements for live load deflection in glulam decks are not included in AASHTO specifications, and the acceptable deflection limit is left to designer judgment. Deck deflection is important because it directly influences the performance and serviceability of the deck, wearing surface, and mechanical connections. When deflections are large, vertical movement of the panel causes vibrations in the structure and rotation of the deck panel about the beam. This can cause bolts or other connections to loosen and asphalt wearing surfaces to crack. Deck movement can also be alarming to users, especially pedestrians.

The maximum recommended live load deflection for noninterconnected glulam panels is 0.10 inch. This limit was derived from research and field observations related to panel attachment and asphalt wearing surface performance.[62] Deflection will control over bending in most design applications, but panel spans remain within the acceptable range of recommended beam spacings previously discussed. Based on this criterion, maximum effective deck spans for live load deflection are shown in Table 7-9. A further reduction in deflection for deck panels supporting pedestrian walkways or an asphalt wearing surface is desirable.

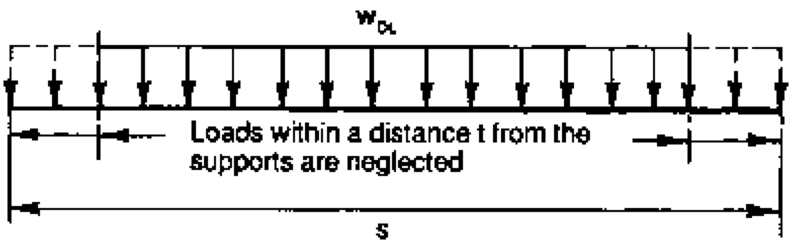
Table 7-9. - Approximate maximum effective span for noninterconnected transverse glulam deck panels based on a maximum vehicle live load deflection of 0.10 inch; deck continuous over more than two supports; loading from a 12,000-pound wheel load; b_d = 15 inches + deck thickness.

		Approximate maximum deck span (in.)		
E (lb/in^2)	E' (lb/in^2)	t = 5 in. or t = 5-1/8 in.	t = 6-3/4 in.	t = 8-1/2 in. or t = 8-3/4 in.
1,300,000	1,082,900	50	68	91
1,400,000	1,166,200	51	70	94
1,500,000	1,249,500	53	72	95
1,700,000	1,416,100	56	75	99
1,800,000	1,499,400	57	76	101

$E' = EC_M = 0.833E$.

8. Check horizontal shear.

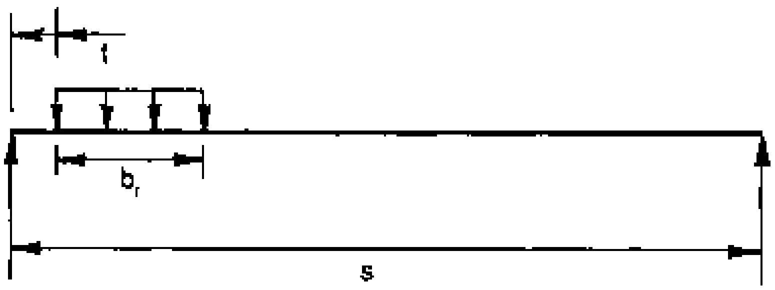
Horizontal shear for dead load is based on the maximum vertical shear occurring at a distance from the support equal to the deck thickness, *t*. Loads occurring within the distance *t* from the supports are neglected. Horizontal shear for dead load is computed as follows:



$$V_{DL} = w_{DL}\left(\frac{s}{2} - t\right) \tag{7-23}$$

where V_{DL} = dead load vertical shear (lb).

Live load vertical shear is computed by placing the edge of the wheel load distribution width, b_t, a distance t from the support.



Applied stress in horizontal shear is based on a different effective panel width than that used for bending and deflection. Current AASHTO specifications (interims through 1987) allow the stress to be distributed over the full panel width (AASHTO 13.3.1). AITC has recently recommended a more conservative distribution width of 15 inches plus twice the deck thickness, but not greater than the panel width. In either case, shear stress is normally not a controlling factor in glulam panel design. The distribution width used in this chapter follows the AITC recommendations. Either convention may be used based on designer judgment.

Horizontal shear stress is computed using

$$f_v = \frac{1.5V}{A_v} \tag{7-24}$$

where $V = V_{DL} + V_{LL}$ (lb), and

$A_v = t(15 + 2t) \le t(\text{panel width})(\text{in}^2)$

Allowable shear stress is computed using

$$F_v' = F_{vy} C_M \tag{7-25}$$

where F_v' = allowable horizontal shear stress (lb/in^2),

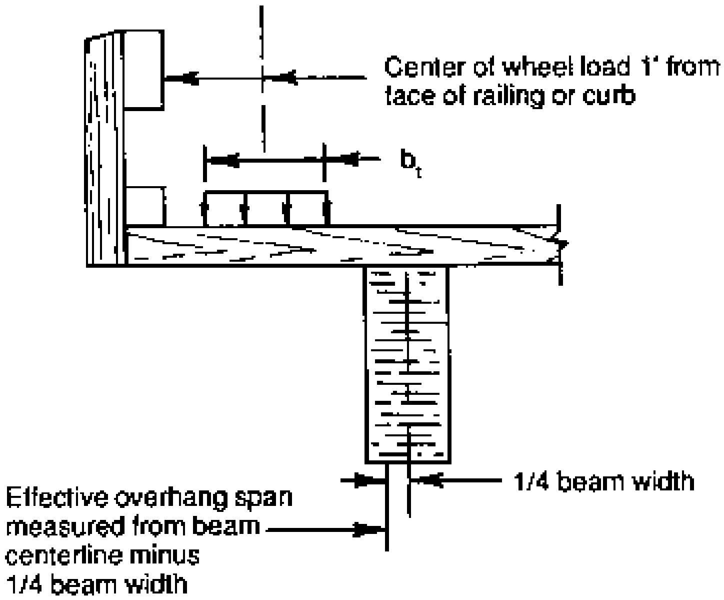
F_{vy} = tabulated shear stress from Table 2, *AITC 117--Design* (lb/in^2), and

C_M = wet-use factor for shear = 0.875.

Values of F_v within each species group are the same for the various combination symbols commonly used for glulam decks. For western species, F_v = 145 lb/in^2, while for Southern Pine, F_v = 175 lb/in^2. When $f_v > F_v'$, the only options are to increase the deck thickness or reduce the effective deck span.

9. Check overhang.

The deck overhang at exterior supports is checked using an effective span measured to the centerline of the outside beam, minus one-fourth the beam width. For vehicle live load stresses and deflection, the wheel load is positioned with the load centroid 1 foot from the face of the railing or curb.



Deck stress in bending and horizontal shear must be within allowable values previously determined.

Example 7-5 - Noninterconnected glulam deck with highway loading

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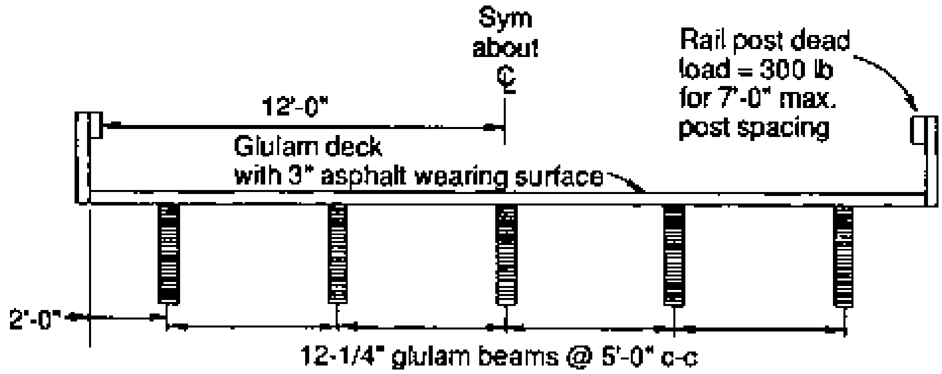
Design a noninterconnected glulam deck for the beam superstructure of Example 7-3. The superstructure has a 24-foot roadway that carries two lanes of AASHTO HS 20-44 loading. Support is provided by five 12-1/4-inch-wide glulam beams that are spaced 5 feet on center and are 95-1/2 feet long. The following assumptions apply:

1. glulam deck panels are manufactured from visually graded Southern Pine;
2. rail system dead load is 300 pounds at each post with a maximum post spacing of 7 feet; and
3. deck live load deflection is limited to approximately 0.10 inch.



Solution

Determine the Deck Span, Design Loads, and Panel Size

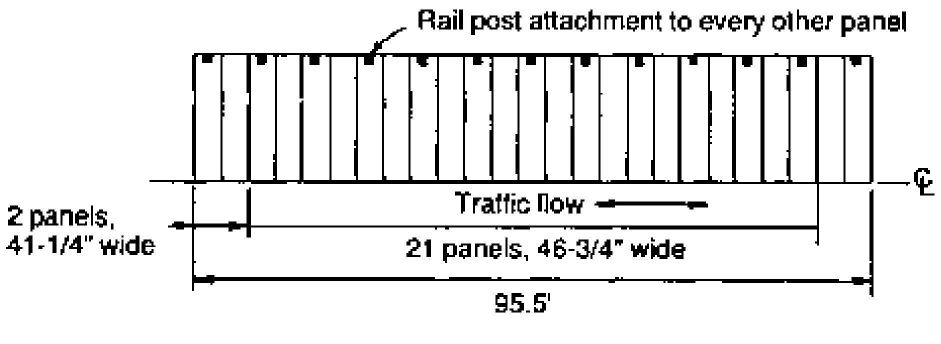
The deck span is the clear distance between supporting beams plus one-half the width of one beam, but not greater than the clear span plus the panel thickness:

Clear distance between beams = 60 in. - 12.25 in. = 47.75 in.

$$s = 47.75 \text{ in.} + \frac{12.25}{2} = 53.88 \text{ in.}$$

If a 5-inch deck is used, *s* will be limited by the clear span plus deck thickness to 47.75 inches + 5 inches = 52.75 inches. For other deck thicknesses, *s* = 53.88 inches will control.

For HS 20-44 loading, AASHTO special provisions apply and the deck will be designed for a 12,000-pound wheel load. Panel width for an out-to-out bridge length of 95-1/2-feet will be based on an alternating repetition of panels to allow standardized panel configurations. In this case, 46-3/4-inch-wide panels will be used with two 41-1/4-inch-wide panels at each end (one of the end panels will be trimmed 3/4 inch before pressure treatment). Rail posts will be placed at the center of end panels and at the center of every second panel:



Estimate Deck Thickness

From approximate maximum deck spans given in Tables 7-8 and 7-9, an initial deck thickness of 5 inches is selected. The effective span used for design will therefore be controlled by the clear span plus deck thickness to 52.75 inches.

Determine Wheel Distribution Widths and Effective Deck Section Properties

In the direction of the deck span,

$$b_t = \sqrt{0.025P} = \sqrt{0.025(12{,}000)} = 17.32 \text{ in.}$$

Normal to the deck span,

$$b_d = t + 15 = 5 + 15 = 20 \text{ in.}$$

Effective deck section properties from Table 7-6 are

$$A = 100 \text{ in}^2$$

$$S_y = 83.33 \text{ in}^3$$

$$I_y = 208.33 \text{ in}^4$$

Determine Deck Dead Load

For a 5-inch deck and 3-inch asphalt wearing surface, dead load unit weight and moment over the effective distribution width of 20 inches are computed as follows:

$$w_{DL} = (20 \text{ in.})\frac{\left[(5 \text{ in.})(50 \text{ lb/ft}^3) + (3 \text{ in.})(150 \text{ lb/ft}^3)\right]}{1728 \text{ in}^3/\text{ft}^3} = 8.1 \text{ lb/in.}$$

$$M_{DL} = \frac{w_{DL}\, s^2}{8} = \frac{8.1(52.75)^2}{8} = 2{,}817 \text{ in-lb}$$

Determine Live Load Moment

For an effective deck span less than 122 inches, maximum live load moment is computed for a (6-foot track width and 12,000-pound wheel load by Equation 7-16:

$$M_{LL} = 3{,}000s - 25{,}983 = 3{,}000(52.75) - 25{,}983 = 132{,}267 \text{ in-lb}$$

Compute Bending Stress and Select a Deck Combination Symbol

The deck is continuous over more than two spans, so bending stress is based on 80 percent of the simple span moment:

$$M = M_{DL} + M_{LL} = 2{,}817 + 132{,}267 = 135{,}084 \text{ in-lb}$$

$$f_b = \frac{0.80\,M}{S_y} = \frac{0.80(135{,}084)}{83.33} = 1{,}297 \text{ lb/in}^2$$

From Table 7-7, F_{by} = 1,750 lb/in^2 is the closest value for a Southern Pine combination that will meet bending requirements. An initial combination symbol No. 47 is selected, and the following values are obtained from *AITC 117-Design:*

F_{by} = 1,750 lb/in^2 C_M = 0.80

F_{vy} = 175 lb/in^2 C_M = 0.875

E_y = 1,400,000 lb/in^2 C_M = 0.833

By Equation 7-20 (or Table 7-7),

$$F_b' = F_{by} C_F C_M = 1{,}750(1.1)(0.80) = 1{,}540 \text{ lb/in}^2$$

f_b = 1,297 lb/in^2 < F_b' = 1,540 lb/in^2, so a 5-inch combination symbol No. 47 panel is satisfactory in bending.

Check Live Load Deflection

Maximum deflection is computed for a 12,000-pound wheel load and 6-foot track width by Equation 7-21:

$$E' = E_y C_M = 1{,}400{,}000(0.833) = 1{,}166{,}200 \text{ lb/in}^2$$

$$\Delta_{LL} = \frac{1.80}{E' I_y}\left(138.8 s^3 - 20{,}780 s + 90{,}000\right)$$

$$\Delta_{LL} = \frac{1.80\left[(138.8)(52.75)^3 - (20{,}780)(52.75) + 90{,}000\right]}{1{,}166{,}200(208.33)} = 0.14 \text{ in.}$$

The deck is continuous over more than two spans, so 80 percent of the simple span deflection is used to account for span continuity:

$$\Delta_{LL} = 0.80(0.14) = 0.11 \text{ in.}$$

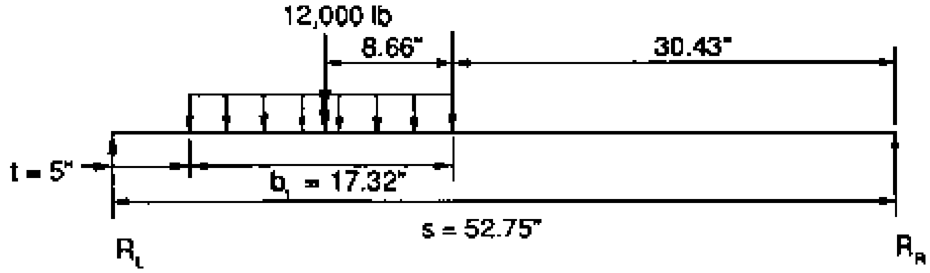
The computed deflection of 0.11 inch is slightly greater than 0.10 inch, but the difference of 0.01 inch is considered insignificant and deflection is acceptable.

Check Horizontal Shear

Dead load vertical shear is computed at a distance *t* from the support. By. Equation 7-23 for w_{DL} = 8.1 lb/in,

$$V_{DL} = w_{DL}\left(\frac{s}{2} - t\right) = 8.1\left(\frac{52.75}{2} - 5\right) = 173.1 \text{ lb}$$

Live load vertical shear is computed by placing the edge of the wheel load distribution width (b_t) a distance *t* from the support. The resultant of the 12,000-pound wheel load acts through the center of the distribution width and V_{LL} is computed by statics:



$$V_{DL} = R_L \frac{12{,}000 \text{ lb}(8.66 \text{ in.} + 30.43 \text{ in.})}{52.75 \text{ in.}} = 8{,}893 \text{ lb}$$

By Equation 7-24,

$$V = V_{DL} + V_{LL} = 173.1 + 8{,}893 = 9{,}066 \text{ lb}$$

$$A_v = t(15 + 2t) = 5(15 + 10) = 125 \text{ in}^2$$

$$f_v = \frac{1.5V}{A_v} = \frac{1.5(9{,}066)}{125} = 109 \text{ lb/in}^2$$

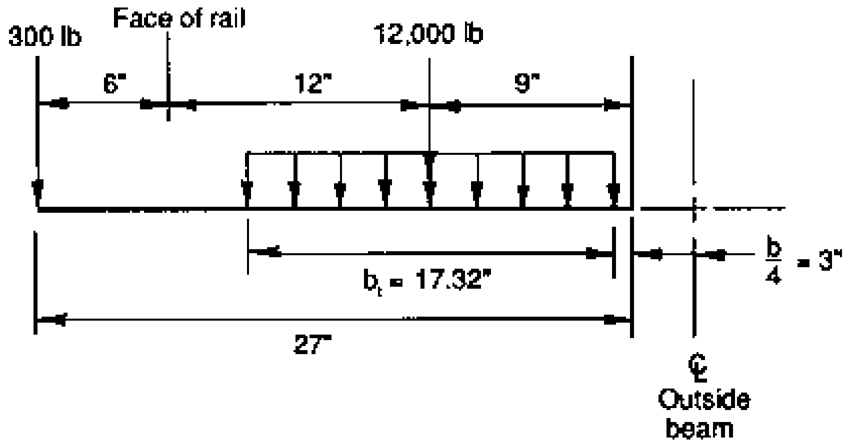
By Equation 7-25,

$$F_v' = F_{vy} C_M = 175(0.875) = 153 \text{ lb/in}^2$$

$f_v = 109 \text{ lb/in}^2 < F_v' = 153$, so the panel is satisfactory in horizontal shear.

Check Overhang

Bending and shear stresses are checked in the deck overhang by positioning the wheel load centroid 1 foot from the rail face. Moments are computed using an effective span measured from the load to the beam centerline, minus one-fourth the beam width:



$$M_{LL} = (9 \text{ in.})(12{,}000 \text{ lb}) = 108{,}000 \text{ in-lb}$$

$$\text{Rail } M_{DL} = (27 \text{ in.})(300 \text{ lb}) = 8{,}100 \text{ in-lb}$$

$$\text{Deck } M_{DL} = (8.1 \text{ lb/in})(27 \text{ in.})\left(\frac{27 \text{ in.}}{2}\right) = 2{,}952 \text{ in-lb}$$

$$M = M_{LL} + M_{DL}$$

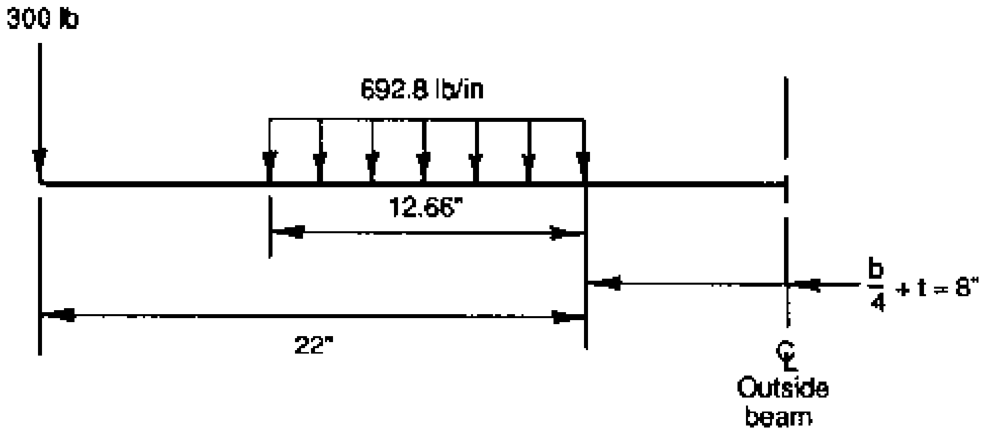
$$= 108{,}000 + 8{,}100 + 2{,}952 = 119{,}052 \text{ in-lb}$$

$$f_b = \frac{M}{S_y} = \frac{119{,}052}{83.3} = 1{,}429 \text{ lb/in}^2 < 1{,}540 \text{ lb/in}^2$$

The overhang is satisfactory in bending.

Horizontal shear in the overhang is based on the maximum vertical shear occurring a distance from the beam centerline equal to one-fourth the beam width plus the deck thickness. Loads acting within this distance from the beam centerline are neglected. The distributed wheel load is equal to the wheel load divided by the distribution width:

$$\text{Distributed wheel load} = \frac{12{,}000 \text{ lb}}{17.32 \text{ in.}} = 692.8 \text{ lb/in}$$



$$V_{LL} = (12.66 \text{ in.})(692.8 \text{ lb/in}) = 8{,}771 \text{ lb}$$

$$\text{Rail } V_{DL} = 300 \text{ lb}$$

$$\text{Deck } V_{DL} = (8.1 \text{ lb/in})(22 \text{ in.}) = 178 \text{ lb}$$

$$V = V_{LL} + V_{DL} = 8{,}771 + 300 + 178 = 9{,}249 \text{ lb}$$

$$f_v = \frac{1.5V}{A_v} = \frac{1.5(9{,}249)}{125} = 111 \text{ lb/in}^2 < 153 \text{ lb/in}^2$$

The overhang is satisfactory.

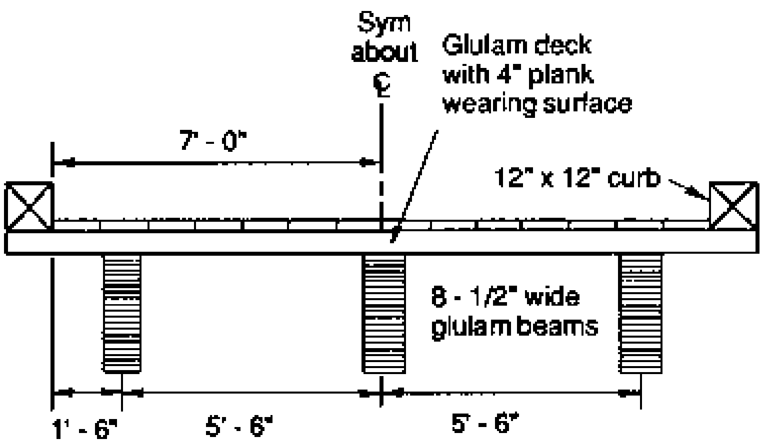
Summary
The deck will consist of 5-inch-thick noninterconnected glulam panels, 25 feet long. A total of 25 panels are required: 21 panels that are 46-3/4 inches wide and 4 panels that are 41-1/4 inches wide (one end panel will be trimmed 3/4 inch before treatment). Deck panels will be manufactured from visually graded Southern Pine, combination symbol No. 47. Stresses and deflection are as follows:

	Center spans	Overhang
f_b	1,297 lb/in²	1,429 lb/in²
F_b'	1,540 lb/in²	1,540 lb/in²
Δ_{LL}	0.11 in.	—
f_v	109 lb/in²	111 lb/in²
F_v'	153 lb/in²	153 lb/in²

Example 7-6 - Noninterconnected glulam deck; single-lane with overload

A glulam beam superstructure has a 14-foot-wide roadway that carries one lane of AASHTO H 20-44 loading with a U80 overload. Support is provided by three 8-1/2-inch-wide glulam beams that are spaced 5 feet 6 inches on center and are 52 feet long. Design a noninterconnected glulam deck for this bridge, assuming

1. glulam deck panels are manufactured from visually graded western species;
2. a 12-inch by 12-inch lumber curb is continuous along each edge of the deck,
3. the deck is covered with a rough-sawn, 4-inch-thick plank wearing surface; and
4. deck live load deflection is limited to approximately 0.10 inch for H 20-44 loads. No deflection criteria apply to the U80 overload.



Solution

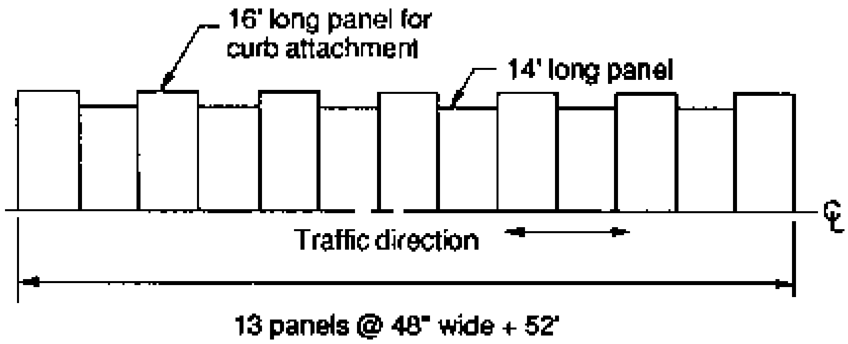
Determine the Deck Span, Design Loads, and Panel Size

Clear distance between beams = 66 in - 8.50 in = 57.50 in

$$s = 57.50 \text{ in} + \frac{8.50}{2} = 61.75 \text{ in.}$$

For H 20-44 loading, AASHTO special provisions apply and a 12,000-pound wheel load is used for design. From Figure 6-5, the U80 wheel load weight is 18,500 pounds.

Panel width for an out-to-out bridge length of 52 feet will be 48 inches. End panels and alternating interior panels will be 16 feet long for curb attachment. Other panels will be 14 feet long:



Estimate Deck Thickness

An initial deck thickness of 6-3/4 inches will be used. Although it is anticipated that U80 loading will control design, stresses will be computed for both vehicles for future reference.

Determine Wheel Distribution Widths and Effective Deck Section Properties

In the direction of the deck span,

$$\text{H 20-44 } b_t = \sqrt{0.025P} = \sqrt{0.025(12{,}000)} = 17.32 \text{ in.}$$

$$\text{U80 } b_t = \sqrt{0.025(18{,}500)} = 21.51 \text{ in.}$$

Normal to the deck span,

$$b_d = t + 15 = 6.75 + 15 = 21.75 \text{ in.}$$

Effective deck section properties from Table 7-6 are

$$A = 146.81 \text{ in}^2$$

$S_y = 165.16 \text{ in}^3$

$I_y = 557.43 \text{ in}^4$

Determine Deck Dead Load

For a 6-3/4-inch deck and a 4-inch plank wearing surface, dead load unit weight and moment over the effective distribution width of 21.75 inches are computed as follows:

$$w_{DL} = 21.75 \text{ in.} \frac{\left[(6.75 + 4 \text{ in.})(50 \text{ lb/in}^3)\right]}{1{,}728 \text{ in}^3/\text{ft}^3} = 6.8 \text{ lb/in.}$$

$$M_{DL} = \frac{w_{DL} s^2}{8} = \frac{6.8\,(61.75)^2}{8} = 3{,}241 \text{ in-lb}$$

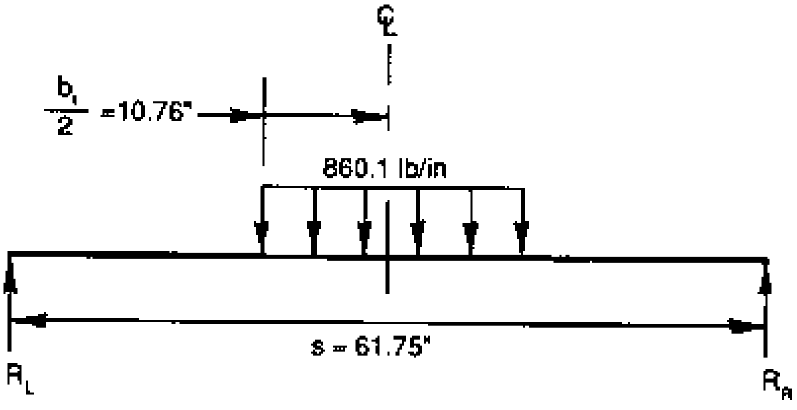
Determine Live Load Moment

By Equation 7-16,

$$\text{H 20-44 } M_{LL} = 3{,}000s - 25{,}983 = 3{,}000(61.75) - 25{,}983 = 159{,}267 \text{ in-lb}$$

U80 moment is computed at the span centerline by centering the distributed wheel load.

$$\text{U80 distributed wheel load} = \frac{18{,}500 \text{ lb}}{21.51 \text{ in.}} = 860.1 \text{ lb/in.}$$



$$R_L = \frac{18{,}500 \text{ lb}}{2} = 9{,}250 \text{ lb}$$

$$\text{U80 } M_{LL} = \left[(9{,}250 \text{ lb})\frac{(61.75 \text{ in.})}{2}\right] - \left[(9{,}250 \text{ lb})\frac{(10.76 \text{ in.})}{2}\right]$$
$$= 235{,}829 \text{ in-lb}$$

Dividing U80 moment by the allowable overload increase of 1.33 and comparing the value to H 20-44 moment indicates the controlling vehicle:

$$\frac{235{,}829 \text{ in-lb}}{1.33} = 177{,}315 > 159{,}267$$

so the U80 will control bending.

Compute Bending Stress and Select a Deck Combination Symbol

The deck is continuous over two spans, so the reduction in bending stress for continuity is not applicable.

$$\text{U80 } M = M_{DL} + M_{LL} = 3{,}241 + 235{,}829 = 239{,}070 \text{ in-lb}$$

$$f_b = \frac{M}{S_y} = \frac{239{,}070}{165.16} = 1{,}448 \text{ lb/in}^2$$

From Table 7-7 for an approximate $F_b' = 1{,}448/1.33 = 1{,}089$ lb/in^2, an F_{by} of 1,450 lb/in^2 is the closest value for a western species combination symbol. An initial combination symbol No. 1 is selected, and the following values are obtained from *AITC 117-Design:*

$F_{by} = 1{,}450$ lb/in^2	$C_M = 0.80$
$F_{vy} = 145$ lb/in^2	$C_M = 0.875$
$E_y = 1{,}500{,}000$ lb/in^2	$C_M = 0.833$

By Equation 7-20,

$$\text{U80 } F_b' = F_{by}(1.33)C_F C_M = 1{,}450(1.33)(1.07)(0.80) = 1{,}651 \text{ lb/in}^2$$

$f_b = 1{,}448$ lb/in$^2 < F_b' = 1{,}651$ lb/in^2, so a 6-3/4-inch combination symbol No. 1 panel is satisfactory in bending.

Check H 20-44 loading:

$$M = M_{DL} + M_{LL} = 3{,}241 + 159{,}267 = 162{,}508 \text{ in-lb}$$

$$f_b = \frac{M}{S_y} = \frac{162{,}508}{165.16} = 984 \text{ lb/in}^2$$

$$F_b' = F_{by} C_F C_M = 1{,}450(1.07)(0.80) = 1{,}241 \text{ lb/in}^2$$

$f_b = 984$ lb/in$^2 < F_b' = 1{,}241$ lb/in^2, so the deck is satisfactory for H 20-44 loading.

Check Live Load Deflection

Maximum H 20-44 deflection for a panel continuous over two spans is computed by Equation 7-21:

$$E' = E_y C_M = 1{,}500{,}000(0.833) = 1{,}249{,}500 \text{ lb/in}^2$$

$$\Delta_{LL} = \frac{1.80}{E' I_y}\left(138.8s^3 - 20{,}780s + 90{,}000\right)$$

$$\Delta_{LL} = \frac{1.80\left[(138.8)(61.75)^3 - 20{,}780\,(61.75) + 90{,}000\right]}{(1{,}249{,}500)\,(557.43)} = 0.08 \text{ in.}$$

0.08 inch is less than 0.10 inch, so deck deflection is acceptable.

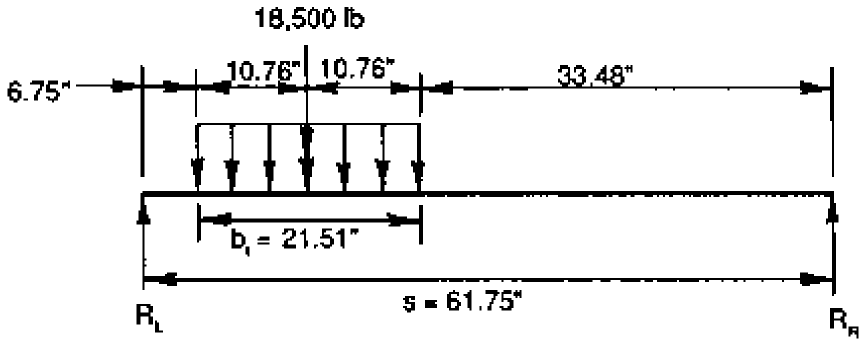
Check Horizontal Shear

Dead load vertical shear is computed by Equation 7-23 for w_{DL} = 6.8 lb/in.:

$$V_{DL} = w_{DL}\left(\frac{s}{2} - t\right) = 6.8\left(\frac{61.8}{2} - 6.75\right) = 164.1 \text{ lb}$$

Live load vertical shear is computed by placing the edge of the wheel load distribution width *(b_l)* a distance *t* from the support.

For U80 loading,



$$V_{LL} = R_L = \frac{(18{,}500 \text{ lb})(10.76 \text{ in.} + 33.48 \text{ in.})}{61.75 \text{ in.}} = 13{,}254 \text{ lb}$$

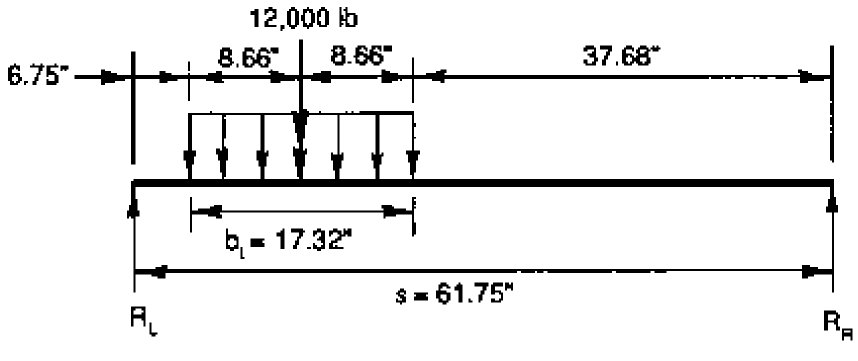
$$V \;\; = V_{DL} + V_{LL} = 164.1 + 13{,}254 = \;13{,}418 \text{ lb}$$

$$A_v \;\; = t\,(15 + 2t) = 6.75\,[15 + (2)(6.75)] = 192.38 \text{ in}^2$$

$$f_v \;\; = \frac{1.5V}{A_v} = \frac{1.5\,(13{,}418)}{192.38} = 105 \text{ lb/in}^2$$

$$F_v' \;\; = F_{vy}(1.33)C_M = 145(1.33)(0.875) = 169 \text{ lb/in}^2 > 105 \text{ lb/in}^2$$

For H 20-44 loading,



$$V_{LL} = R_L = \frac{12{,}000 \text{ lb}(8.66 \text{ in.} + 37.68 \text{ in.})}{61.75 \text{ in.}} = 9{,}005 \text{ lb}$$

$$V = V_{DL} + V_{LL} = 164.1 + 9{,}005 = 9{,}169 \text{ lb}$$

$$f_v = \frac{1.5V}{A_v} = \frac{1.5(9{,}169)}{192.38} = 71 \text{ lb/in}^2$$

$$F_v' = F_{vy} C_M = 145(0.875) = 127 \text{ lb/in}^2 > 71 \text{ lb/in}^2$$

The panel is satisfactory in horizontal shear for both vehicles.

Check Overhang

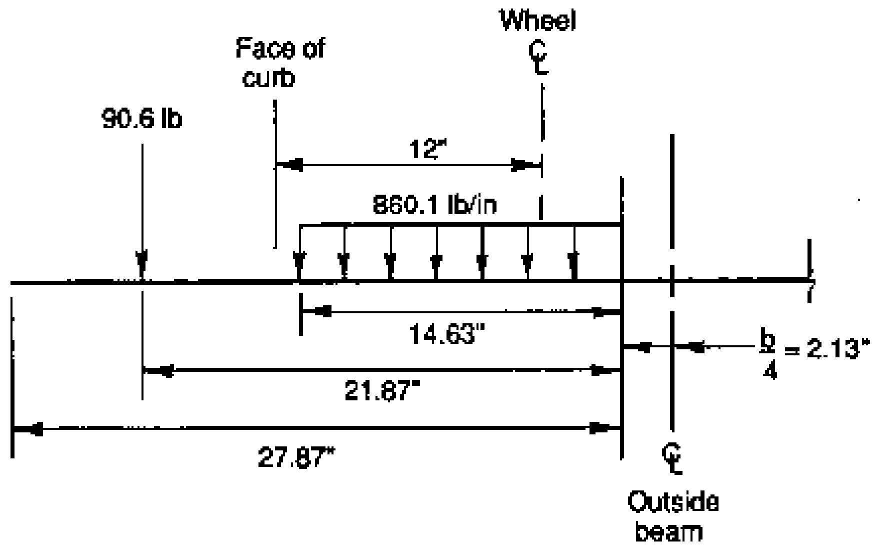
Bending and shear stresses are checked in the deck overhang by positioning the wheel load centerline 1 foot from the curb face, which is 6 inches from the outside beam centerline. Moments are computed using an effective span measured from the load to the beam centerline, minus one-fourth the beam width. Horizontal shear is based on the maximum vertical shear occurring a distance from the beam centerline equal to one-fourth the beam width plus the deck thickness. Loads acting within this distance from the beam centerline are neglected for shear.

Dead load of the 12- by 12-inch curb, and the distributed dead load of the deck and wearing surface, is computed for the U80 distribution width b_d = 21.75 in:

$$\text{Curb DL} = \frac{21.75 \text{ in.}}{12 \text{ in./ft}}(50 \text{ lb/ft}) = 90.6 \text{ lb}$$

$$\text{Deck } w_{DL} = (21.75 \text{ in.})\left[\frac{(6.75 \text{ in.})(50 \text{ lb/ft}^3)}{1{,}728 \text{ in}^3/\text{ft}^3}\right] = 4.3 \text{ lb/in.}$$

$$\text{Wearing surface } w_{DL} = (21.75 \text{ in.})\left[\frac{(4 \text{ in.})(50 \text{ lb/ft}^3)}{1{,}728 \text{ in}^3/\text{ft}^3}\right] = 2.5 \text{ lb/in.}$$



Summing moments at a point *b/4* = 2.13 inches from the outside beam centerline,

$$M_{LL} = (14.63\text{ in.})(860.1\text{ lb/in.})\left(\frac{14.63\text{ in.}}{2}\right) = 92{,}047\text{ in-lb}$$

$$\text{Curb } M_{DL} = (21.87\text{ in.})(90.6\text{ lb}) = 1{,}981\text{ in-lb}$$

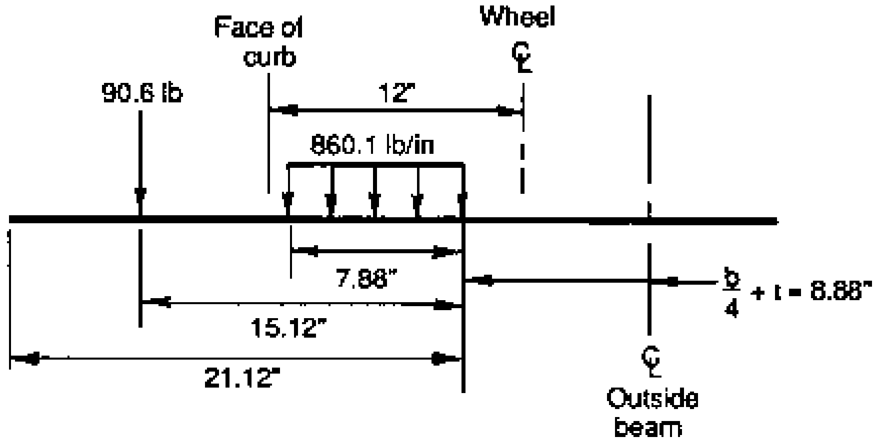
$$\text{Deck } M_{DL} = (4.3\text{ lb/in.})(27.87\text{ in.})\left(\frac{27.87\text{ in.}}{2}\right) = 1{,}670\text{ in-lb}$$

$$\text{Wearing surface } M_{DL} = (2.5\text{ lb/in.})(15.87\text{ in.})\left(\frac{15.87\text{ in.}}{2}\right) = 314.8\text{ in-lb}$$

$$M = M_{LL} + M_{DL} = 92{,}047 + 1{,}981 + 1{,}670 + 314.8$$
$$= 96{,}013\text{ in-lb}$$

$$f_b = \frac{M}{S_y} = \frac{96{,}013}{165.16} = 581\text{ lb/in}^2 < \text{U80}F_b' = 1{,}651\text{ lb/in}^2$$

Horizontal shear is computed at a distance of *b/4* + *t* = 8.88 inches from the outside beam centerline:



$$V_{LL} = (7.88\text{ in.})(860.1\text{ lb/in.}) = 6{,}778\text{ lb}$$

$$\text{Curb } V_{DL} = 90.6\text{ lb}$$

$$\text{Deck } V_{DL} = (4.3\text{ lb/in.})(21.12\text{ in.}) = 90.8\text{ lb}$$

$$\text{Wearing surface } V_{DL} = (2.5\text{ lb/in.})(9.12\text{ in.}) = 22.8\text{ lb}$$

$$V = V_{LL} + V_{DL} = 6{,}778 + 90.6 + 90.8 + 22.8 = 6{,}982\text{ lb}$$

$$f_v = \frac{1.5V}{A_v} = \frac{1.5(6{,}982)}{192.38} = 54\text{ lb/in}^2 < \text{U80}F_v' = 169\text{ lb/in.}$$

The overhang is satisfactory for U80 loading with low stress levels. Further checks for the lighter H 20-44 loading are not required.

Summary

The deck will consist of 6-3/4-inch noninterconnected glulam panels that are 48 inches wide. A total of 13 panels are required: 7 panels 16 feet long and 6 panels 14 feet long. Panels will be manufactured from visually graded western species combination symbol No. 1. Stresses and deflections are as follows:

	H 20-44 loading	U80 loading
Center spans		
f_b	984 lb/in^2	1,448 lb/in^2
F_b'	1,241 lb/in^2	1,651 lb/in^2
Δ_{LL}	0.08 in.	—
f_v	71 lb/in^2	105 lb/in^2
F_v'	127 lb/in^2	169 lb/in^2
Overhang		
f_b	< U80	581 lb/in^2
F_b'	< U80	1,651 lb/in^2
f_v	< U80	54 lb/in^2
F_v'	< U80	169 lb/in^2

DOWELED GLULAM DECKS

Doweled glulam decks consist of a series of glulam deck panels interconnected at the panel joints with steel dowels (Figure 7-20). The dowels transfer loads between panels and reduce relative displacements and rotations between adjacent panels. As a result, doweled decks generally have lower live load deflections and may result in longer deck spans or thinner panels than noninterconnected decks. These advantages can be significant in some cases but may not be sufficient to offset the increased costs required for dowel installation.

The suitability of a doweled deck for a specific application depends on the design requirements of the structure and the economics of fabrication and construction. Doweled panels are more expensive than noninterconnected decks because they require precise fabrication for proper installation and performance. As a general rule, they are most practical when an asphalt wearing surface is used and the deflection at the panel joints must be limited to prevent cracking. However, it may be more cost effective to use a noninterconnected deck and limit deflections by using a thicker deck or decreased deck span. When paving is not planned, noninterconnected panels will generally provide the most economical deck.

Design Procedures

Doweled deck design is basically a two-part process involving separate criteria for the glulam panels and interconnecting dowels. First, the glulam panels are designed for the primary moment, shear, and deflection acting between beams in the *x* direction, parallel to the length of the laminations

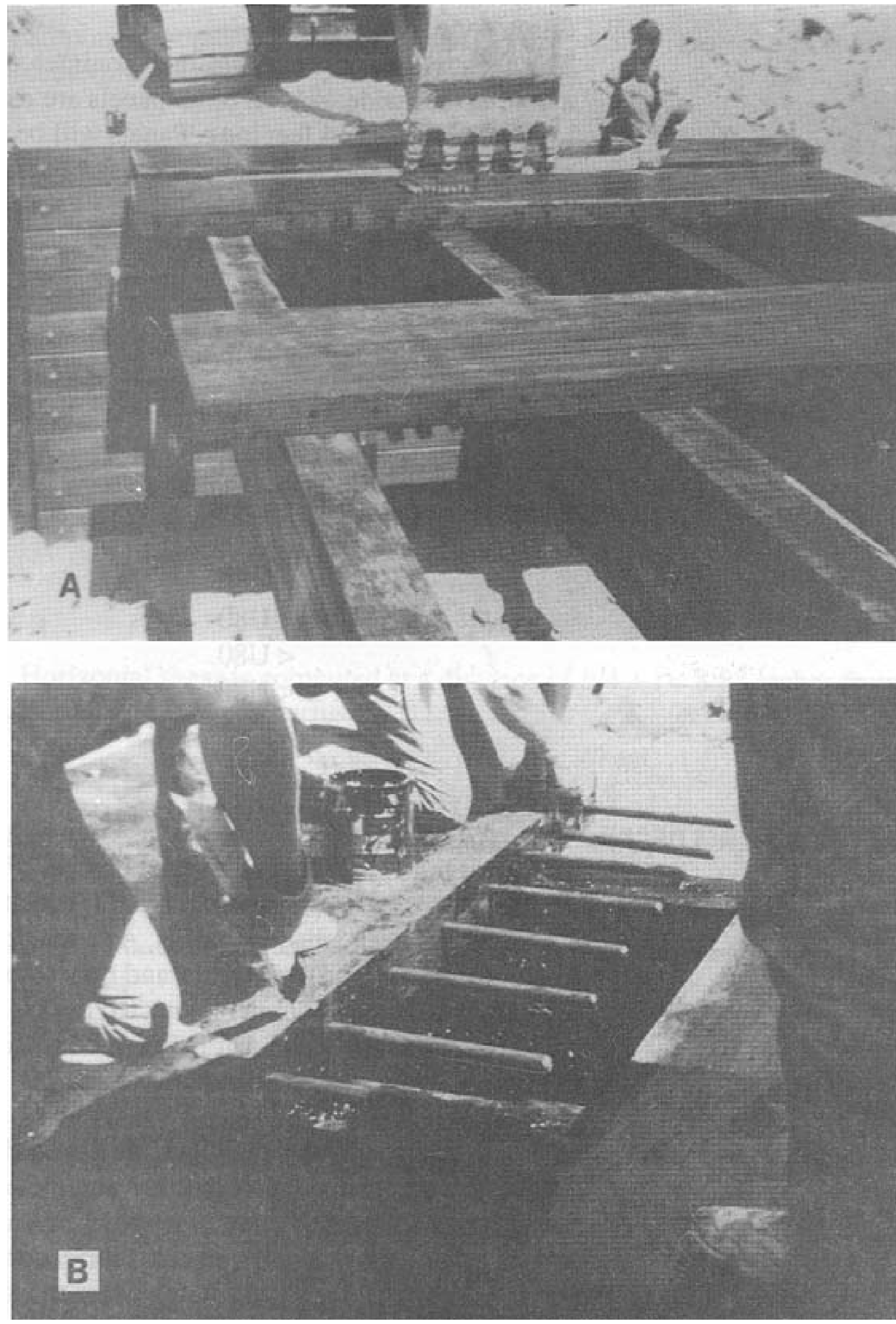


Figure 7-20 - Construction of a doweled glulam deck. The panels are (A) lifted into position and (B) interconnected with steel dowels (photos courtesy of Steve Bunnell, USDA Forest Service).

(Figure 7-21). These strength computations are based on the maximum unit stress acting in the panels. Second, the size and spacing of the dowels are determined from the average secondary moment and shear acting parallel to the supporting beams in the *y* direction, perpendicular to the length of the laminations. These computations assume that the dowels provide deck continuity for the length of the bridge.

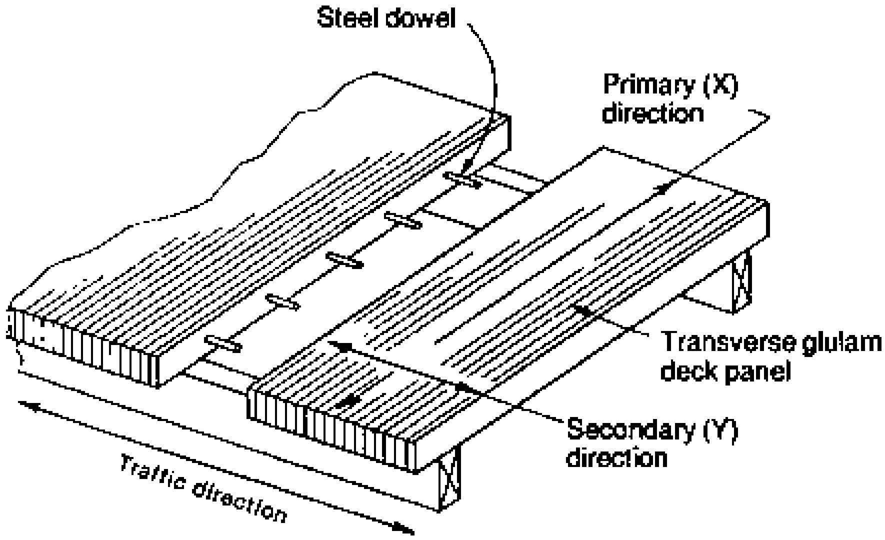


Figure 7-21. - Primary and secondary directions for doweled glulam deck panels.

Basic design procedures for doweled glulam decks are given below in a sequential order used for most design applications. The procedures were adopted by AASHTO in 1975 based on research conducted at the USDA Forest Service, Forest Products Laboratory.[29,30] They are based on experimental and analytical analyses of the deck as an orthotropic plate, acting as a simple span between two supports. The procedures were developed for single wheel loads of 12,000 pounds and 16,000 pounds and are valid for effective spans of 122 inches or less for standard track widths of 6 feet.

1. **Define the deck span, design loads, and panel size.**

The effective deck span, *s*, is the clear distance between supporting beams plus one-half the width of one beam, but not greater than the clear span plus the panel thickness (AASHTO 3.25.1.2). The maximum effective span for doweled decks designed by these procedures is 122 inches. Panel configuration should be based on the same considerations previously discussed for noninterconnected glulam decks.

The design load for doweled decks is the maximum wheel load of the design vehicle. Special AASHTO provisions for HS 20-44 and H 20-44 loads on timber decks *do not* apply to doweled decks designed in accordance with these procedures. Wheel loads for standard AASHTO trucks are 16,000 pounds for HS 20-44 and H 20-44, and 12,000 pounds for HS 15-44 and H 15-44.

2. Estimate deck thickness.

Deck thickness, *t*, must be estimated for initial calculations. Use a minimum thickness of 5-1/8 inches (5 inches for Southern Pine) for HS 15-44 and H 15-44 loads (12,000-pound wheel load) and 6-3/4 inches for HS 20-44 and H 20-44 loads (16,000-pound wheel load).

3. Compute the primary dead load moment and vertical shear.

Dead load moment and shear are based on the unit dead load, DL, of the deck and wearing surface, including allowance for future wearing surface overlays. Primary dead load moment is computed at the effective span centerline by

$$M_{DLx} = \frac{\mathrm{DL}\, s^2}{1152} \tag{7-26}$$

where M_{DLx} = primary dead load moment (in-lb/in), and

DL = dead load of the deck and wearing surface (lb/ft^2).

Primary dead load vertical shear is computed at a distance *t* from the support by

$$R_{DLx} = \frac{\mathrm{DL}}{144}\left(\frac{s}{2} - t\right) \tag{7-27}$$

where R_{DLx} = primary dead load vertical shear (in-lb/in).

4. Determine primary live load moment and vertical shear.

Primary live load moment and vertical shear are computed directly, assuming the deck to act as a simple span between supporting beams (AASHTO 3.25.1.3):

$$M_x = P\left[\left(0.51 \log_{10} s\right) - K\right] \tag{7-28}$$

$$R_x = 0.034P \tag{7-29}$$

where M_x = primary live load bending moment (in-lb/in),

P = design wheel load (lb),

K = design constant based on the wheel load contact area, and

R_x = primary live load vertical shear (lb/in).

Design values for *P*, *K*, and R_x for standard highway loads are given in Table 7-10.

Table 7-10. - Design values for primary live load moment and shear for doweled glulam deck panels.

Vehicle Type	P (lb)	K[a]	R_x (lb/in)
HS 20-44 and H 20-44	16,000	0.51	544
HS 15-44 and H 15-44	12,000	0.47	408

[a] For wheel loads greater than 16,000 pounds, $K = 0.51$ may be used with slightly conservative results.

5. Select a panel combination symbol and compute allowable stresses.

Select an axial combination symbol from Table 2 of *AITC 117--Design* based on the same selection criteria given for noninterconnected panels. Compute allowable stresses for bending and horizontal shear by adjusting tabulated values by all applicable modification factors:

$$F_b' = F_{by} C_F C_M \tag{7-30}$$

$$F_v' = F_{vy} C_M \tag{7-31}$$

F_b' and F_v' may be increased by a factor of 1.33 for overloads in AASHTO Load Croup IB.

6. Compute required deck thickness.

Deck thickness is based on the most restrictive requirements for primary moment or horizontal shear, but the nominal deck thickness cannot be less than 6 inches (actual thickness of 5-1/8 inches for western species or 5 inches for Southern Pine) (AASHTO 3.25.1.1). The minimum required deck thickness is obtained from

$$t = \sqrt{\frac{6(M_x + M_{DLx})}{F_b'}} \tag{7-32}$$

$$t = \frac{3(R_x + R_{DLx})}{2F_v'} \tag{7-33}$$

whichever is the largest (AASHTO 3.25.1.3).

When the deck is continuous over more than two spans, M_{DLx} and M_x used in Equation 7-32 are 80 percent of the simple-span values computed by Equations 7-26 and 7-28 to account for the effects of span continuity.

The required deck thickness may be computed for several combination symbols to obtain the most economical panel. When the required deck thickness varies significantly from the estimated thickness, dead load moment, M_{DLx}, and vertical shear, R_{DLx}, must be revised.

7. Check live load deflection.

Maximum live load deflection in the primary direction is computed by

$$\Delta_{LL} = \frac{0.51 Ps(s-10)}{E' t^3} \quad \textbf{(7-34)}$$

where $E' = E_y C_M$.

When the deck is continuous over more than two spans, the live load deflection is 80 percent of the deflection computed by Equation 7-34 to account for span continuity.

The recommended deflection limits for doweled glulam decks are the same as those previously discussed for noninterconnected glulam decks. Maximum effective deck spans based on an allowable deck deflection of 0.10 inch are given in Table 7-11 for decks continuous over more than two spans.

Table 7-11. - Approximate maximum effective span for doweled transverse glulam deck panels based on a maximum vehicle live load deflection of 0.10 inch; deck continuous across more than two spans.

		Approximate maximum deck span (in.)		
E (lb/in²)	E' (lb/in²)	t = 5 in. or t = 5-1/8 in.	t = 6-3/4 in.	t = 8-1/2 in. or t = 8-3/4 in.
		12,000-lb wheel load		
1,300,000	1,082,900	58	88	>110
1,400,000	1,166,200	60	91	>110
1,500,000	1,249,500	64	94	>110
1,700,000	1,416,100	68	100	>110
1,800,000	1,499,400	70	103	>110
		16,000-lb wheel load		
1,300,000	1,082,900	51	77	>110
1,400,000	1,166,200	53	80	>110
1,500,000	1,249,500	57	82	>110
1,700,000	1,416,100	60	87	>110
1,800,000	1,499,400	61	90	>110

$E' = EC_M = 0.833\ E$.

8. Compute secondary moment and shear.

Requirements for the number and size of dowels are based on the secondary live load moment and shear (AASHTO 3.25.1.4). Equations for computing these values depend on the effective deck span, *s*.

When the effective deck span is less than or equal to 50 inches **($s \leq 50$),**

$$M_y = \frac{Ps}{1{,}600}(s-10) \tag{7-35}$$

$$R_y = \frac{6Ps}{1{,}000} \tag{7-36}$$

where M_y = secondary live load moment (in-lb), and

R_Y = secondary live load shear (lb).

When the effective deck span is more than 50 inches ($s > 50$),

$$M_y = \frac{Ps}{20}\frac{(s-30)}{(s-10)} \tag{7-37}$$

$$R_y = \frac{P}{2s}(s-20) \tag{7-38}$$

9. Determine required size and spacing of steel dowels.

The number of dowels required for each deck span is based on the dowel diameter and properties given in Table 7-12. Select a dowel diameter and compute the required number of dowels using

$$n = \frac{1{,}000}{\sigma_{PL}}\left(\frac{R_y}{R_D}+\frac{M_y}{M_D}\right) \tag{7-39}$$

where n = number of steel dowels required for each deck span,

σ_{PL} = proportional limit stress for timber, perpendicular to grain (1,000 lb/in^2 for Douglas Fir-Larch and Southern Pine),

R_D = dowel shear capacity from Table 7-12 (lb), and

M_D = dowel moment capacity from Table 7-12 (in-lb).

The required number of dowels from Equation 7-39 is given for standard AASHTO highway loads in Figure 7-22. Dowel placement is shown in Figure 7-23.

Table 7-12. - Properties and required lengths of steel dowels for doweled glulam deck panels.

Dowel diameter (in.)	Shear capacity R_D (lb)	Moment capacity M_D (in-lb)	Steel stress coefficients C_R (1/in²)	Steel stress coefficients C_M (1/in³)	Required dowel length (in.)
1/2	600	850	36.9	81.5	8.5
5/8	800	1,340	22.3	41.7	10.0
3/4	1,020	1,960	14.8	24.1	11.5
7/8	1,260	2,720	10.5	15.2	13.0
1	1,520	3,630	7.75	10.2	14.5
1-1/8	1,790	4,680	5.94	7.15	15.5
1-1/4	2,100	5,950	4.69	5.22	17.0
1-3/8	2,420	7,360	3.78	3.92	18.0
1-1/2	2,770	8,990	3.11	3.02	19.5

10. Check dowel stress.

Applied stress in the steel dowels must not exceed the allowable stress computed by

$$\sigma_A = 0.8F_y \tag{7-40}$$

$$\sigma = \frac{1}{n}\left(C_R R_y + C_m M_y\right) \tag{7-41}$$

where σ_A = allowable steel stress in bending (AASHTO Table 10.32.1A) (lb/in²),

σ = dowel stress from applied loads (lb/in²),

F_y = minimum specified yield point of the steel dowels (lb/in²), and

C_R, C_m = steel stress coefficients from Table 7-12.

When $\sigma > \sigma_A$, stress in the steel dowels exceeds allowable values and the dowel diameter must be increased.

11. Check deck overhang.

There are no analysis criteria given in AASHTO for checking dowel deck stresses in the overhang at outside beams. Although slightly conservative, it is recommended that overhangs be checked using the same criteria previously discussed for noninterconnected decks, using an effective panel distribution width of 15 inches plus twice the deck thickness (15 + 2*t*).

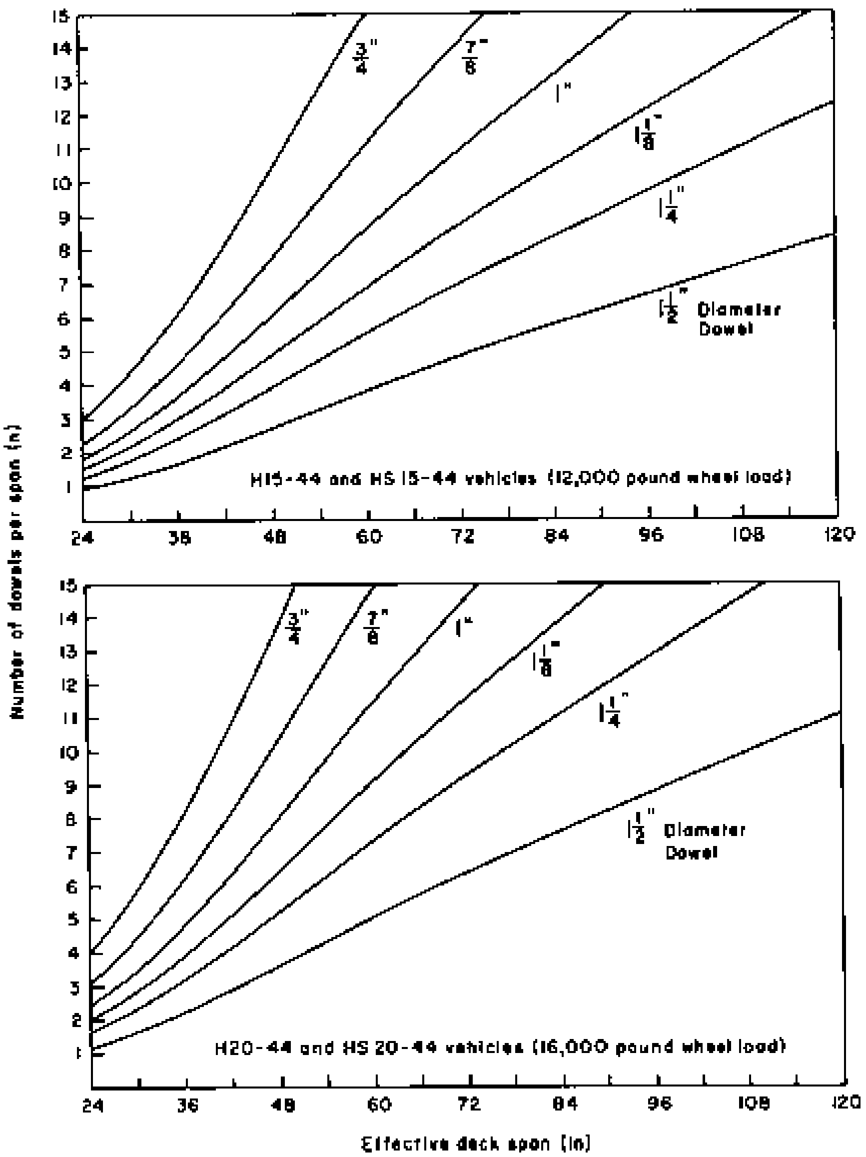


Figure 7-22. - Number of dowels required for each effective span of a doweled glulam deck.

Example 7-7 - Doweled glulam deck with highway loading

A glulam beam bridge spans 71 feet 6 inches out to out and carries two traffic lanes of HS 20-44 loading on a 28-foot-wide roadway. Support is provided by five 12- 1/4-inch-wide glulam beams spaced 6 feet on center, Design a doweled glulam deck for the beam superstructure, assuming

1. glulam deck panels are visually graded western species;
2. rail system dead load is 150 pounds at each post with a maximum post spacing of 6 feet;
3. the deck will be surfaced with 3 inches of asphalt (includes future overlay); and

4. deck live load deflection is limited to approximately 0.10 inch.

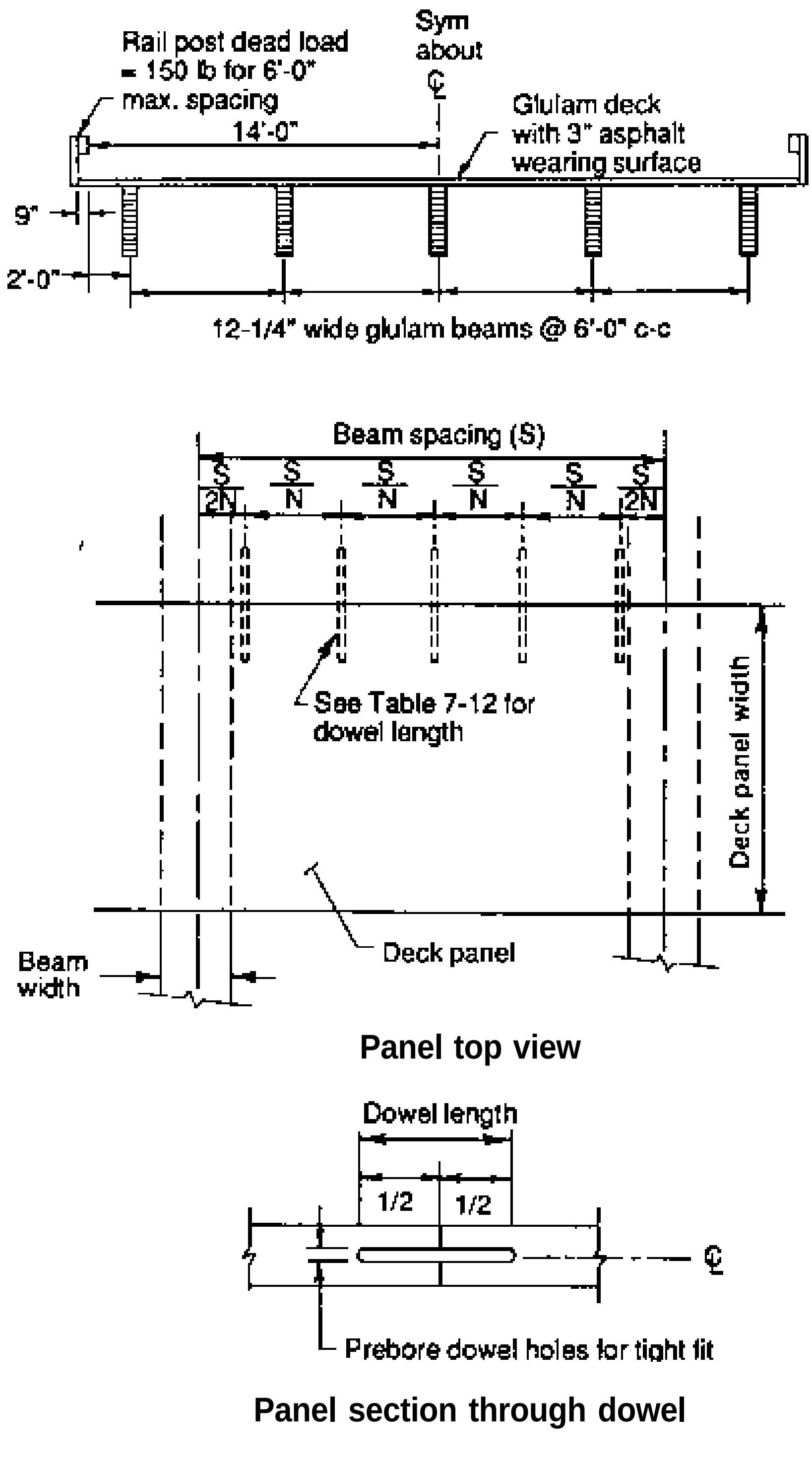


Figure 7-23. - Dowel placement requirements for glulam deck panels.

Solution

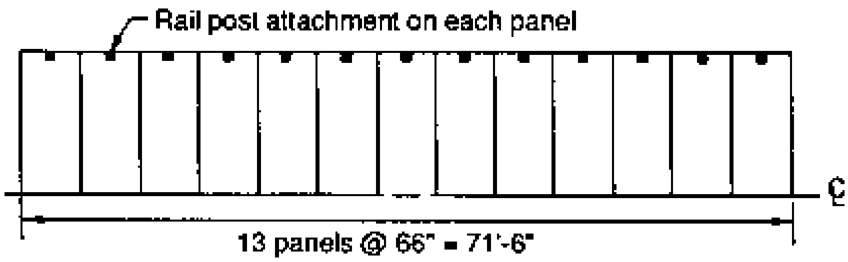
Determine Deck Span, Design Loads, and Panel Size

Clear distance between beams = 72 in.- 12.25 in.= 59.75 in.

$$s = 59.75 \text{ in.} + \frac{12.25}{2} = 65.88 \text{ in.}$$

If a 5-l/8-inch deck is used, *s* will be limited by the clear span plus deck thickness to 59.75 + 5-1/8 = 64.88 inches. For other deck thicknesses, *s* = 65.88 inches will control.

For HS 20-44 loading on doweled decks, AASHTO special wheel load provisions do not apply, and the deck will be designed for a 16,000-pound wheel load. Panel length will be increased 1-1/2 feet over the roadway width for curb/rail attachment. Panel width for an out-to-out bridge length of 71 feet 6 inches will be 66 inches with a railpost attachment centered on each panel (local availability of deck panels in this width may be limited by manufacturing or treating limitations and should be verified).



Estimate Deck Thickness

For HS 20-44 loading, an initial panel thickness of 6-3/4 inches will be used. For this deck thickness, *s* = 65.88 inches.

Compute Primary Dead Load Moment and Vertical Shear

For a 6-3/4-inch deck and 3-inch asphalt wearing surface:

$$\text{DL} = \frac{6.75 \text{ in.}}{12 \text{ in./ft}}\left(50 \text{ lb/ft}^3\right) + \frac{3 \text{ in.}}{12 \text{ in./ft}}\left(150 \text{ lb/ft}^3\right) = 65.6 \text{ lb/ft}^2$$

By Equation (7-26),

$$M_{\text{DL}x} = \frac{\text{DL}s^2}{1{,}152} = \frac{65.6(65.88)^2}{1{,}152} = 247.2 \text{ in-lb/in.}$$

By Equation 7-27,

$$R_{\text{DL}x} = \frac{\text{DL}}{144}\left(\frac{s}{2} - t\right) = \frac{65.6}{144}\left(\frac{65.88}{2} - 6.75\right) = 11.9 \text{ lb/in.}$$

Determine Primary Live Load and Vertical Shear

From Table 7-10 for HS 20-44 loading, P = 16,000 lb and K = 0.51.

By Equation 7-28,

$$M_x = P\,[(0.51 \log_{10} s) - K] = 16{,}000\,[(0.51 \log_{10} 65.88) - 0.51]$$

$$= 6{,}681 \text{ in-lb/in}$$

By Equation 7-29 (or Table 7-10),

$$R_x = 0.034P = 0.034(16{,}000 \text{ lb}) = 544 \text{ lb/in}$$

Select a Panel Combination Symbol and Compute Allowable Stresses

From *AITC 117--Design*, combination symbol No. 1 is selected with the following tabulated values:

$$F_{by} = 1{,}450 \text{ lb/in}^2 \qquad C_M = 0.80$$

$$F_{vy} = 145 \text{ lb/in}^2 \qquad C_M = 0.875$$

$$E_y = 1{,}500{,}000 \text{ lb/in}^2 \qquad C_M = 0.833$$

Allowable stresses are computed:

$$F_b' = F_{by} C_F C_M = 1{,}450(1.07)(0.80) = 1{,}241 \text{ lb/in}^2$$

$$F_v' = F_{vy} C_M = 145(0.875) = 127 \text{ lb/in}^2$$

$$E' = E_y C_M = 1{,}500{,}000(0.833) = 1{,}249{,}500 \text{ lb/in}^2$$

In this case, the deck is continuous over more than two spans and 80 percent of the simple span moments are used to account for span continuity. Minimum required deck thickness based on bending is computed by Equation 7-32:

$$t = \sqrt{\frac{6\left(M_x + M_{DLx}\right)}{F_b'}} = \sqrt{\frac{6\,(0.80)\,(6{,}681 + 247.2)}{1{,}241}} = 5.2 \text{ in.}$$

Minimum required deck thickness based on shear is computed by Equation 7-33:

$$t = \frac{3\left(R_x + R_{DLx}\right)}{2F_v'} = \frac{3\,(544 + 11.94)}{2(127)} = 6.6 \text{ in.}$$

A 6-3/4-inch deck exceeds the minimum 6.6-inch thickness required for shear and is satisfactory.

Check Live Load Deflection

Because the deck is continuous over more than two spans, live load deflection is 80 percent of that computed by Equation 7-34:

$$\Delta_{LL} = (0.80)\frac{0.51Ps\,(s-10)}{E't^3}$$

$$= (0.80)\frac{0.51(16{,}000)(65.88)(65.88-10)}{1{,}249{,}500\,(6.75)^3} = 0.06 \text{ in.}$$

The actual deflection of 0.06 inch is less than the maximum allowable of 0.10 inch, so deck deflection is acceptable.

Compute Secondary Moment and Shear

s = 65.88 in. > 50, so secondary moment and shear are computed by Equations 7-37 and 7-38, respectively:

$$M_y = \frac{Ps}{20}\,\frac{(s-30)}{(s-10)} = \frac{16{,}000(65.88)}{20}\,\frac{(65.88-30)}{(65.88-10)} = 33{,}841 \text{ in-lb}$$

$$R_y = \frac{P}{2s}\,(s-20) = \frac{16{,}000}{2\,(65.88)}\,(65.88-20) = 5{,}571 \text{ lb}$$

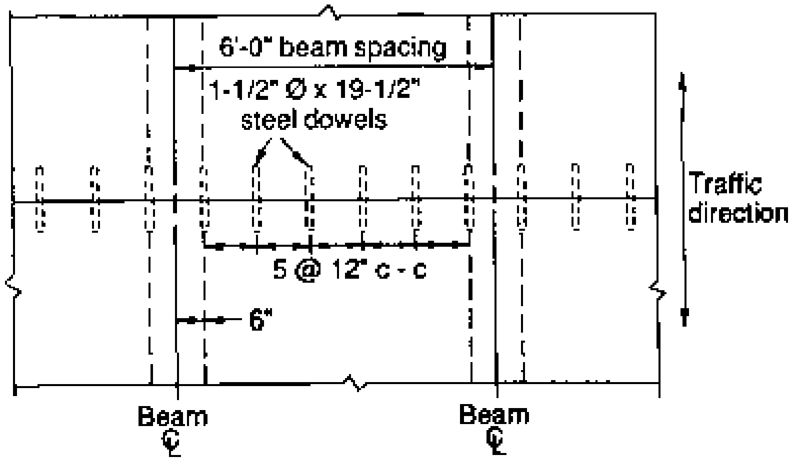
Determine the Required Size and Spacing of Steel Dowels

An estimated number of dowels for various dowel diameters is obtained from Figure 7-11. For an effective deck span of 65.88 inches, the required number of dowels for each deck span varies from approximately 13 for 1-inch-diameter dowels to 6 for 1-1/2-inch-diameter dowels. The 1- 1/2-inch-diameter dowels are selected, and the required number of dowels is confirmed by Equation 7-39 based on the dowel shear and moment capacity given in Table 7-12:

$$n = \frac{1{,}000}{\sigma_{PL}}\left(\frac{R_y}{R_D} + \frac{M_y}{M_D}\right) = \frac{1{,}000}{1{,}000}\left(\frac{5{,}571}{2{,}770} + \frac{33{,}841}{8{,}990}\right) = 5.8 \text{ dowels}$$

Six 1-1/2-inch-diameter dowels per deck span is satisfactory.

From Table 7-12, a minimum dowel length of 19.5 inches is required. The dowel layout obtained from Figure 7-23 is as follows:



Check Dowel Stress

Assuming A36 steel dowels $(F_y$ = 36,000 lb/in^2), allowable dowel stress is computed by Equation 7-40:

$$\sigma_A = 0.80F_y = 0.80(36,000) = 29,000 \text{ lb/in}^2$$

Applied dowel stress is computed by Equation 7-41 based on previously computed values of R_y and M_y and coefficients given in Table 7-12:

$$\sigma = \frac{1}{n}\left(C_R R_y + C_m M_y\right) = \frac{1}{6}\left[3.11\,(5,571) + 3.02\,(33,841)\right]$$

$$= 19,921 \text{ lb/in}^2$$

29,000 lb/in^2 > 19,921 lb/in^2, so dowel stress is acceptable.

Check Overhang

Stresses in the deck overhang are checked in the same manner as for noninterconnected glulam decks, but an increased wheel load distribution for bending of 15 inches plus twice the deck thickness (15 + *2t)* is used for doweled decks. In this case, the deck is thicker and the distribution width greater than the deck overhang previously checked in Example 7-5. Refer to that example for procedures.

Summary

The deck will consist of 13 combination symbol No. 1 glulam panels that are 6-3/4 inches thick, 66 inches wide and 29-1/2 feet long. Panels will be interconnected with 1-1/2-inch-diameter A36 steel dowels, 19-1/2 inches long. The dowels will be spaced 12 inches on center along the deck panel edges.

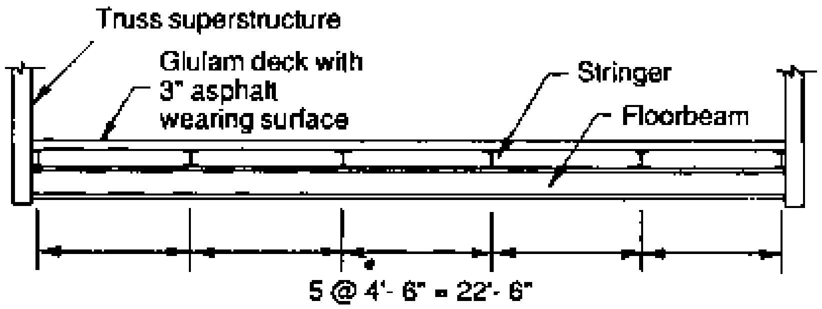
Example 7-8 - Doweled glulam deck with highway loading

An old steel truss is structurally deficient and will be rehabilitated for HS 15-44 loads. As part of the rehabilitation, the existing concrete deck will be removed and replaced with transverse doweled glulam panels. The bridge is 74 feet 3 inches long (out to out) and carries two traffic lanes on a roadway width of approximately 23 feet. Deck support is provided by six steel beams with 7-inch flange widths, spaced 4-1/2 feet on center. Design a doweled glulam deck for this structure, assuming

1. glulam deck panels are manufactured from visually graded Southern Pine;

2. the deck will be surfaced with 3 inches of asphalt (includes future overlay); and

3. deck live load deflection is limited to approximately 0.10 inch.



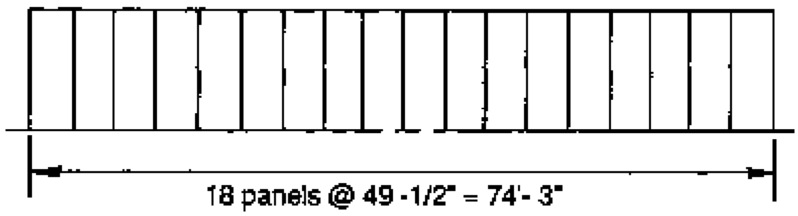
Solution

Determine Deck Span, Design Loads, and Panel Size

Clear distance between beams = 54 in. - 7 in. = 47 in.

$$s = 47 \text{ in.} + \frac{7}{2} = 50.50 \text{ in.}$$

For HS 15-44 loads, the deck will be designed for a 12,000-pound wheel load. Panel width for an out-to-out bridge length of 74 feet 3 inches will be 49-1/2 inches. Panel length will equal the roadway width of 23 feet.



Estimate Deck Thickness

For HS 15-44 loading, an initial panel thickness of 5 inches is selected.

Compute Primary Dead Load Moment and Vertical Shear

For a 5-inch deck and 3-inch asphalt wearing surface,

$$DL = \frac{5 \text{ in.}}{12 \text{ in./ft}}(50 \text{ lb/ft}^3) + \frac{3 \text{ in.}}{12 \text{ in./ft}}(150 \text{ lb/ft}^3) = 58.3 \text{ lb/ft}^2$$

By Equation 7-26,

$$M_{DLx} = \frac{DLs^2}{1152} = \frac{58.3(50.50)^2}{1152} = 129.1 \text{ in-lb/in.}$$

By Equation 7-27,

$$R_{DLx} = \frac{DL}{144}\left(\frac{s}{2} - t\right) = \frac{58.3}{144}\left(\frac{50.50}{2} - 5\right) = 8.2 \text{ lb/in.}$$

Determine Primary Live Load and Vertical Shear

From Table 7-10 for HS 15-44 loading, P = 12,000 pounds and K = 0.47.

By Equation 7-28,

$$M_x = P\ [(0.51 \log_{10} s) - K] = 12{,}000\ [(0.51 \log_{10} 50.50) - 0.47]$$

$$= 4{,}784 \text{ in-lb/in.}$$

By Equation 7-29 (or Table 7-10),

$$R_x = 0.034\,P = 0.034(12{,}000 \text{ lb}) = 408 \text{ lb/in.}$$

Select a Panel Combination Symbol and Compute Allowable Stresses

From *AITC 117--Design*, combination symbol No. 46 is selected with the following tabulated values:

$F_{by} = 1{,}450 \text{ lb/in}^2$ $\quad$ $C_M = 0.80$

$F_{vy} = 175 \text{ lb/in}^2$ $\quad$ $C_M = 0.875$

$E_y = 1{,}300{,}000 \text{ lb/in}^2$ $\quad$ $C_M = 0.833$

Allowable stresses are computed:

$$F_b' = F_{by} C_F C_M = 1{,}450(1.10)(0.80) = 1{,}276 \text{ lb/in}^2$$

$$F_v' = F_{vy} C_M = 175(0.875) = 153 \text{ lb/in}^2$$

$$E' = E_y C_M = 1{,}300{,}000(0.833) = 1{,}082{,}900 \text{ lb/in}^2$$

Using 80 percent of the simple span moments, minimum required deck thickness based on bending is computed by Equation 7-32:

$$t = \sqrt{\frac{6\left(M_x + M_{DLx}\right)}{F_b'}} = \sqrt{\frac{6(0.80)(4{,}784 + 129.1)}{1{,}276}} = 4.3 \text{ in.}$$

Minimum required deck thickness based on shear is computed by Equation 7-33:

$$t = \frac{3\left(R_x + R_{DLx}\right)}{2F_v'} = \frac{3\,(408 + 8.2)}{2\,(153)} = 4.1 \text{ in.}$$

A 5-inch deck meets minimum deck thickness requirements for moment and shear.

Check Live Load Deflection

Live load deflection is 80 percent of that computed by Equation 7-34 to account for span continuity:

$$\Delta_{LL} = (0.80)\,\frac{0.51\,Ps\,(s - 10)}{E' t^3}$$

$$= (0.80)\frac{0.51\,(12{,}000)\,(50.50)\,(50.50 - 10)}{1{,}082{,}900\,(5)^3} = 0.07 \text{ in.}$$

Deck deflection is less than the maximum allowable of 0.10 inch.

Compute Secondary Moment and Shear

s = 50.50 inches > 50, so secondary moment and shear are computed by Equations 7-37 and 7-38, respectively:

$$M_y = \frac{Ps}{20}\,\frac{(s - 30)}{(s - 10)} = \frac{12{,}000\,(50.50)}{20}\,\frac{(50.50 - 30)}{(50.50 - 10)} = 15{,}337 \text{ in-lb}$$

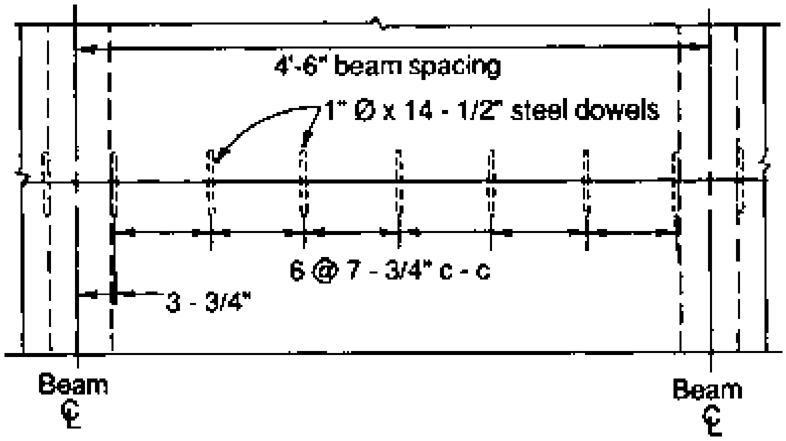
$$R_y = \frac{P}{2s}\,(s - 20) = \frac{12{,}000}{2\,(50.50)}\,(50.50 - 20) = 3{,}624 \text{ lb}$$

Determine the Required Size and Spacing of Steel Dowels

From Figure 7-22, a 1-inch dowel diameter is selected. The required number of dowels is computed by Equation 7-39:

$$n = \frac{1{,}000}{\sigma_{PL}}\left(\frac{R_y}{R_D} + \frac{M_y}{M_D}\right) = \frac{1{,}000}{1{,}000}\left(\frac{3{,}624}{1{,}520} + \frac{15{,}337}{3{,}630}\right) = 6.6 \text{ dowels}$$

Seven dowels 14.5 inches long will be used for each deck span. Spacing from Figure 7-23 is slightly adjusted to the closest 1/4 inch:



Check Dowel Stress

For A36 steel dowels, allowable dowel stress is computed by Equation 7-40:

$$\sigma_A = 0.80F_y = 0.80(36{,}000) = 29{,}000 \text{ lb/in}^2$$

Applied dowel stresses are computed by Equation 7-41:

$$\sigma = \frac{1}{n}\left(C_R R_y + C_m M_y\right) = \frac{1}{7}\left[7.75\,(3{,}364) + 10.20\,(15{,}337)\right]$$

$$= 26{,}360 \text{ lb/in}^2$$

29,000 lb/in^2 > 26,360 lb/in^2, so dowel stress is acceptable.

Summary

The deck will consist of 18 glulam deck panels that are 5 inches thick, 49-1/2 inches wide, and 23 feet long. Panels will be manufactured from visually graded Southern Pine, combination symbol No. 46. Panels will. be interconnected with 1-inch-diameter by 14-1/2-inch-long A36 steel dowels, placed between panels at 7-3/4 inches on center.

GLULAM DECK ATTACHMENT

Glulam decks are attached to supporting beams with mechanical fasteners such as bolts and lag screws. The attachments must securely hold the panels and transmit longitudinal and transverse forces from the deck to the beams. They should also be easy to install and maintain and be adjustable for construction tolerances in deck alignment. The most desirable connection requires no field fabrication where holes or cuts made after preservative treatment increase susceptibility to decay.

The performance of deck attachments is affected primarily by live load deflection in the panels. Deflections cause attachments to loosen from vibrations and from panel rotation about the support. The larger the deflection, the more significant the effects. Acceptable panel deflection is difficult to quantify and should be based on the best judgment of the designer. Recommended maximum deck deflections given in preceding discussions should provide acceptable attachment performance.

Some of the common attachment configurations for glulam panels on timber or steel beams are discussed below. The attachments are sufficient to resist vertical loads, longitudinal forces from vehicle braking, and transverse forces from wind on the vehicle. A decreased spacing may be required when centrifugal forces are applied. Although the attachments also provide a varying degree of lateral beam support, such support is currently not recognized in design.

Attachment to Glulam Beams

Glulam decks are placed directly on glulam beams without material at the deck-beam interface. Material such as roofing felt placed between the deck and beam is not recommended because the material can decompose with age and hold moisture, enhancing conditions for decay. Deck panels are attached to beams with bolted brackets that connect to the beam side, or with lag screws that are placed through the deck and into the beam top. The bracket configuration uses a cast aluminum alloy bracket (Weyco bracket) that bolts through the deck and connects to the beam in a routed slot (Figure 7-24). It includes small teeth that firmly grip the deck and beam but do not penetrate through the preservative treatment. This bracket, which is available from a number of glulam suppliers and manufacturers, is the preferred attachment for glulam beams because it provides a tight connection, does not alter the preservative effectiveness, and is easily tightened in service.

When panels are attached with lag screws, the screws are placed through the panel and into beam tops (Figure 7-25). It is impractical to drill beam lead holes before pressure treatment; therefore, holes must be field bored and treated before placing the screws. Lag screw attachments are not recommended because the field boring increases the susceptibility to beam and deck decay, and they are not accessible for tightening if the deck is paved.

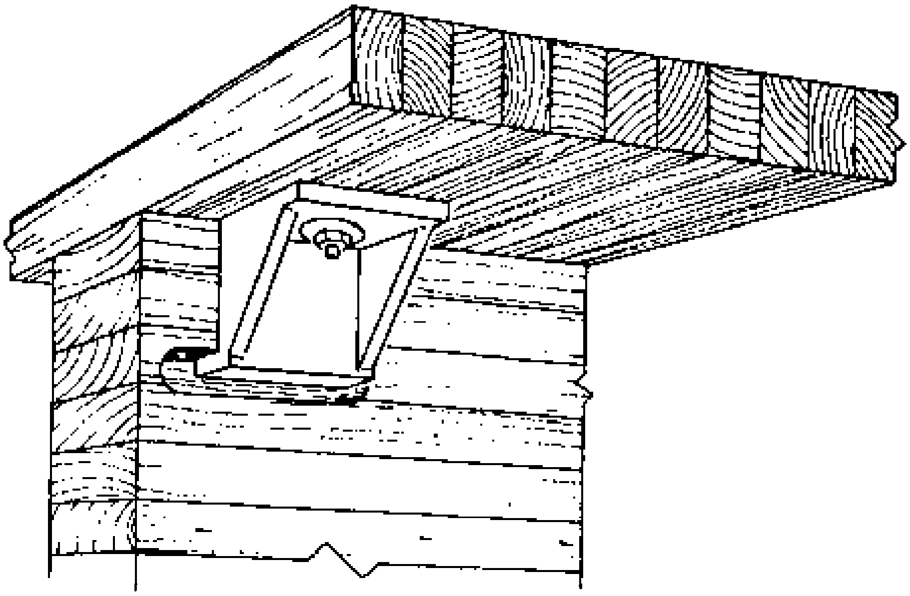


Figure 7-24. - Aluminum deck bracket for attaching glulam decks to glulam beams.

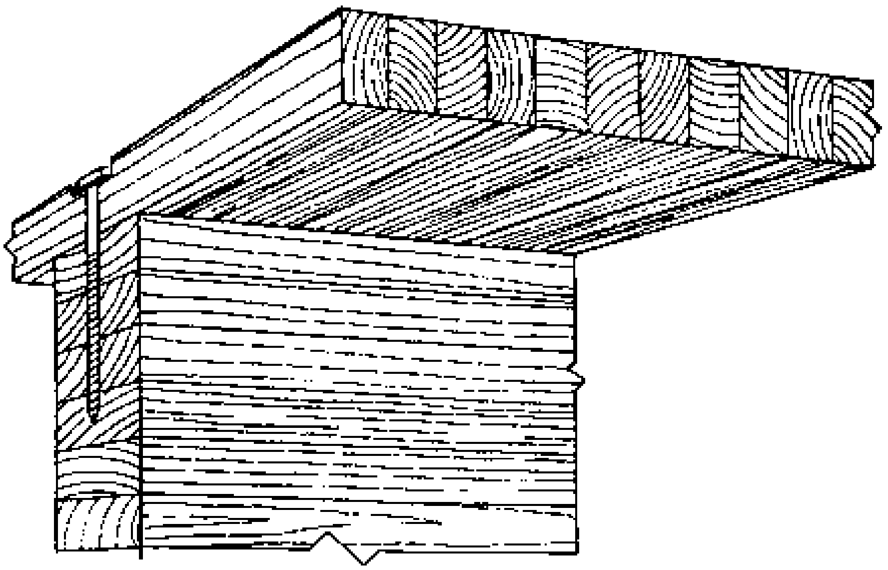


Figure 7-25. - Lag screw connection for attaching glulam decks to glulam beams.

Attachment to Steel Beams

Glulam decks are used on steel beams in new construction and rehabilitation of existing structures. Panels are placed directly on the beams with no special treatment to the top beam flange; however, when panels are placed on unpainted weathering steel beams (AASHTO M 222), a corrosion coating on the top flange should be considered to reduce the potential for steel corrosion at the panel-flange interface. The most suitable attachment for steel beams is a bracket connection that bolts through the panel and over the top beam flange. Through-bolting of the panel directly to the flange is not recommended because it allows little or no tolerance for placement or minor panel movements from variations in moisture content or thermal expansion of the steel.

The most common attachments for glulam panels on steel beams are the C-clip and angle bracket. A C-clip is a galvanized, forged-steel bracket that bolts through the panel and over the top beam flange (Figure 7-26). The clip is provided with small teeth on the deck side to prevent rotation of the bracket without penetrating the preservative envelope. C-clips are commercially available from several glulam suppliers and manufacturers and are suitable for use on beam flanges of approximately 3/4 inch or less. For thicker flanges, the angle bracket is used. Angle brackets are galvanized steel brackets fabricated from standard A36 steel angles (Figure 7-27). They are similar in connection and performance to C-clips, but can be fabricated locally. Angle clips are cut from standard 1/4- or 5/16-inch angle stock and leg dimensions can be varied for any flange thickness.

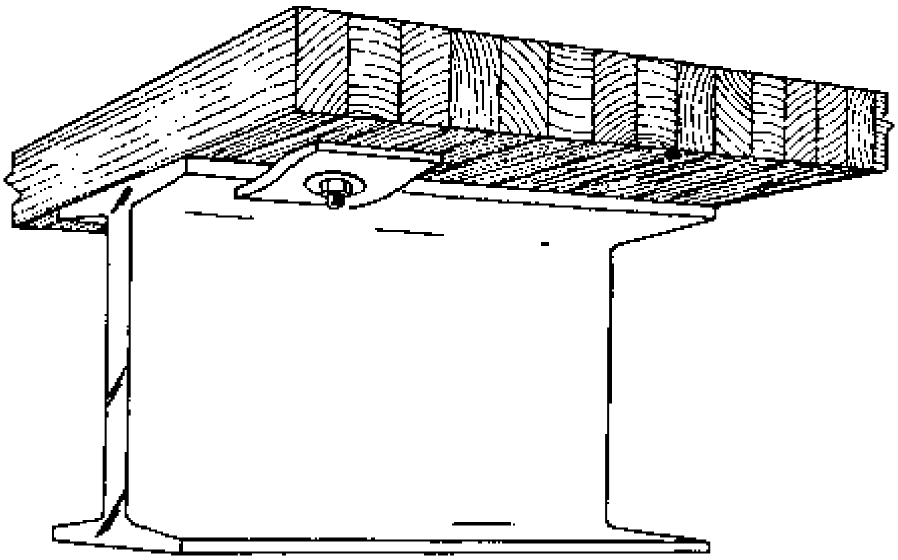


Figure 7-26. - C-clip for attaching glulam decks to steel beams.

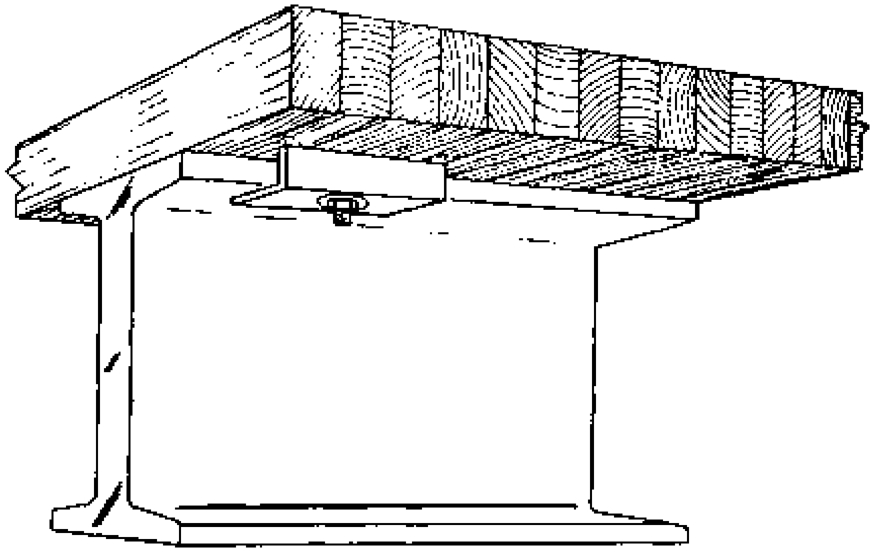


Figure 7-27. - Steel angle bracket for attaching glulam decks to steel beams.

ADDITIONAL DETAILS AND CONSIDERATIONS FOR GLULAM DECKS

Design details for fabrication and placement of bridge components can influence performance and should be suited to specific project needs. Several common details used with glulam deck panels are discussed below. The applicability of these details will vary for different projects and is left to designer judgment.

Transverse Joint Configuration

A bridge deck should provide a watertight roof over beams and other components of the superstructure. Glulam panels are especially suited for this purpose because of their relatively large size. Glulam decks can be made watertight by sealing the joint between adjacent panels with a bituminous mastic sealer (roofing cement is commonly used). It is recommended that the sealer be brushed or spread on panel edges just before placement, but some sealers can be poured into the joint after panels are set (Figure 7-28). Joint sealing is inexpensive and can contribute significantly to long structure life. It is strongly recommended for all panel configurations.

Dimensional Stability

Although glulam exhibits a much higher dimensional stability than sawn lumber, it can be affected by substantial changes in moisture content. The magnitude and effects of moisture changes are greatly reduced when panels are treated with oil-type preservatives and protected with a watertight asphalt wearing surface.[7,29] Cases involving problems with dimensional stability are not common; however, the designer should be aware of the potential for swelling or shrinkage as well as the steps to reduce or eliminate their effects.

Figure 7-28. - Bituminous sealer is spread on the edges of glulam deck panels to waterproof the panel joints.

The biggest adjustment in moisture content normally occurs during the first 2 years after construction when the panels reach equilibrium moisture content with the environment. After equilibrium is reached, subsequent changes in moisture content from seasonal variations occur gradually and have a relatively minor effect on the member. Glulam is manufactured at a moisture content of 16 percent or less, which may be reduced slightly when treated with oil-type preservatives. The panel moisture content is also affected by storage conditions between manufacture and installation. When installed in arid regions, some checking of panel ends may occur as panels dry and subsequently shrink in service. In such locations, shrinkage can be reduced if a lower panel moisture content is specified when the material is ordered. As discussed in Chapter 3, maximum moisture contents as low as 10 percent may be specified for glulam based on designer judgment. Although lower moisture contents will slightly increase costs, the potential for panel shrinkage can be greatly reduced.

In contrast to shrinkage, swelling may occur when dry panels (moisture content less than 16 percent) are installed in wet or humid areas without the protection of a watertight wearing surface. There has been at least one case where significant swelling occurred in panels protected with an asphalt wearing surface, although this condition is very rare. Swelling can cause breaks in the wearing surface, substructure backwalls, curbs, and railing depending on the magnitude of the moisture changes and the bridge span. Little can be done to increase panel moisture content for installation. In cases where the bridge is over 50 feet long, and the deck moisture content is expected to exceed 18 percent (as when unpaved decks are used in warm, humid climates), a transverse joint or gap of approximately

1/2 inch between every third or forth panel will allow the necessary room for potential expansion. If the deck is not paved and if beams are designed for wet-condition stresses, the gap can be left open, based on designer judgment. A preferable solution is to seal the gap with metal flashing or commercial joint material that will allow some panel movement.

Nosing Angles

Steel nosing angles are placed on the edge of end panels to minimize damage from vehicle impact and abrasion. They are used when approach roads are unpaved or when the potential for vehicle impact exists. The angles are generally galvanized and are attached to the deck with lag screws.

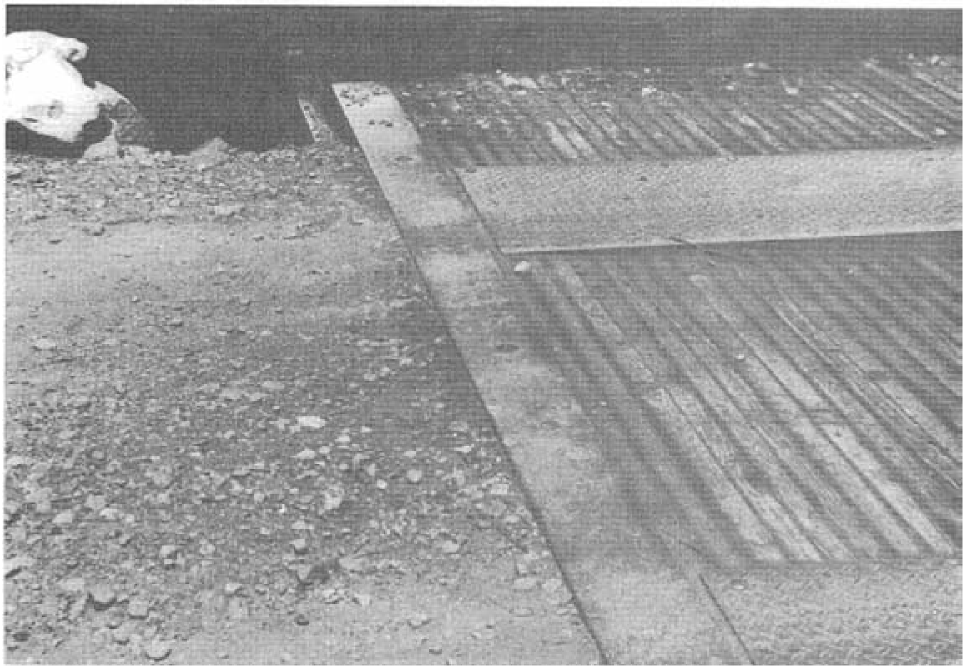


Figure 7-29. - Steel nosing angle placed across an unpaved deck to reduce damage from vehicle impact and abrasion.

PART II: SAWN LUMBER SYSTEMS

7.6 GENERAL

Sawn lumber beam bridges consist of a series of closely spaced lumber beams supporting a transverse nail-laminated or plank deck (Figure 7-30). For AASHTO highway loads, they are most practical for clear spans up to approximately 25 feet, when sawn lumber in the required sizes is available. Longer crossings are made with a series of single spans, usually in a trestle arrangement. Lumber beam bridges are among the oldest and simplest of all bridge types and were widely used in the United States through the 1950's. Their use has declined significantly over the past 20 years because of the popularity of glulam and its increased member size and improved performance. It has also become increasingly difficult to obtain sawn lumber beams in the sizes and grades typically required for bridges.

Figure 7-30. - Typical sawn lumber beam bridge with a transverse nail-laminated deck.

The following sections address design considerations, procedures, and details for sawn lumber beam bridges with transverse nail-laminated or plank decks. Although design with sawn lumber differs from glulam because of smaller member sizes and the wider variety of species and grades, many of the concepts are the same. When possible, reference will be made to previous material discussed for glulam.

7.7 DESIGN OF BEAMS AND BEAM COMPONENTS

As with other beam superstructures, sawn lumber beam systems consist of beams, transverse bracing, and bearings. Design considerations and procedures are addressed in that order.

BEAM DESIGN

Sawn lumber beams are designed from the species and grades of visually graded lumber given in Table 4A of the NDS.[37] Although any species can be used provided it is treatable with preservatives, most bridges are constructed from Douglas Fir-Larch or Southern Pine because of the high strength and availability of these species.

Douglas Fir-Larch beams are generally available in widths up to 16 inches, depths up to 24 inches, and lengths up to 40 feet. There may be a substantial price premium for larger sizes, however, and 6- to 8-inch widths up to 16 inches deep are normally most economical. Beams are most efficiently designed from the Beams and Stringers (B&S) size classification where tabulated bending stress, F_b, is based on loads applied to the narrow face of the member (Beams and Stringers are sawn lumber of rectangular cross section, 5 or more inches thick with the width more than 2 inches greater than the thickness). Grades for bridge beams in this classification are normally No. 1 or Select Structural. Beams can also be specified from the Posts and Timbers (P&T) size classification but these sizes generally do not provide the most efficient section in bending (Posts and Timbers are sawn lumber of square or approximately square cross-section, 5 by 5 inches and larger, with the width not more than 2 inches greater than thickness). When P&T sizes are graded to B&S requirements, design values for the applicable B&S grades may be used.

For Southern Pine, beams are generally available in widths up to 10 inches, depths up to 12 inches, and lengths up to 24 feet. Grades for bridge beams are normally Dense Structural 72 or Dense Structural 65 in the 2-1/2 inches and thicker size classification. Southern Pine does not follow many of the conventions and standards used for other species, and the designer should carefully check design tables for footnotes. Beams are generally specified from the table noted "surfaced green; used any conditions." Values in this table have been adjusted for wet-use conditions and further adjustment by C_M is not required.

Bridge beams can be specified as surfaced (S4S), rough-sawn, or full-sawn (Chapter 3). Rough- or full-sawn lumber should be edge planed (S2E) to ensure an even depth for all members. When design is based on rough- or full-sawn sizes, the applicable moisture content and size used for design must be clearly indicated on the specifications and drawings.

Live Load Distribution

Vehicle live load distribution criteria for moment, shear, and reactions in sawn lumber beams follow the same basic criteria previously discussed for glulam. However, because the distribution factors for moment are based on the relative deck stiffness, different interior beam DF equations are required for the various decks used on lumber beams. Empirical equations from AASHTO for computing interior beam distribution factors for plank and nail-laminated lumber decks are given in Table 7-13. Examples of live load distribution for sawn lumber beams are included in examples later in this section.

Table 7-13. - Interior beam live load distribution factors for plank and nail-laminated timber decks.

	DF for moment (wheel lines/beam)	
Deck type[a]	**Bridges designed for one traffic lane**	**Bridges designed for two or more traffic lanes**
Plank	$S/4.0$	$S/3.75$
Nail-laminated; 4 in. thick or multiple layer floors over 5 in. thick[b]	$S/4.5$	$S/4.0$
Nail-laminated; 6 in. or more thick	$S/5.0$ If S exceeds 5 ft, use footnote c.	$S/4.25$ If S exceeds 6.5 ft, use footnote c.

[a] Deck thickness is based on nominal thickness.

[b] Multiple layer floors consist of two or more layers of planks, each layer being laid at an angle to the other.

[c] In this case, the distribution factor for each beam is the reaction of the wheel lines, assuming the deck between beams to act as a simple beam.

S = average center-to-center beam spacing (feet).

From AASHTO[1] Table 3.23.1; © 1983. Used by permission.

Beam Configuration

The number and spacing of beams can affect the overall economy and performance of the sawn lumber bridges in many of the same ways previously discussed for glulam. The effects are normally less pronounced, however, because beam spacing is often controlled primarily by strength requirements and material availability. Because of the large number of species, grades, and sizes of lumber beams, specific recommendations on bridge beam configuration are impractical. In general terms, the designer should first check material availability, then try several configurations to determine the most economical combination that meets strength and stiffness requirements.

Site restrictions are normally not a problem with sawn lumber beams because beams are not available in large depths. Deck considerations can influence beam spacing, although to a lesser degree than for glulam. Nominal 4-inch-thick plank decks are feasible for spacings up to approximately 20 inches, while nail-laminated decks are practical for spans up to approximately 38 inches for nominal 4-inch decks and 72 inches for nominal 6-inch decks. The most significant deck effect on beam spacing is at the break between a 4-inch and a 6-inch nail-laminated deck where cost savings for the thinner deck may be greater than the increased cost for closer beam spacing.

Perhaps the most important consideration in lumber beam configuration is the live load distribution to outside beams. The most suitable design is one where moment distribution factors are approximately equal for all beams, interior and outside. This allows the use of one beam size and grade across the width of the structure. The outside beam distribution factor is controlled by limiting the deck overhang so that the reaction at the beam in wheel lines does not exceed the interior beam DF given in Table 7-13.

Beam Design Procedures

Design procedures for sawn lumber beams follow the same basic procedures used for glulam timber. Minor differences in procedures and criteria are illustrated in the following examples.

Example 7-9 - Lumber beam design; two-lane HS 15-44 loading

A lumber beam bridge is required to span 17 feet center to center of bearings and support two lanes of HS 15-44 loading over a roadway width of 24 feet. The deck is nominal 4-inch-thick nail-laminated lumber with a full sawn 3-inch timber wearing surface. Design the beam system for this structure, assuming

1. beam spacing is limited by deck requirements to a maximum of 26 inches;

2. a curb and vehicular railing are provided with an approximate dead load of 60 lb/ft;

3. all lumber except the wearing surface is dressed (S4S);

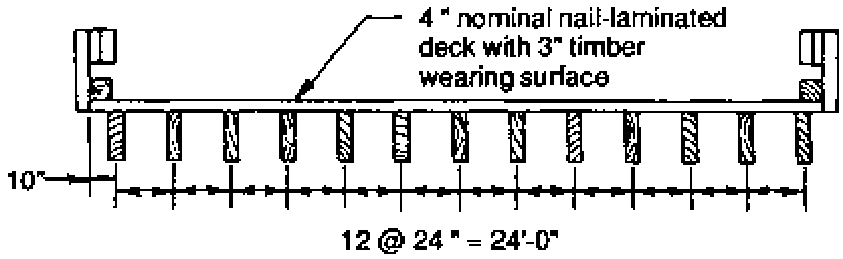
4. beams are visually graded Douglas Fir-Larch;

5. beam live load deflection must not exceed *L/360;* and

6. AASHTO requirements for Load Group IA do not apply.

Solution

From the given information, an initial configuration of 13 beams spaced 24 inches on center is selected. The face of the rail is aligned with the outside beam centerline with an additional 10-inch deck extension for the curb and rail attachment:



Select Lumber Species and Grade

From NDS Table 4A, an initial beam species and grade are selected as Douglas Fir-Larch, visually graded No. 1 in the Beams and Stringers (B&S) size classification (WWPA rules). Tabulated values are as follows:

$$F_b = 1{,}350 \text{ lb/in}^2$$

$$F_v = 85 \text{ lb/in}^2$$

$$F_{c\perp} = 625 \text{ lb/in}^2$$

$$E = 1{,}600{,}000 \text{ lb/in}^2$$

Compute Deck Dead Load and Dead Load Moment

Dead load of the deck (3-1/2 inches actual thickness) and wearing surface is computed as

$$\text{DL} = \frac{(3.5 \text{ in.} + 3 \text{ in.})(50 \text{ lb/ft}^3)}{12 \text{ in./ft}} = 27.1 \text{ lb/ft}^2$$

For interior beams, each beam supports a tributary deck width of 2 feet:

$$\text{Deck } w_{DL} = 2 \text{ ft } (27.1 \text{ lb/ft}^2) = 54.2 \text{ lb/ft}$$

$$\text{Deck } M_{DL} = \frac{w_{DL} L^2}{8} = \frac{54.2\,(17)^2}{8} = 1{,}958 \text{ ft-lb}$$

For outside beams, each beam supports 1 foot of combined deck and wearing surface, 10 inches (0.83 feet) of deck only and 60 lb/ft of curb and railing:

$$\text{Deck } w_{DL} = (1\text{ ft})(27.1\text{ lb/ft}^2) + 0.83\text{ ft}\left[\frac{(3.5\text{ in.})(50\text{ lb/ft}^3)}{12\text{ in./ft}}\right] = 60\text{ lb/ft}$$

$$= 99.2\text{ lb/ft}$$

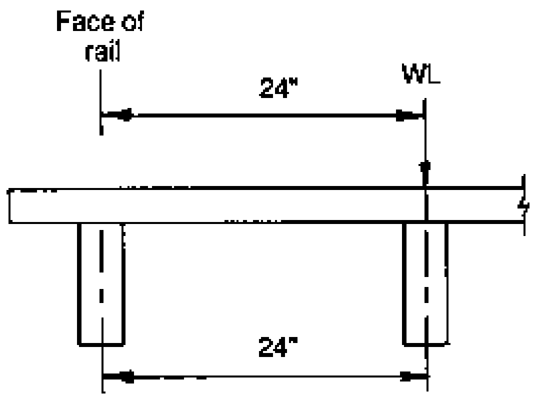
$$\text{Deck } M_{DL} = \frac{w_{DL}L^2}{8} = \frac{99.2(17)^2}{8} = 3{,}584\text{ ft-lb}$$

Compute Live Load Moment

The equation for the interior beam moment DF is obtained from Table 7-13:

$$\text{Interior beam DF} = \frac{S}{4} = \frac{2}{4} = 0.50\text{ WL/beam}$$

The outside beam moment DF is computed by positioning the wheel line 2 feet from the rail face, assuming the deck acts as a simple span between beams. In this case, the rail face is aligned with the outside beam centerline and the wheel line is directly over the first interior beam:



The moment DF to outside beams is technically zero; however, AASHTO requires that the DF to outside beams not be less than that to interior beams. The moment DF is therefore 0.50 WL/beam.

From Table 16-8, the maximum moment for one wheel line of an HS 15-44 truck on a 17-foot span is 51 ft-k. The design live load moment is computed by multiplying the maximum moment for one wheel line by the moment DF:

$$M_{LL} = M(\text{DF}) = 51(0.50)(1{,}000\text{ lb/k}) = 25{,}500\text{ ft-lb}$$

Determine Beam Size Based on Bending

The allowable stress in bending is equal to tabulated stress adjusted by all applicable modification factors. In this case

$$F_b' = F_b C_M C_F$$

At this point the beam size, dead load, C_M and C_F are unknown. Assuming a beam dead load of 50 lb/ft, an initial interior beam size is computed based on the tabulated bending stress:

$$\text{Estimated Beam } M_{DL} = \frac{w_{DL} L^2}{8} = \frac{50\,(17)^2}{8} = 1,806 \text{ ft-lb}$$

Using the inside beam M_{DL} = 1,958 ft-lb,

$$M = (\text{Beam } M_{DL} + \text{Deck } M_{DL}) + M_{LL} = (1,806 + 1,958) + 25,500$$

$$= 29,264 \text{ ft-lb}$$

$$S = \frac{M}{F_b} = \frac{29,264(12 \text{ in./ft})}{1,350} = 260.13 \text{ in}^3$$

From Table 16-2, an initial interior beam size of 6 by 18 inches is selected with the following properties:

b = 5-1/2 in.	S = 280.73 in^3
d = 17-1/2 in.	I = 2,456.38 in^4
A = 96.25 in^2	w_{DL} = 33.4 lb/ft

Modification factors and the allowable bending stress are computed as follows:

From Table 5-7, C_M = 1.0 for lumber 5 inches or thicker.

$$C_F = \left(\frac{12}{d}\right)^{1/9} = \left(\frac{12}{17.5}\right)^{1/9} = 0.96$$

$$F_b' = F_b C_M C_F = 1350(1.0)(0.96) = 1,296 \text{ lb/in}^2$$

Bending stress is computed based on the actual beam dead load:

$$\text{Beam } M_{DL} = \frac{w_{DL} L^2}{8} = \frac{33.4\,(17)^2}{8} = 1,207 \text{ ft-lb}$$

$$M = (\text{Beam } M_{DL} + \text{Deck } M_{DL}) + M_{LL}$$

$$= (1,207 + 1,958) + 25,500 = 28,665 \text{ ft-lb}$$

$$f_b = \frac{M}{S} = \frac{28,665(12 \text{ in/ft})}{280.73} = 1,225 \text{ in}^2$$

f_b = 1,225 lb/in^2 < F_b' = 1,296 lb/in^2, so 6- by 18-inch beams are satisfactory in bending for interior beams. Checking outside beams:

$$M = M_{DL} + M_{LL} = (1{,}207 + 3{,}584) + 25{,}500 = 30{,}291 \text{ ft-lb}$$

$$f_b = \frac{M}{S} = \frac{30{,}291(12 \text{ in/ft})}{280.73} = 1{,}295 \text{ lb/in}^2$$

f_b = 1,295 lb/in^2 < Fb' = 1,296 lb/in^2, so outside beams are satisfactory in bending.

The beams must next be checked for lateral stability. Transverse bracing (blocking) will be provided at the beam ends and the span centerline:

$$\ell_u = \frac{L}{4} = \frac{17}{2} = 8.5 \text{ ft} \quad \text{and} \quad \frac{\ell_u}{d} = \frac{8.5(12 \text{ in/ft})}{17.5} = 5.83$$

By Equation 5-7,

$$\ell_e = 1.63\ell_u + 3d = 1.63(8.5)(12 \text{ in/ft}) + 3(17.5) = 218.76$$

By Equation 5-3,

$$C_s = \sqrt{\frac{\ell_e d}{b^2}} = \sqrt{\frac{218.76(17.5)}{(5.5)^2}} = 11.25 < 50$$

C_s > 10, so further stability calculations are required:

$$E' = EC_M = 1{,}600{,}000(1.0) = 1{,}600{,}000 \text{ lb/in}^2$$

By Equation 5-9,

$$F_b'' = F_b C_M = 1{,}350(1.0) = 1{,}350 \text{ lb/in}^2$$

$$C_k = 0.811\sqrt{\frac{E'}{F_b''}} = 0.811\sqrt{\frac{1{,}600{,}000}{1{,}350}} = 27.92$$

C_s = 11.25 < C_k = 27.92, so the beam is in the intermediate slenderness range. By Equation 5-10,

$$C_L = 1 - \frac{1}{3}\left(\frac{C_s}{C_k}\right)^4 = 1 - \frac{1}{3}\left(\frac{11.25}{27.92}\right)^4 = 0.99$$

C_L = 0.99 > C_F = 0.96; therefore, strength rather than stability controls allowable bending stress.

Check Live Load Deflection

Live load deflection is checked by assuming deflection is distributed the same as bending; one beam resists the deflection produced by 0.50 wheel lines. From Table 16-8, the deflection coefficient for one wheel line of an HS 15-44 truck on a 17-foot simple span is 2.12×10^9 lb-in^3.

$$\Delta_{LL} = \frac{0.50\left(2.12\times10^9\right)}{E'I} = \frac{0.50\left(2.12\times10^9\right)}{(1{,}600{,}000)(2{,}456.38)} = 0.27 \text{ in.} = L/756$$

L/756 < *L/360*, so deflection is acceptable.

Check Horizontal Shear

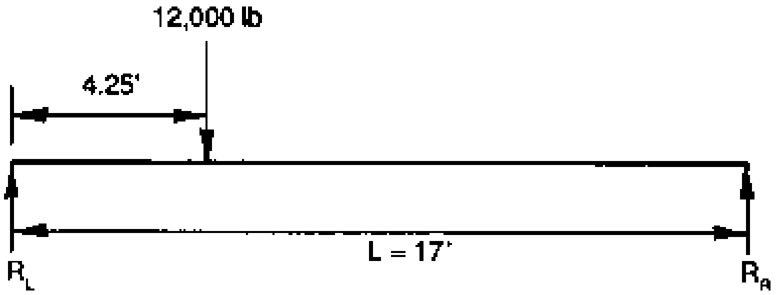
From bending calculations, outside beam dead load is 99.2 lb/ft for the deck and railing and 33.4 lb/ft for the beam, for a total of 132.6 lb/ft. Neglecting loads within a distance of d = 17.5 inches from the supports, dead load vertical shear is computed by Equation 7-6:

$$V_{DL} = w_{DL}\left(\frac{L}{2} - d\right) = 132.6\left(\frac{17}{2} - \frac{17.5}{12 \text{ in/ft}}\right) = 934 \text{ lb}$$

Live load vertical shear is computed at the lesser of *3d* or *L/4* from the support:

$$3d = \frac{3(17.5)}{12 \text{ in/ft}} = 4.38 \text{ ft} \qquad \frac{L}{4} = \frac{(17)}{4} = 4.25 \text{ ft}$$

L/4 = 4.25 feet controls, and the maximum vertical shear is determined at that location for one wheel line of an HS 15-44 truck:



$$V_{LU} = R_L = \frac{12{,}000 \text{ lb}\left(17 \text{ ft} - 4.25 \text{ ft}\right)}{17 \text{ ft}} = 9{,}000 \text{ lb}$$

For a moment DF to outside beams of 0.50,

$$V_{LD} = 9{,}000(0.50) = 4{,}500 \text{ lb}$$

By Equation 7-1,

$$V_{LL} = 0.50\ [(0.6V_{LU}) + V_{LD}]$$

$$= 0.50\ [(0.6)(9{,}000) + 4{,}500] = 4{,}950 \text{ lb}$$

$$V = V_{DL} + V_{LL} = 934 + 4{,}950 = 5{,}884 \text{ lb}$$

$$f_v = \frac{1.5\,V}{A} = \frac{1.5\,(5{,}884)}{96.25} = 92 \text{ lb/in}^2$$

$F_v' = F_v(C_M)$ (shear stress modification factor)

Without the shear stress modification factor,

$$F_v' = F_v(C_M) = 85(1.0) = 85 \text{ lb/in}^2$$

Without an increase in allowable stress by the shear stress modification factor (Table 7-17), the beam is overstressed by approximately 7 lb/in^2. It is reasonable to assume that some splitting of the beam may occur as it seasons; however, a full-length split assumed by no stress increase is unlikely. A slight increase in allowable stress of approximately 10 percent is considered appropriate in this case. This is a matter of designer judgment that must be specifically addressed in each case.

$$F_v' = 85(1.0)(1.10) = 94 \text{ lb/in}^2$$

$f_v = 92$ lb/in^2 $< F_v' = 94$ lb/in^2, so the beam is acceptable in horizontal shear.

Determine Bearing Length and Stress

From Table 5-7, $C_M = 0.67$, and

$$F_{c\perp}' = F_{c\perp}(C_M) = 625(0.67) = 419 \text{ lb/in}^2$$

For a unit dead load $w_{DL} = 132.6$ lb/ft to outside beams,

$$R_{DL} = \frac{w_{DL}\,L}{8} = \frac{(132.6)\,(17)}{8} = 1{,}127 \text{ lb}$$

The live load reaction DF is determined as the reaction at the beam, assuming the deck acts as a simple span between supports. For a 24-inch beam spacing, the maximum reaction is 1.0 WL/beam. From Table 16-8, the maximum reaction for one wheel line of an HS 15-44 truck on a 17-foot span is 14.12 k = 14,120 lb:

$$R_{LL} = R(\text{DF}) = 14{,}120(1.0) = 14{,}120 \text{ lb}$$

By Equation 7-8,

$$\text{Required bearing length} = \frac{R_{DL} + R_{LL}}{b\left(F_{c\perp}'\right)} = \frac{1{,}127 + 14{,}120}{5.5\,(419)} = 6.6 \text{ in.}$$

A bearing length of 7 inches will be used, for an out-to-out beam length of 17 feet 7 inches. Applied stress is computed by Equation 7-9:

$$f_{c\perp} = \frac{R_{DL} + R_{LL}}{A} = \frac{1{,}127 + 14{,}120}{5.5\,(7)} = 396 \text{ lb/in}^2$$

Summary

The superstructure will consist of thirteen 6- by 18-inch dressed lumber beams spaced 24 inches on center. The beams will be 17 feet 7 inches long and span a distance of 17 feet measured center to center of bearings. Transverse blocking will be provided for lateral support at the bearings and at the span centerline. Lumber will be specified as Douglas Fir-Larch in the B & S size classification, visually graded No. 1 or better to WWPA rules. Stresses and deflection are as follows:

	Interior beams	**Outside beams**
f_b	1,225 lb/in^2	1,295 lb/in^2
F_b'	1,296 lb/in^2	1,296 lb/in^2
Δ_{LL}	0.27 in. = *L/756*	0.27 in. = *L/756*
f_v	< Outside beam	92 lb/in^2
F_v'	94 lb/in^2	94 lb/in^2
$f_{c\perp}$	< Outside beam	396 lb/in^2
$F_{c\perp}'$	419 lb/in^2	419 lb/in^2

Example 7-10 - Lumber beam design; single-lane H 10-44 loading

A farmer wants to construct a bridge over a small creek to access additional acreage. Based on a study of the site, an 11-foot span, measured center-to-center of bearings, will be adequate. The bridge must be capable of supporting farming equipment that closely resembles an AASHTO H 10-44 truck. The required roadway width is approximately 10-1/2 feet with 6- by 6-inch curbs installed along each edge. Design the beam system for this structure, assuming

1. the beams and curbs are full-sawn Douglas Fir-Larch;

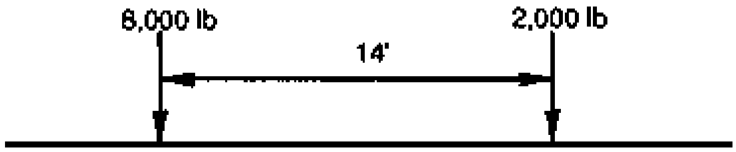
2. the transverse timber deck is constructed of surfaced 4-inch planks, with no wearing surface;

3. beam spacing is limited by deck span capabilities to approximately 14 inches; and

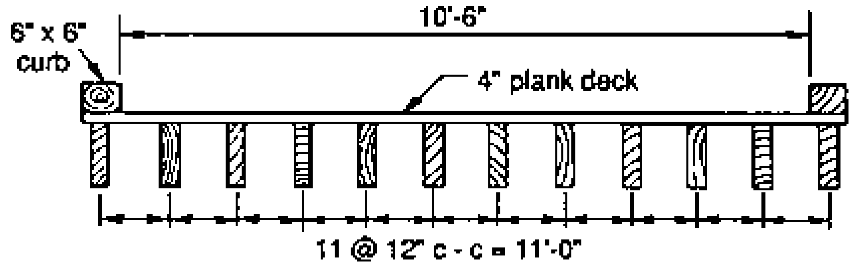
4. live load deflection and AASHTO Load Group IA loading need not be considered.

Solution

For an AASHTO H 10-44 truck the GVW is 10 tons distributed 20 percent to the front axle and 80 percent to the rear axle (Example 6-1). The vehicle configuration for one wheel line is as follows:



Because this bridge spans a short crossing, it is anticipated that shear will control beam design. The design procedure will be to size the beams based on horizontal shear, then check for bending. An initial configuration of 11 beams spaced 12 inches on center is selected:



Compute Deck Dead Load

Interior beams support 1 foot of deck width. Outside beams support a more severe loading from a 9-inch deck width plus the 6- by 6-inch curb:

$$\text{Deck } w_{DL} = \left(\frac{(3.5 \text{ in.})(9 \text{ in.}) + (6 \text{ in.})(6 \text{ in.})}{144 \text{ in}^2/\text{ft}^2}\right)(50 \text{ lb/ft}^2) = 23.4 \text{ lb/ft}$$

Compute Live Load Distribution Factors

Live load distribution for shear is based on the distribution factors used for moment. Assuming the deck acts as a simple span between beams, placing the wheel line 2 feet from the face of the curb results in no live load distribution to outside beams. Therefore, the moment DF for interior and outside beams will be controlled by interior beams. From Table 7-13 for a single-lane plank deck,

$$\text{Moment DF} = \frac{S}{4} = \frac{1}{4} = 0.025 \text{ WL/beam}$$

Determine Beam Size Based on Horizontal Shear

From NDS Table 4A for visually graded Douglas Fir-Larch, there are two tabulated shear values given for different size classifications. For all grades in the J&P size classification (lumber 2 to 4 inches thick), F_v = 95 lb/in^2. For all grades in the B&S size classification, F_v = 85 lb/in^2. The smaller 4-inch material is selected as a first choice.

Starting with a 4- by 12-inch full-sawn beam, section properties required for shear are computed:

b = 4 in.

d = 12 in.

A = 4 in. (12 in.) = 48 in^2

$$w_{DL} = \frac{48 \text{ in}^2}{144 \text{ in}^2/\text{ft}^2}\left(50 \text{ lb/ft}^3\right) = 16.7 \text{ lb/ft}$$

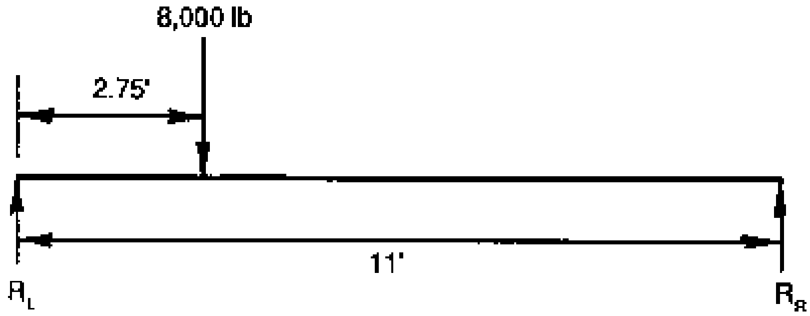
Dead load vertical shear is computed for combined deck and beam dead load by Equation 7-6:

$$V_{DL} = w_{DL} = \left(\frac{L}{2} - d\right) = (23.4 + 16.7)\left(\frac{11}{2} - \frac{12}{12 \text{ in./ft}}\right) = 180.5 \text{ lb}$$

Live load vertical shear is computed from the maximum vertical shear occurring at the lesser of *3d* or *L/4* from the support:

$$3d = 3\left(\frac{12}{12 \text{ in./ft}}\right) = 3 \text{ ft} \qquad \frac{L}{4} = \frac{11}{4} = 2.75 \text{ ft}$$

L/4 = 2.75 feet controls, and the maximum vertical shear is determined at that point for one wheel line of an H 10-44 truck:



$$V_{LU} = R_L = \frac{(8{,}000 \text{ lb})(11 \text{ ft} - 2.75 \text{ ft})}{11 \text{ ft}} = 6{,}000 \text{ lb}$$

For a moment DF to outside beams of 0.25,

$$V_{LD} = 6{,}000(0.25) = 1{,}500 \text{ lb}$$

$$V_{LL} = 0.50\,[(0.6V_{LU}) + V_{LD}] = 0.50\,[(0.6)(6{,}000) + 1{,}500] = 2{,}550 \text{ lb}$$

$$V = V_{DL} + V_{LL} = 180.5 + 2{,}550 = 2{,}731 \text{ lb}$$

By Equation 5-18,

$F_v' = F_v(C_M)$ (shear stress modification factor)

From Table 5-7, $C_M = 0.97$ for wet-condition use. Because it is likely that some beam splitting may occur as the material seasons, the shear stress modification factor (Table 7-17) will be limited to 1.0 based on designer judgment.

$F_v' = (95 \text{ lb/in}^2)\,(0.97)(1.0) = 92 \text{ lb/in}^2$

Rearranging Equation 5-17, the required beam area is computed:

$$A = \frac{1.5V}{F_v} = 1.5\,\frac{(2{,}731)}{92.15} = 44.45 \text{ in}^2$$

44.45 $\text{in}^2 < 48 \text{ in}^2$, so a 4- by 12-inch beam is satisfactory with the following applied stress:

$$f_v = \frac{1.5V}{A} = \frac{1.5(2{,}731)}{48 \text{ in}^2} = 85 \text{ lb/in}^2$$

Check Bending and Select Beam Grade

For a 4- by 12-inch full-sawn beam,

$$S = \frac{bd^2}{6} = \frac{4\,(12)^2}{6} = 96 \text{ in}^3$$

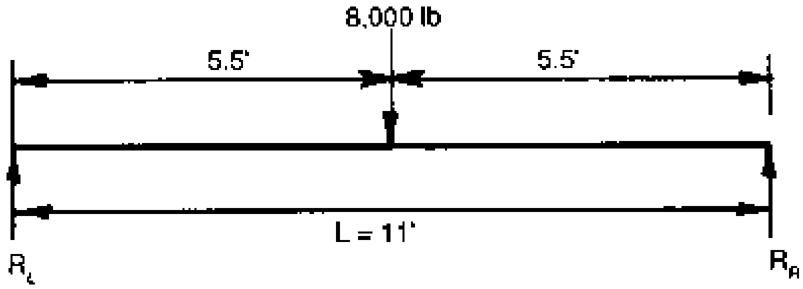
$$I = \frac{bd^3}{12} = \frac{4\,(12)^3}{12} = 576 \text{ in}^4$$

$$\text{Beam } M_{DL} = \frac{w_{DL}L^2}{8} = \frac{(16.7 \text{ ft})(11 \text{ ft})^2}{8} = 252.6 \text{ ft-lb}$$

$$\text{Deck } M_{DL} = \frac{(23.4 \text{ ft})(11 \text{ ft})^2}{8} = 353.9 \text{ ft-lb}$$

Total M_{DL} = 252.6 + 353.9 = 606.5 ft-lb

For an H 10-44 truck on an 11-foot span, maximum live load moment occurs when the 8,000-pound wheel load is positioned at the span centerline:



$$M_{LL}/\text{WL} = R_L \frac{L}{2} = 4{,}000 \frac{11}{2} = 22{,}000 \text{ ft-lb}$$

Applying the moment DF = 0.25, applied bending stress is computed:

M_{LL} = 0.25(22,000 ft-lb) = 5,500 ft-lb

M = $M_{DL} + M_{LL}$ = 606.5 + 5,500 = 6,107 ft-lb

$$f_b = \frac{M}{S} = \frac{6{,}107\ (12 \text{ in/ft})}{96} = 763 \text{ lb/in}^2$$

From NDS Table 4A, No, 2 Douglas Fir-Larch is selected with the following tabulated values:

F_b = 1,250 lb/in^2 $\quad C_M$ = 0.86

F_v = 95 lb/in^2 $\quad C_M$ = 0.97

$F_{c\perp}$ = 625 lb/in^2 $\quad C_M$ = 0.67

E = 1,700,000 lb/in^2 $\quad C_M$ = 0.97

$$F_b' = F_b C_M C_F = 1250(0.86)(1.0) = 1{,}075 \text{ lb/in}^2$$

f_b = 763 lb/in^2 < F_b' = 1,075 lb/in^2, so the beam is satisfactory in bending. The beam is next checked for lateral stability. Because of the very short span, transverse bracing (blocking) will be provided at the beam ends only:

$$\ell_u = L = 11 \text{ ft} \qquad \text{and} \qquad \frac{\ell_u}{d} = \frac{11(12\text{in/ft})}{12} = 11$$

By Equation 5-7,

$$\ell_e = 1.63\ell_u + 3d = 1.63(11)(12 \text{ in./ft}) + 3(12) = 251.16$$

By Equation (5-3),

$$C_s = \sqrt{\frac{\ell_e d}{b^2}} = \sqrt{\frac{251.16(12)}{(4)^2}} = 13.72 < 50$$

$$E' = EC_M = 1{,}700{,}000(0.97) = 1649{,}000 \text{ lb/in}^2$$

By Equation 5-9,

$$F_b'' = F_b C_M = 1{,}250\ (0.86) = 1{,}075 \text{ lb/in}^2$$

$$C_k\_0.811\sqrt{\frac{E'}{F_b''}} = 0.811\sqrt{\frac{1{,}649{,}000}{1{,}075}} = 31.76$$

$C_s = 13.72 < C_k = 31.76$; therefore the beam is in the intermediate slenderness range. By Equation 5-10,

$$C_L = 1 - \frac{1}{3}\left(\frac{C_s}{C_k}\right)^4 = 1 - \frac{1}{3}\left(\frac{13.72}{31.76}\right)^4 = 0.99$$

$C_L = 0.99 < C_F = 1.0$, so stability controls over strength and allowable bending stress must be adjusted by C_L:

$$F_b' = F_b C_M C_L = 1250(0.86)(0.99) = 1{,}064 \text{ lb/in}^2$$

$F_b' = 1{,}064 \text{ lb/in}^2 > f_b = 763 \text{ lb/in}^2$, so the 4- by 12-inch No. 2 beams are satisfactory.

Determine Bearing Length and Stresses

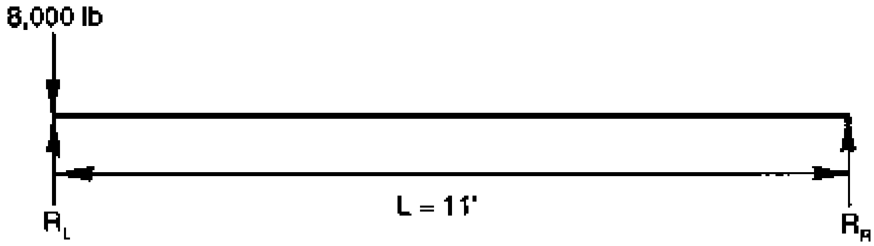
Allowable stress in compression perpendicular to grain is computed by Equation 5-20:

$$F_{c\perp}' = F_{c\perp}\ (C_M) = 625(0.67) = 419 \text{ lb/in}^2$$

For a unit dead load of 23.4 lb/ft for the deck and 16.7 lb/ft for the beams,

$$R_{DL} = \frac{w_{DL}L}{2} = \frac{(23.4 + 16.7)(11)}{2} = 220.6 \text{ lb}$$

Assuming the deck acts as a simple span over the 12-inch beam spacing, the reaction DF is 1.0 WL/beam. The reaction for one wheel line of an H 10-44 truck is computed and multiplied by the reaction DF:



$$R_L = 8{,}000 \text{ lb}$$

$$R_{LL} = R(DF) = 8{,}000(1.0) = 8{,}000 \text{ lb}$$

By Equation 7-8,

$$\text{Required bearing length} = \frac{R_{DL} + R_{LL}}{b\left(F_{c\perp}'\right)} = \frac{220.6 + 8{,}000}{4\,(419)} = 4.9 \text{ in.}$$

A bearing length of 6 inches will be used, for an out-to-out beam length of 11 feet 6 inches Applied stress is computed by Equation 7-9:

$$f_{c\perp} = \frac{R_{DL} + R_{LL}}{A} = \frac{220.6 + 8{,}000}{4(6)} = 343 \text{ lb/in}^2$$

Summary

The superstructure will consist of twelve 4- by 12-inch full-sawn lumber beams, 11 feet 6 inches long, spaced 12 inches on center. Transverse blocking will be provided for lateral support at the bearings. Stresses based on No. 2 Douglas Fir-Larch in the J&P size classification are as follows:

	Interior beams	**Outside beams**
f_b	< Outside beams	763 lb/in^2
F_b'	1,075 lb/in^2	1,075 lb/in^2
f_v	< Outside beams	85 lb/in^2
F_v'	92 lb/in^2	92 lb/in^2
$f_{c\perp}$	< Outside beams	343 lb/in^2
$F_{c\perp}'$	419 lb/in^2	419 lb/in^2

Design of Transverse Bracing

Transverse bracing for sawn lumber beams is normally provided by lumber blocks placed between the beams (Figure 7-31). Blocks should be positioned as close as practical to the beam top and preferably extend the entire beam depth. They are generally 4 inches thick for beams up to 12 inches wide, and 6 inches thick for wider beams. As a minimum, blocks should be placed at both bearings, and at centerspan for span lengths over 20 feet.

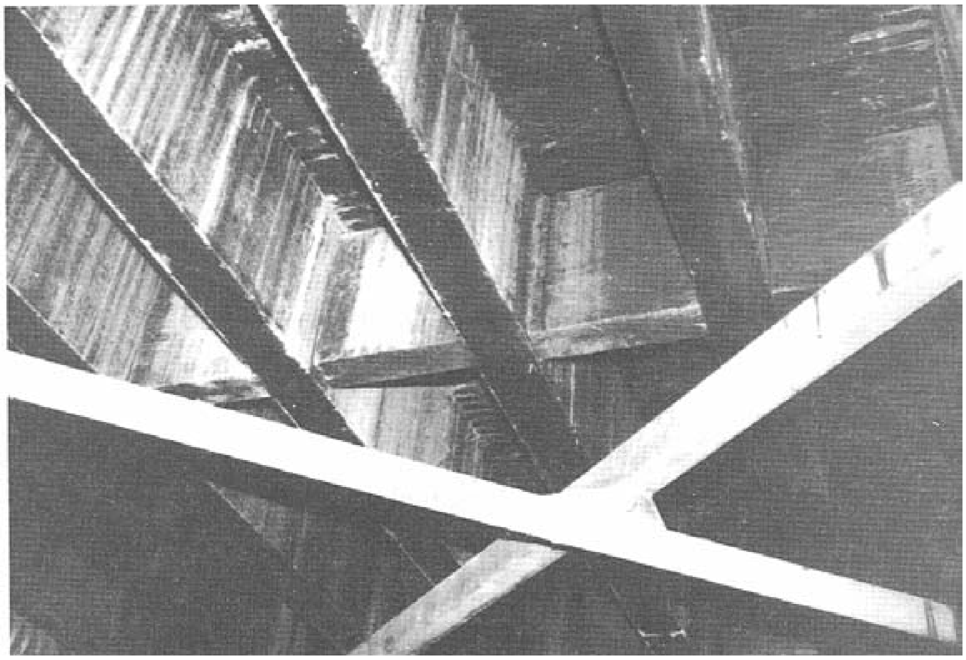


Figure 7-31. - Lumber blocks placed as transverse bracing for sawn lumber beams.

An examination of existing lumber beam bridges will show that the number of different block attachments has been limited only by designer imagination. Two of the most common attachments used in recent years are steel brackets attached to the beam sides and rods placed through the beams. The simplest brackets are prefabricated steel joist or beam hangers commonly used in building construction (Figure 7-32). These hangers, which are nailed or spiked to the beams and blocks, are available in a variety of standard sizes for members up to 6 inches wide and 16 inches deep. They are relatively inexpensive, simple to install, and provide adequate performance. For the rod configuration, a 3/4-inch-diameter steel rod is placed continuously through all beams across the structure width (Figure 7-33). Lumber blocks are then toenailed to adjacent beams and connected to the rod with 3/16-inch driven staples. This system provides the added advantage of tying all beams together, but it requires additional fabrication and materials and is normally more difficult to erect than other systems.

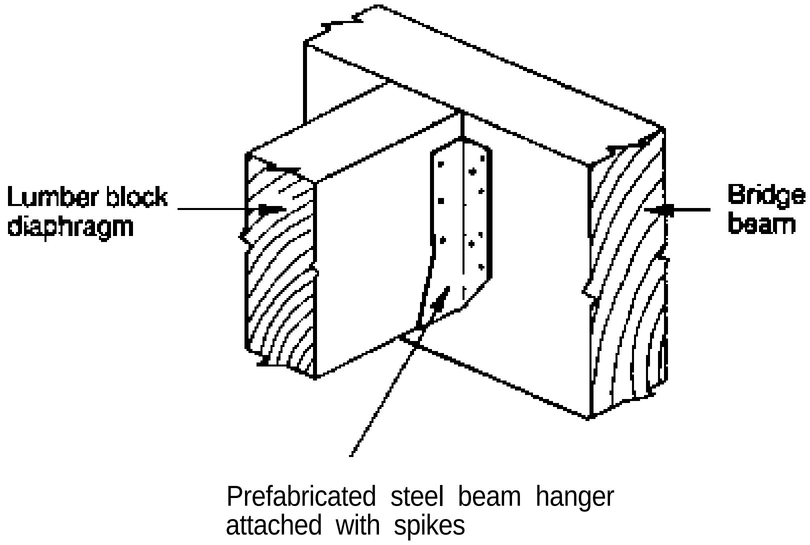


Figure 7-32. - Lumber block diaphragm configuration using steel beam hangers.

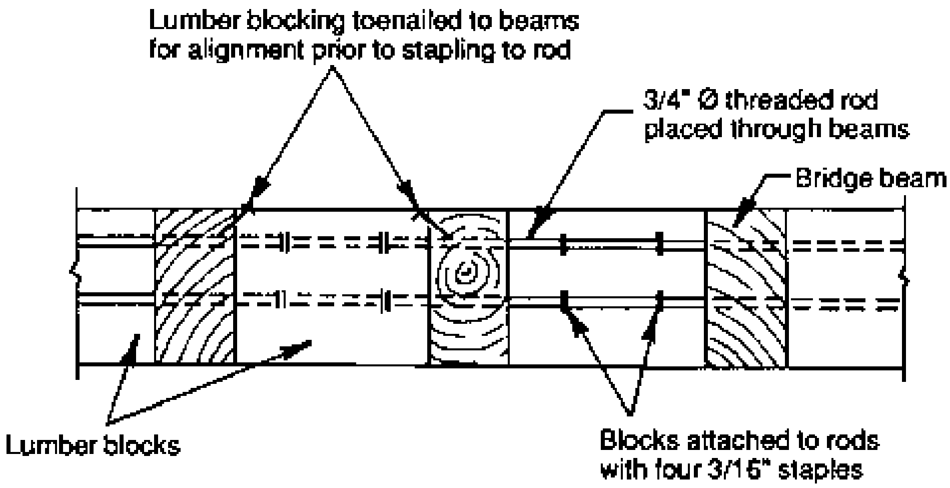


Figure 7-33. - Lumber block diaphragm configuration using steel rods and driven staples.

Design of Bearings

Bearings for sawn lumber beams must provide sufficient area for compression and must be able to transfer longitudinal and transverse loads from the superstructure to the substructure. The design considerations for glulam beams also apply to lumber beams, although some details are often modified because of the smaller beam size. The most suitable bearing is generally the steel bearing shoe arrangement. For sawn lumber applications, the shoe is constructed of standard steel angles with one beam attachment bolt and two anchor bolts, one for each angle (Figure 7-34).

Because of the smaller beam sizes, the base plate and bearing pad used for glulam are normally not required for sawn lumber beams, but may be provided at the option of the designer.

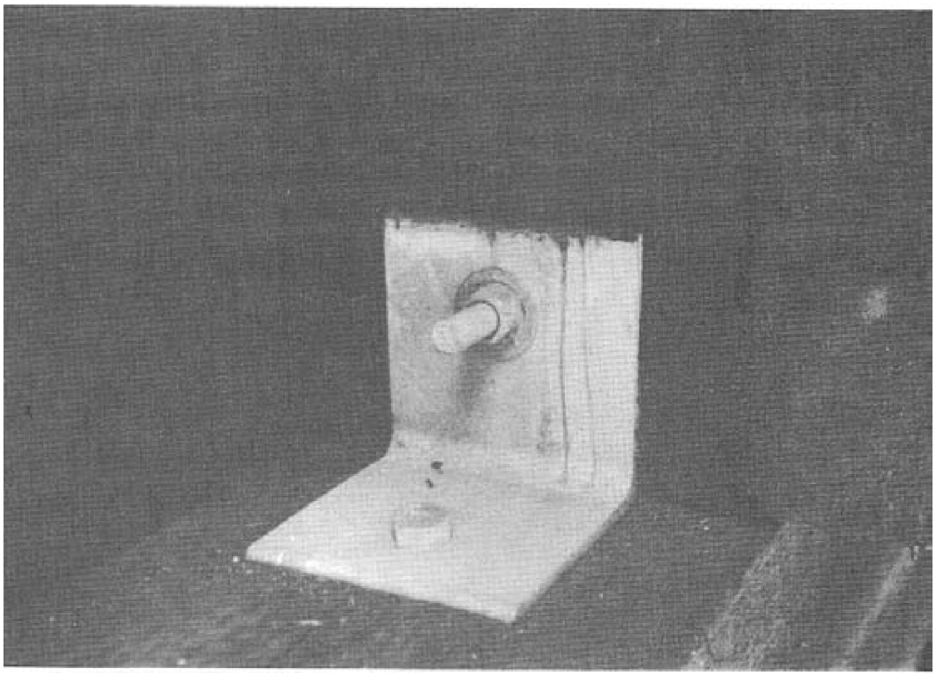


Figure 7-34. - Steel angle bearing attachment for sawn lumber beams.

When bearing is on a timber cap or sill, it has been common practice in the past to anchor each beam directly to the support with a 1/2- to 3/4-inch steel drift pin placed through the beam center. Although this type of attachment is satisfactory from a structural standpoint, it can significantly increase the decay hazard if good fabrication and construction practices are not followed. When drift pins are used, lead holes in the beams and cap should be bored before the members are pressure-treated with preservatives. When this is not practical, field-bored holes must be thoroughly treated with preservatives before placing the pin (Chapter 12).

7.8 NAIL-LAMINATED DECKS

Transverse nail-laminated decks consist of a series of dimension lumber laminations placed on edge and nailed together on their wide faces (Figure 7-35). The deck is constructed by progressively nailing laminations to the preceding section to form a continuous surface over the bridge length. Nail-laminated decks are similar in arrangement to glulam, but load transfer between laminations is done mechanically by nails rather than by glue. The laminations are generally nominal 2 by 4 or 2 by 6 sawn

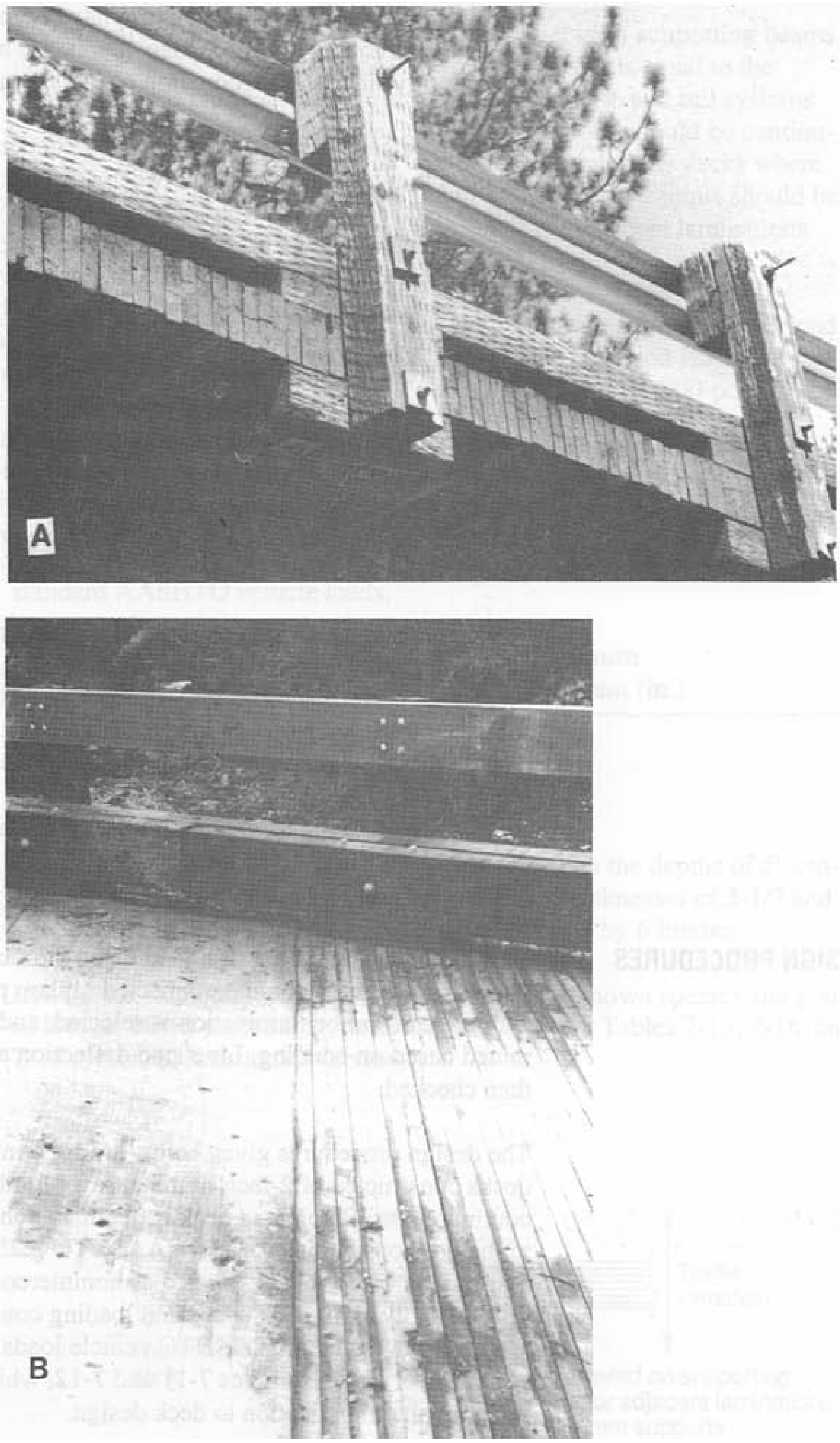


Figure 7-35.—(A) Edge view of a transverse nail-laminated lumber deck. (B) Top view comparison of a nail-laminated lumber deck (right) and glulam deck (left).

lumber for spans up to approximately 6 feet under standard AASHTO highway loads. Nail-laminated decks have been widely used on timber and steel superstructures for more than 40 years. Their popularity has declined significantly since the introduction of glulam panels.

The performance of nail-laminated decks depends on the effectiveness of the nails in transferring loads between adjacent laminations. Loose nails lead to reduced load distribution and increased deck deflection. This typically causes laminations to separate and asphalt paving to deteriorate. Although the static strength of a loose deck may remain high, deck serviceability under dynamic vehicle loads is greatly reduced. Looseness is normally caused by two factors, high deck deflections and dimensional changes from moisture variations. Deflections can be controlled in design, but have frequently been neglected in the past. Moisture effects have a somewhat lesser effect that deflection and depend on local environmental conditions and the degree of exposure to weathering. Dimensional stability of nail-laminated decks is improved when seasoned, edge-grain lumber is used and the deck is protected by a watertight wearing surface (Chapter 11).

Nail-laminated decks are economical and are easily constructed with locally available materials. When properly designed, they provide acceptable performance on low- to moderate-volume bridges that are not subjected to heavy highway loads. They do not provide a service life comparable to properly designed glulam panels because the nails penetrate the preservative layer of the wood, making it more susceptible to decay. In areas where de-icing chemicals are used, the chemicals may also corrode the nails over time.

DESIGN PROCEDURES

Nail-laminated decks are designed using the same basic procedures previously discussed for noninterconnected glulam panels. An initial species and grade of lumber lamination is selected, and deck thickness is determined based on bending. Live load deflection and horizontal shear are then checked.

The design procedures given below are for continuous nail-laminated decks constructed of 2-inch nominal sawn lumber, 4 to 6 inches deep. A continuous nail-laminated deck is one in which all laminations are nailed to the previous laminations (see AASHTO 3.25.1.1 for design criteria for nail-laminated decks constructed as noninterconnected panels). The criteria apply to all deck spans and loading conditions, but design aids are limited to standard AASHTO vehicle loads on effective deck spans of 72 inches or less. Examples 7-11 and 7-12, which follow the procedures, illustrate their application to deck design.

1. Define deck span, configuration, and design loads.

The effective deck span s is the clear distance between supporting beams plus one-half the width of one beam. The deck width is equal to the roadway width plus additional width required for curb and rail systems (Chapter 10). Whenever possible, lumber laminations should be continuous (one piece) for the entire deck width. On multiple-lane decks where sawn lumber is not available in the required lengths, butt joints should be placed at the center of the support, with joints for adjacent laminations staggered on different supports (Figure 7-36).

The design live load on nail-laminated decks is the maximum wheel load of the design vehicle. For standard AASHTO H 20-44 and HS 20-44 loads, special provisions for timber decks apply and a 12,000-pound wheel load is used for all four standard AASHTO truck loads.

2. Estimate deck thickness.

Deck thickness must be estimated for initial calculations. The following values provide a reasonable estimate of the maximum deck span for standard AASHTO vehicle loads.

Initial deck thickness (in.)	Maximum effective span (in.)
3-1/2	30
4	38
5-1/2	67
6	72

Deck thicknesses of 3-1/2 and 4 inches are based on the depths of dimension and full-sawn 2 by 4 lumber, respectively. Thicknesses of 5-1/2 and 6 inches are based on the same relative depths for 2 by 6 lumber.

Initial deck thickness may also be estimated for a known species and grade of lumber based on bending, deflection, or shear by Tables 7-15, 7-16, and 7-18 presented later in this section.

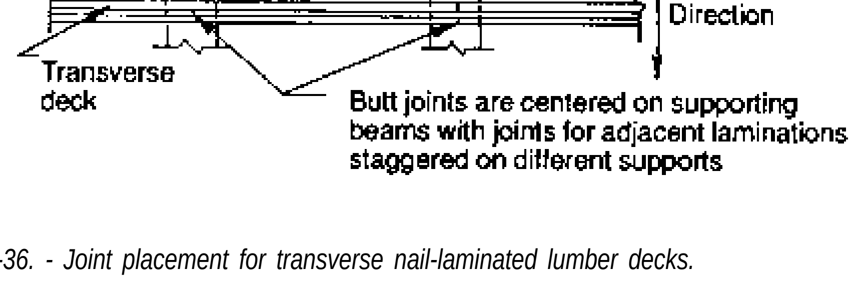


Figure 7-36. - Joint placement for transverse nail-laminated lumber decks.

3. Determine wheel distribution widths and effective deck section properties.

In the direction of the deck span, the wheel load, P, is assumed to be a uniformly distributed load acting over a width, b_t (AASHTO 3.25.1):

$$b_t = \sqrt{0.025P} \tag{7-42}$$

For a 12,000-pound wheel load, b_t = 17.32 inches.

In the direction normal to the deck span, the wheel load distribution width, b_d, is equal to 15 inches plus the deck thickness, t (AASHTO 3.25.1.1), as computed by

$$b_d = 15 + t \tag{7-43}$$

The deck is designed as a beam of width b_d and depth t. Effective section properties are computed by the same equations used for noninterconnected glulam decks, and are given in Table 7-14 for nominal 2 by 4 and 2 by 6 sawn lumber decks.

Table 7-14. - Effective deck section properties for continuous transverse nail-laminated decks.

t (in.)	b_d (in.)	A (in^2)	S (in^3)	I (in^4)
3-1/2	18.5	64.75	37.77	66.10
4	19.0	76.00	50.67	101.33
5-1/2	20.5	112.75	103.35	284.22
6	21.0	126.00	126.00	378.00

4. Compute dead load, dead load moment, and live load moment.

Deck dead load, dead load moment, and live load moment are computed in the same manner as for noninterconnected glulam decks. The uniform dead load moment for the effective deck section is determined by assuming the deck acts as a simple span between supports. Live load moment is computed by positioning the vehicle wheel load on the span to produce the maximum moment.

For a standard 12,000-pound wheel load and 6-foot-track width, the maximum live load moment on effective deck spans greater than 17.32 inches, but less than or equal to 122 inches ($17.32 < s < 122$), is given by

$$M_{LL} = 3{,}000s - 25{,}983 \tag{7-44}$$

where M_{LL} is the maximum live load moment (in-lb).

5. Compute bending stress and select a lamination species and grade. For decks continuous over two spans or less, bending stress is based on the simple span moment, computed by

$$f_b = \frac{M}{S} \tag{7-45}$$

where $M = M_{DL} + M_{LL}$ computed for a simple span (in-lb) and

S = section modulus of the effective deck section (in^3).

For decks continuous over more than two spans, bending stress is based on 80 percent of simple span moment to account for deck continuity and is computed by

$$f_b = \frac{0.8M}{S} \tag{7-46}$$

After f_b is computed, a species and grade of sawn lumber is selected based on the size classification for the estimated deck thickness. Allowable bending stress is computed by adjusting the tabulated stress by all applicable modification factors (for nail-laminated decks, the tabulated bending stress listed in the NDS Table 4A for repetitive member use may be used):

$$F_b' = F_b C_M \tag{7-47}$$

The allowable stress computed by Equation 7-47 may be increased by a factor of 1.33 for overloads in AASHTO Load Group IB.

If $f_b \leq F_b'$, the lamination size, species, and grade are satisfactory in bending. If f_b is substantially lower than F_b', it may be more economical to select a lower-grade material or reduce the deck thickness.

If $f_b > F_b'$, the lamination is insufficient in bending and the grade of sawn lumber or the deck thickness must be increased. If the thickness is increased, revise calculations starting at step 2.

Table 7-15 gives approximate maximum spans based on bending for nail-laminated decks continuous over more than two spans.

Table 7-15. - Approximate maximum effective span for continuous transverse nail-laminated decks based on bending; deck continuous across over more than two spans; loading from a 12,000-pound wheel load plus the deck dead load; b_d = 15 inches + deck thickness.

		Maximum deck span (in.)			
F_b (lb/in^2)	F_b' (lb/in^2)	t = 3-1/2 in.	t = 4 in.	t = 5-1/2 in.	t = 6 in.
1,100	946	23	29	49	58
1,150	989	24	29	51	60
1,200	1,032	25	30	53	62
1,250	1,075	26	31	54	64
1,300	1,118	26	32	56	66
1,350	1,161	27	33	58	69
1,400	1,204	28	34	60	71
1,450	1,247	28	35	62	73
1,500	1,290	29	36	64	75
1,550	1,333	30	37	65	77
1,600	1,376	30	38	67	80
1,650	1,419	31	38	69	82
1,700	1,462	32	39	71	84
1,750	1,505	32	40	73	86
1,800	1,548	33	41	74	88
1,850	1,591	34	42	76	91
1,900	1,634	34	43	78	93
1,950	1,677	35	44	80	95
2,000	1,720	36	45	82	97

$F_b' = F_b C_M = F_b(0.86)$.

6. Check live load deflection.

Live load deck deflection is computed by the standard methods of engineering analysis, assuming the deck behaves elastically as a simple beam between supports. The maximum deflection for a standard 12,000-pound wheel load on deck spans greater than 17.32 inches, but less than 110 inches, is given by

$$\Delta_{LL} = \frac{1.80}{E'I}\left(138.8s^3 - 20{,}780s + 90{,}000\right) \tag{7-48}$$

where I is the effective moment of inertia of the effective deck section of width b_d and depth t.

When the deck is continuous over more than two spans, the deflection computed by Equation 7-48 may be multiplied by 0.80 to account for span continuity. Deflection coefficients for decks that are continuous over more than two spans are given in Figure 7-37.

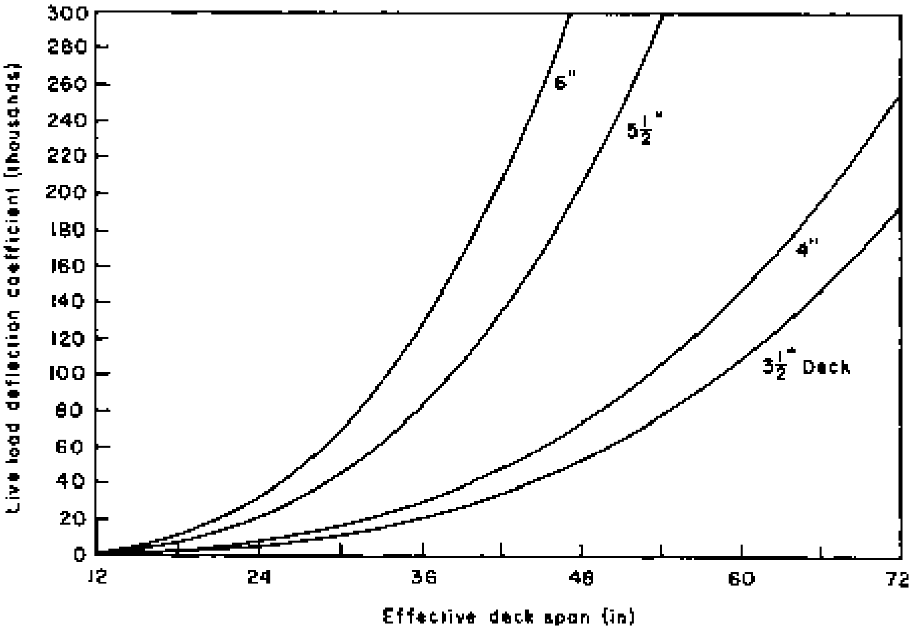


Figure 7-37. - Vehicle live load deflection coefficients for 12,000-pound wheel load(s) on a continuous, transverse nail-laminated lumber deck that is continuous over more than two spans. Divide the deflection coefficient by E' to obtain the deck deflection in inches.

Deflection is an important consideration in nail-laminated deck design and must be limited to ensure deck and wearing surface performance. The maximum acceptable deflection should be based on the type and volume of traffic and the type of wearing surface. The maximum recommended deflection is *s/500,* where *s* is the effective deck span. Based on this limit, maximum effective deck spans for a 12,000-pound wheel load are given in Table 7-16. When the computed live load deflection exceeds acceptable limits, the lumber grade must be increased to provide a higher *E* value, or the deck thickness must be increased.

7. Check horizontal shear.

Horizontal shear is based on the maximum vertical shear occurring at a distance from the support equal to the deck thickness, *t.* Dead load vertical shear, V_{DL}, is determined by

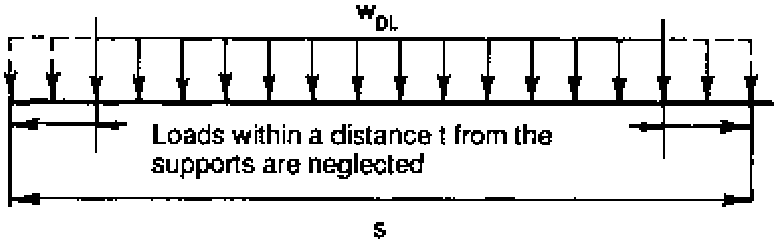


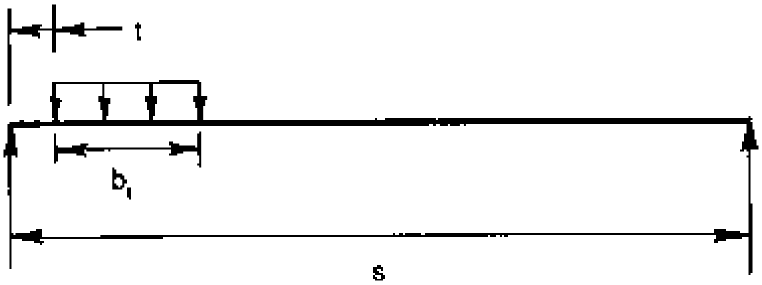
Table 7-16. - Approximate maximum effective span for continuous transverse nail-laminated decks based on a maximum vehicle live load deflection of *s/500;* deck continuous over more than two spans; loading from a 12,000-pound wheel load; b_d = 15 inches + deck thickness.

E (lb/in^2)	E' (lb/in^2)	Maximum deck span (in.)			
		t = 3-1/2 in.	t = 4 in.	t = 5-1/2 in.	t = 6 in.
1,500,000	1,455,000	33	40	62	>72
1,600,000	1,552,000	34	41	67	>72
1,700,000	1,649,000	35	42	69	>72
1,800,000	1,746,000	36	44	71	>72

$E' = EC_M = 0.97\,E$

$$V_{DL} = w_{DL}\left(\frac{s}{2} - t\right) \qquad (7\text{-}49)$$

Live load vertical shear is determined by placing the edge of the wheel load distribution width, b_t, a distance, t, from the support.



Applied stress in horizontal shear must be less than or equal to the allowable stress for the laminations, as computed by

$$f_v = \frac{1.5V}{A} \leq F_v' = F_v C_M \text{ (shear stress modification factor)} \qquad (7\text{-}50)$$

where $V = V_{DL} + V_{LL}$ (lb) and

A = area of effective deck section (in^2).

The shear stress modification factor given for sawn lumber in footnotes to the NDS Table 4A (Table 7-17) is generally taken as 2.0 for nail-laminated decks; however, the value should be based on designer judgment for the specific application and material.

Table 7-17. - Shear stress modification factor for sawn lumber.

Length of split on wide face of 2" lumber (nominal):	Multiply tabulated "F_v" value by:
No split	2.00
1/2 x wide face	1.67
3/4 x wide face	1.50
1 x wide face	1.33
1-1/2 x wide face or more	1.00

Length of split on wide face of 3" and thicker lumber (nominal):	Multiply tabulated "F_v" value by:
No split	2.00
1/2 x narrow faace	1.67
1 x narrow face	1.33
1-1/2 x narrow face or more	1.00

Size of shake[a] in 3" and thicker lumber (nominal):	Multiply tabulated "F_v" value by:
No shake	2.00
1/6 x narrow face	1.67
1/3 narrow face	1.33
1/2 x narrow face or more	1.00

[a] Shake is measured at the end between lines enclosing the shake and parallel to the wide face.

Specific horizontal shear values may be established by use of this table when the length of split, or size of check or shake, is known and no increase in them is anticipated. For California Redwood, Southern Pine, Virginia Pine-Pond Pine, and Yellow Poplar, refer to the NDS for specific values of F_v for which these adjustments apply.

From the NDS;[38] © 1986. Used by permission.

If $f_v > F_v'$, the deck does not have sufficient strength in horizontal shear and either F_v must be increased by selecting another grade or species of lamination or f_v must be reduced by increasing the deck thickness. For most species, tabulated values for horizontal shear do not increase substantially as grade increases, and increasing deck thickness is the only option. Maximum effective spans for continuous nail-laminated decks based on shear criteria are given in Table 7-18.

8. Check overhang.

The deck overhang at outside beams is checked for strength using an effective deck span measured to the centerline of the support, minus one-fourth of the beam width. For vehicle live load stresses, the wheel load is positioned with the load centroid 1 foot from the face of the railing or curb, as previously discussed for noninterconnected glulam decks. Deck stresses in bending and shear must be within allowable values previously computed.

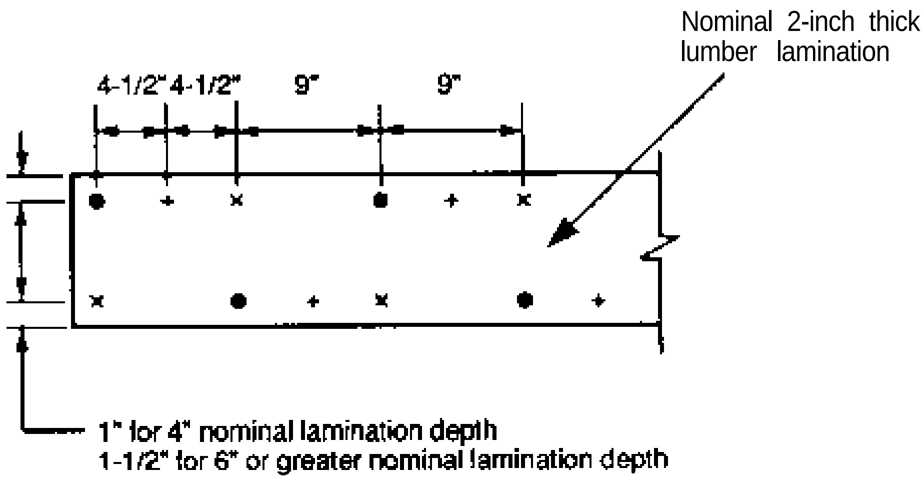
9. Determine nail size and placement pattern.

Laminations are nailed with galvanized common wire nails or threaded hardened-steel nails of sufficient length to penetrate 2.5 laminations. For 1-1/2-inch laminations, *20d* (4-inch) nails are used. For full-sawn 2-inch laminations, *40d* (5-inch) nails are sufficient. Nails are placed on approximately 9-inch centers near the top and bottom edges of the lamination.[22,60] The placement pattern is staggered over three successive laminations as shown in Figure 7-38.

Table 7-18. - Approximate maximum effective span for continuous transverse nail-laminated decks based on horizontal shear; loading from a 12,000-pound wheel load plus the deck dead load; b_d = 15 inches + deck thickness.

		Maximum deck span (in.)			
F_v (lb/in²)	F_v' (lb/in²)	t = 3-1/2 in.	t = 4 in.	t = 5-1/2 in.	t = 6 in.
100	194	40	68	>72	>72
95	185	36	57	>72	>72
90	175	32	48	>72	>72
85	165	30	41	>72	>72
80	155	27	36	>72	>72
75	146	25	33	>72	>72

$F_v' = F_v C_M$ (shear stress modification factor) = F_v(0.97)(2.0). The 2.0 shear stress modification factor assumes no splitting of the deck laminates across the wheel load distribution width, b_d.



● indicates nails in first lamination
x indicates nails in second lamination
+ indicates nails in third lamination

Figure 7-38. - Nail placement pattern for transverse nail-laminated lumber decks.

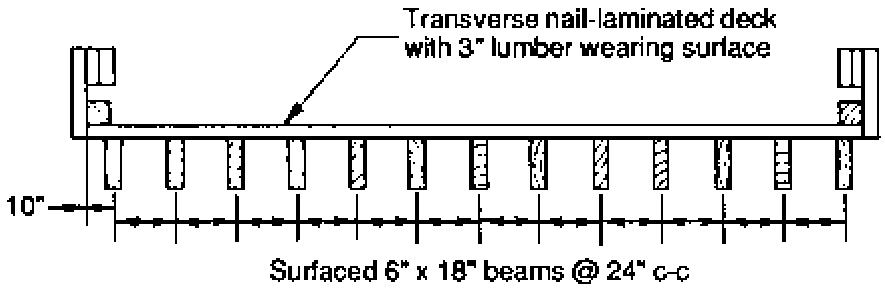
Example 7-11 - Nail-laminated lumber deck design; two-lane HS 15-44 loading

Design a transverse continuous nail-laminated lumber deck for the beam superstructure of Example 7-9. The superstructure has a two-lane, 24-foot roadway that carries AASHTO HS 15-44 loading. Support is provided by surfaced 6- by 18-inch lumber beams, spaced 24 inches on center. The out-to-out bridge span is 17 feet 7 inches. The following assumptions apply:

1. Deck laminations are visually graded Southern Pine.

2. The deck is provided with a full-width lumber wearing surface of full-sawn planks, 3 inches thick.

3. Deck live load deflection must be limited to *s/500.*



Solution

Define the Deck Span, Configuration, and Design Loads

The effective deck span is the clear distance between supporting beams plus one-half the width of one beam, but not greater than the clear span plus the deck thickness:

Clear distance between beams = 24 in. - 5.5 in. = 18.50 in.

$$s = 18.5 \text{ in.} + \frac{5.5 \text{ in.}}{2} = 21.25 \text{ in.}$$

The deck will be thicker than 2.75 inches, so s = 21.25 inches will control design.

For HS 15-44 loading the design load is one 12,000-pound wheel. Laminations will be continuous across the deck width in lengths of 25 feet 8 inches (25.67 feet).

Estimate Deck Thickness

An initial deck thickness of 4 inches (3.5 inches actual) is selected.

Determine Wheel Distribution Widths and Effective Deck Section Properties

In the direction of the deck span,

$$b_t = \sqrt{0.025P} = \sqrt{0.025\,(12{,}000)} = 17.32 \text{ in.}$$

Normal to the deck span,

$$b_d = 15 + t = 15 + 3.5 = 18.5 \text{ in.}$$

Effective deck section properties from Table 7-14 are

$$A = 64.75 \text{ in}^2$$

$$S = 37.77 \text{ in}^3$$

$$I = 66.10 \text{ in}^4$$

Compute Dead Load, Dead Load Moment, and Live Load Moment

For a 3.5-inch deck and 3-inch timber wearing surface, the dead load unit weight and moment over the effective distribution width of 18.5 inches are computed:

$$w_{DL} = (18.5 \text{ in.})\frac{(13.5 \text{ in.}+3 \text{ in.})(50 \text{ lb/ft}^3)}{1{,}728 \text{ in}^3/\text{ft}^3} = 3.5 \text{ lb/in.}$$

$$M_{DL} = \frac{w_{DL}s^2}{8} = \frac{3.5(21.25)^2}{8} = 197.6 \text{ in-lb}$$

Live load moment is computed by Equation 7-44:

$$M_{LL} = 3{,}000s - 25{,}983 = 3{,}000\,(21.25) - 25{,}983 = 37{,}767 \text{ in-lb}$$

Compute Bending Stress and Select a Lamination Species and Grade

The deck is continuous over more than two spans, so bending stress is based on 80 percent of the simple span moment:

$$M = M_{DL} + M_{LL} = 196.6 + 37{,}767 = 37{,}964 \text{ in-lb}$$

$$f_b = \frac{0.80M}{S} = \frac{0.80(37{,}964)}{37.7} = 804 \text{ lb/in}^2$$

From NDS Table 4A, No.2 Southern Pine in the size classification 2 to 4 inches thick, 2 to 4 inches wide is selected from the table “surfaced dry used at 19% m.c.” For wet-use conditions (>19 percent), NDS Table 4A footnotes require that tabulated values be taken from the Southern Pine table “surfaced green used any condition.” These values are adjusted for moisture content and further application of C_M is not required:

F_b = 1,300 lb/in^2 (repetitive member use)

F_v = 85 lb/in^2

E = 1,400,000 lb/in^2

$F_b' = F_b C_M$ = 1,300(1.0) = 1,300 lb/in^2

f_b = 804 lb/in^2 < F_b' = 1,300 lb/in^2, so a 4-inch nominal deck is satisfactory in bending. Although the allowable stress is considerably higher than the applied stress, No. 2 is the lowest grade of structural lumber that meets stress requirements.

Check Live Load Deflection

The deck is continuous over more that two spans, so deflection is 80 percent of the simple span deflection computed by Equation 7-48 (or by Figure 7-37):

$$E' = EC_M = 1{,}400{,}000\ (1.0) = 1{,}400{,}000\ \text{lb/in}^2$$

$$\Delta_{LL} = 0.80\left[\frac{1.80}{EI}\left(138.8s^3 - 20{,}780s + 90{,}000\right)\right]$$

$$\Delta_{LL} = 0.80\left[\frac{1.80\left[(138.8)(21.25)^3 - 20{,}780\,(21.25) + (90{,}000)\right]}{1{,}400{,}000\,(66.10)}\right]$$

$$= 0.02\ \text{in.}$$

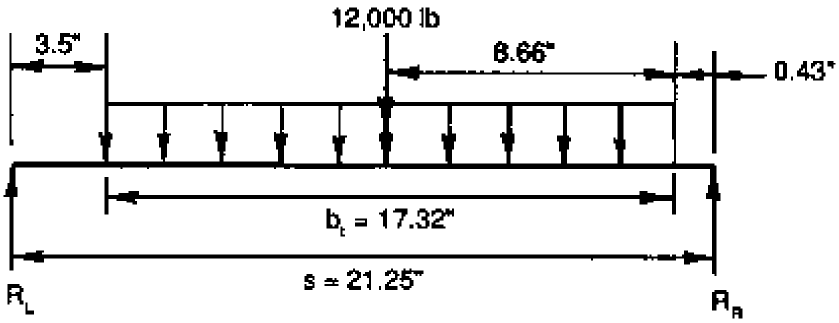
0.02 inch = *s/1,063* < *s/500,* so live load deflection is acceptable.

Check Horizontal Shear

Dead load vertical shear is computed at a distance t from the support by Equation 7-49:

$$V_{DL} = w_{DL}\left(\frac{s}{2} - t\right) = 3.5\left(\frac{21.25}{2} - 3.5\right) = 24.9\ \text{lb}$$

Live load vertical shear is computed by placing the edge of the wheel load distribution width (b_t) a distance t from the support. The resultant of the 12,000-pound wheel load acts through the center of the distribution width and V_{LL} is computed by statics:



$$V_{LL} = R_L = \frac{(12{,}000 \text{ lb})(8.66 \text{ in.} + 0.43 \text{ in.})}{21.25 \text{ in.}} = 5{,}133 \text{ lb}$$

$$V = V_{DL} + V_{LL} = 24.9 + 5{,}133 = 5{,}158 \text{ lb}$$

$$f_v = \frac{1.5V}{A} = \frac{1.5(5{,}158)}{64.75} = 119 \text{ lb/in}^2$$

By Equation 7-50,

$$F_v' = F_v C_M (\text{shear stress modification factor})$$

For nail-laminated lumber treated with oil-type preservatives, a shear stress modification factor of 2.0 is applicable (Table 7-17):

$$F_v' = 85(1.0)(2.0) = 170 \text{ lb/in}^2$$

$f_v = 119$ lb/in$^2 < F_v' = 170$ lb/in^2, so the deck is satisfactory in horizontal shear.

Summary

The deck will consist of 141 surfaced 2- by 4-inch lumber laminations that are 25 feet 8 inches long. The laminations will be nailed together and to the beams using the nailing pattern shown in Figures 7-38 and 7-39. The lumber will be No. 2 or better Southern Pine (surfaced dry), visually graded to SPIB rules. Stresses and deflection are as follows:

$$f_b = 804 \text{ lb/in}^2$$

$$F_b' = 1{,}300 \text{ lb/in}^2$$

$$\Delta_{LL} = 0.02 \text{ in.} = L/1{,}063$$

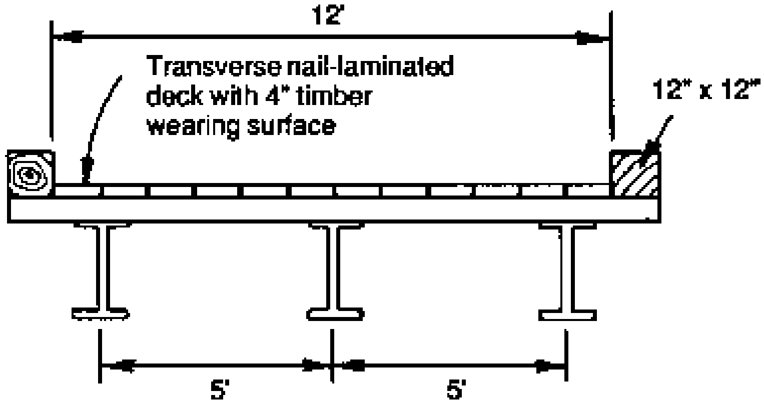
$$f_v = 119 \text{ lb/in}^2$$

$$F_v' = 170 \text{ lb/in}^2$$

Example 7-12 - Nail-laminated lumber deck design; single-lane, HS 20-44 loading

An existing bridge spans 38 feet out-to-out and is supported by three steel wide flange beams, spaced 5 feet on center. The roadway width of 12 feet carries one lane of AASHTO HS 20-44 loading. The existing concrete deck is to be removed and replaced with a continuous transverse nail-laminated lumber deck with a 4-inch-thick plank wearing surface. Design the deck for this structure, assuming the following:

1. All lumber is surfaced (S4S) visually graded Douglas Fir-Larch.
2. The beam top flange width is 12 inches.
3. Deck live load deflection is limited to *s/500*.



Solution

Define the Deck Span, Configuration, and Design Loads

Clear distance between beams = 60 in - 12 in = 48 in

$$s = 48 \text{ in.} + \frac{12}{2} \text{ in.} = 54 \text{ in.}$$

For HS 20-44 loading, AASHTO special wheel load provisions apply and the deck will be designed for a 12,000-pound wheel load. Laminations will be continuous across the deck width in lengths of 14 feet.

Estimate Deck Thickness

An initial deck thickness of 6 inches (5.5 inches actual) is selected. Deck span will be controlled by the clear distance plus deck thickness:

s = 48 in. + 5.5 in. = 53.5 in.

Determine Wheel Distribution Widths and Effective Deck Section Properties

$$b_t = \sqrt{0.025P} = \sqrt{0.025(12{,}000)} = 17.32 \text{ in.}$$

$$b_d = 15 + t = 15 + 5.5 = 20.5 \text{ in.}$$

From Table 7-14,

$$A = 112.75 \text{ in}^2$$

$$S = 103.35 \text{ in}^3$$

$$I = 284.22 \text{ in}^4$$

Compute Dead Load, Dead Load Moment, and Live Load Moment

For a 5.5-inch deck and 3.5-inch timber wearing surface over the effective distribution width of 20.5 inches,

$$w_{DL} = (20.5 \text{ in.})\left[\frac{(5.5 \text{ in.} + 3.5 \text{ in.})(50 \text{ lb/ft}^3)}{1{,}728 \text{ in}^3/\text{ft}^3}\right] = 5.3 \text{ lb/in.}$$

$$M_{DL} = \frac{w_{DL}\, s^2}{8} = \frac{5.3\,(53.50)^2}{8} = 1{,}896 \text{ in-lb}$$

Live load moment is computed by Equation 7-44:

$$M_{LL} = 3{,}000s - 25{,}983 = 3{,}000(53.50) - 25{,}983 = 134{,}517 \text{ in-lb}$$

Compute Bending Stress and Select a Lamination Species and Grade

The deck is continuous over two spans, so the 80-percent reduction in bending for span continuity does not apply.

$$M = M_{DL} + M_{LL} = 1{,}896 + 134{,}517 = 136{,}413 \text{ in-lb}$$

$$f_b = \frac{M}{S} = \frac{136{,}413}{103.35} = 1{,}320 \text{ lb/in}^2$$

From NDS Table 4A, visually graded No.1 Douglas Fir-Larch in the J&P size classification is selected. Tabulated values are as follows:

$F_b = 1{,}750$ lb/in^2 (repetitive uses) $\qquad C_M = 0.86$

$F_v = 95$ lb/in^2 $\qquad C_M = 0.97$

$E = 1{,}800{,}000$ lb/in^2 $\qquad C_M = 0.97$

$$F_b' = F_b C_M = 1{,}750(0.86) = 1{,}505 \text{ lb/in}^2$$

$f_b = 1{,}320$ lb/in$^2 < F_b' = 1{,}505$ lb/in^2, so a 6-inch nominal deck is satisfactory in bending.

Check Live Load Deflection

Maximum deflection is computed by Equation 7-48 (or Figure 7-37):

$$E' = EC_M = 1{,}800{,}000(0.97) = 1{,}746{,}000 \text{ lb/in}^2$$

$$\Delta_{LL} = \frac{1.80}{E'I}\left(138.8s^3 - 20{,}780s + 90{,}000\right)$$

$$\Delta_{LL} = \frac{1.80\left[\,(138.8)(53.5)^3 - 20{,}780\,(53.5) + (90{,}000)\,\right]}{1{,}649{,}000\,(284.22)} = 0.07 \text{ in.}$$

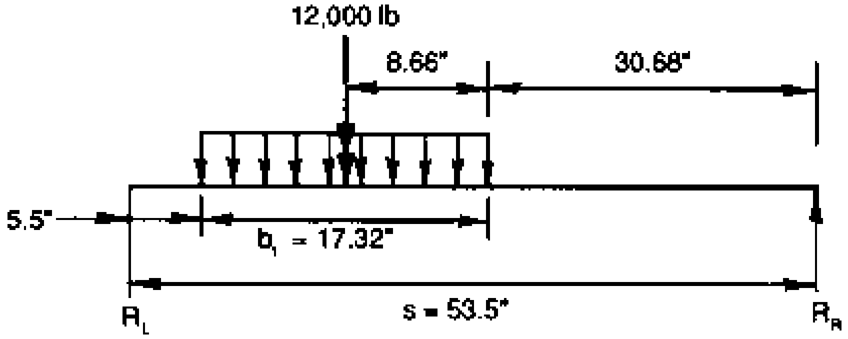
0.07 in.= *s/764* < *s/500*, so live load deflection is acceptable.

Check Horizontal Shear

For $w_{DL} = 5.3$ lb/in.,

$$V_{DL} = w_{DL}\left(\frac{s}{2} - t\right) = 5.3\left(\frac{53.5}{2} - 5.5\right) = 112.6 \text{ lb}$$

For a 12,000-pound wheel load,



$$V_{LL} = R_L = \frac{(12{,}000 \text{ lb})(8.66 \text{ in.} + 30.68 \text{ in.})}{53.5 \text{ in.}} = 8{,}824 \text{ lb}$$

$$V = V_{DL} + V_{LL} = 112.6 + 8{,}824 = 8{,}937 \text{ lb}$$

$$f_v = \frac{1.5V}{A} = \frac{1.5(8{,}937)}{112.75} = 119 \text{ lb/in}^2$$

$$F_v' = F_v C_M (\text{shear stress modification factor})$$

Using a shear stress modification factor of 2.0 (Table 7-17),

$$F_v' = 95(0.97)(2.0) = 184.30 \text{ lb/in}^2$$

$f_v = 119$ lb/in^2 < $F_v' = 184$ lb/in^2, so the deck is satisfactory in horizontal shear.

Summary

The deck will consist of 304 surfaced 2-inch by 6-inch lumber laminations, 14 feet long. The laminations will be nailed as shown in Figures 7-38 and 7-39. The lumber will be No. 1 or better Douglas Fir-Larch, visually graded to WCLIB rules. Stresses and deflection are as follows:

$$f_b = 1{,}320 \text{ lb/in}^2$$

$$F_b' = 1{,}505 \text{ lb/in}^2$$

$$\Delta_{LL} = 0.07 \text{ in.} = L/764$$

$$f_v = 119 \text{ lb/in}^2$$

$$F_v' = 184 \text{ lb/in}^2$$

DECK ATTACHMENT

Nail-laminated decks can be placed on timber or steel beams using several attachment configurations. For timber beams, the most common attachment is to nail the laminations to beam tops as the deck is constructed. Every other lamination is toenailed to every other beam with nails the same size as those used for laminating. When this method is used, the NDS recommends that toenails be driven at an angle of approximately 30 degrees with the piece and started approximately one-third the length of the nail from the edge of the piece (Figure 7-39). Although nailing provides satisfactory performance from a structural standpoint, the nails penetrate the beam top and increase susceptibility to decay. A more suitable connection is achieved using bolted bracket attachments like those used for glulam panels. On steel beams, nail-laminated decks can be attached with bolted C-clip or angle-clip attachments previously discussed. Another method of attachment involves a thin steel plate (or sheet) connector that fits over the top beam flange and is nailed to the lamination (Figure 7-40).

7.9 PLANK DECKS

Transverse plank decks consist of a series of sawn lumber planks placed flatwise across supporting beams (Figure 7-41). The planks are normally 10 or 12 inches wide and 4 inches thick, although a minimum plank thickness of 3 inches is allowed by AASHTO (AASHTO 13.9.4.1). Plank decks are used primarily on low-volume or special-use roads. They are not suitable for asphalt pavement because of large live load deflections and movements from moisture changes in the planks. In addition, plank decks are normally not practical in applications where traffic railing is required to meet full AASHTO standards (Chapter 10).

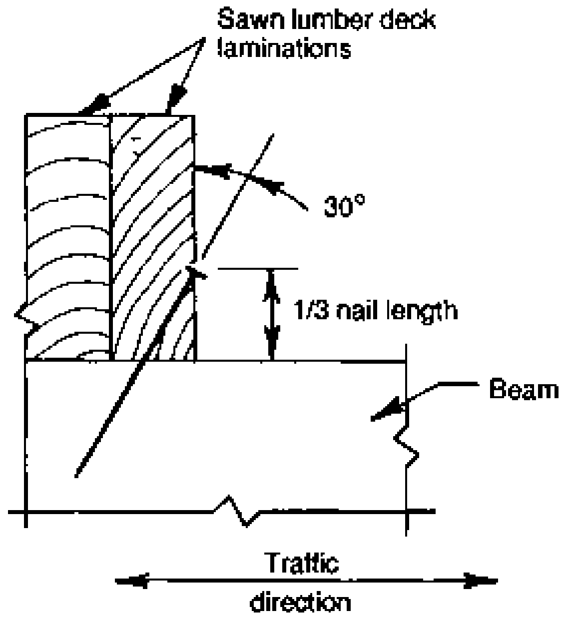


Figure 7-39. - Recommended toenail placement for attaching transverse lumber laminations to timber beams.

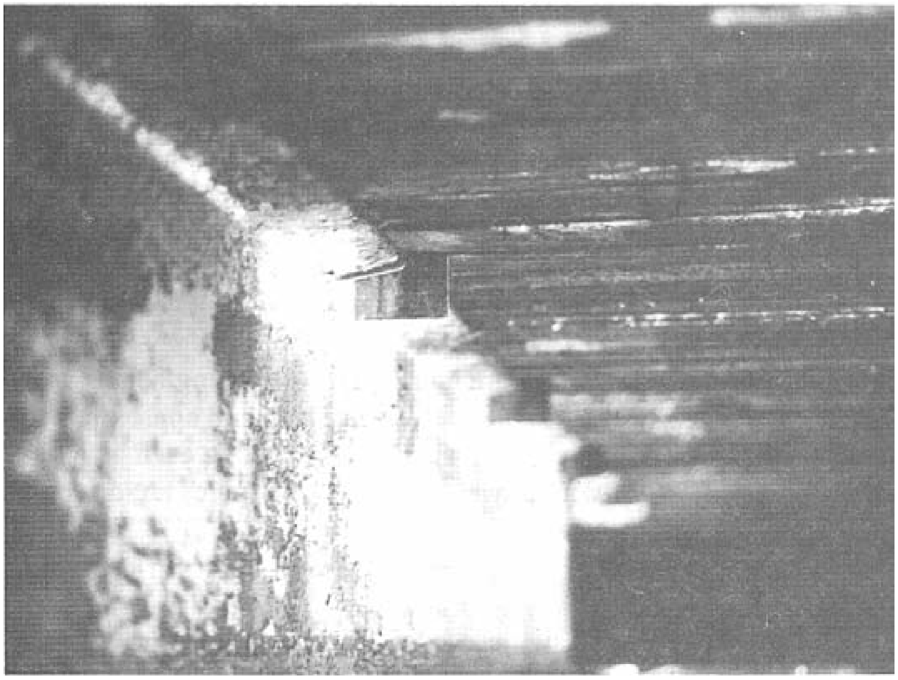


Figure 7-40. - Steel plate deck attachment for nail-laminated lumber decks on steel beams. The thin steel plate is placed over the top beam flange and is nailed to the lumber laminations during deck construction.

Figure 7-41. - Transverse plank deck on a single-lane, low-volume road (photo courtesy of Wheeler Consolidated, Inc.).

The performance of plank decks can be improved when edge-grain rather than flat-grain lumber is used (Chapter 3). In edge-grain material, dimensional changes from moisture result in fairly uniform changes in plank width and depth. For flat-grain material, dimensional changes depend on the orientation of growth rings, and swelling or shrinking can cause planks to cup. If edge-grain lumber is not available, flat-grain lumber should be placed with the bark side up so any cupping that occurs will be downward, rather than upward where water can be trapped. When green (unseasoned) planks are used, they should be placed with a tight joint between planks. When seasoned planks are used, a small gap of 1/4 to 1/2 inch should be left between planks to allow for potential swelling as the moisture content of the planks increases.

Planks are attached to supporting beams with galvanized spikes that are 1/4 to 3/8 inch in diameter and approximately twice as long as the deck is thick. Two spikes are placed in each plank at each beam. Resistance to withdrawal is improved if spikes are driven at a slight angle rather than vertically into the beam.

DESIGN PROCEDURES

Design procedures for transverse plank decks are fundamentally the same as those previously given for nail-laminated decks. Instead of a wheel load distribution width, however, wheel loads on plank decks are assumed to be distributed over the plank width (AASHTO 3.25.1.1). Because of the relatively short-span capabilities of plank decks, design is often controlled by horizontal shear rather than bending.

Design procedures for plank decks are illustrated in the following example. Approximate maximum spans for plank decks based on bending and shear are given in Tables 7-19 and 7-20.

Table 7-19. - Approximate maximum effective span for transverse plank decks based on bending; deck continuous over more than two spans; loading from a 12,000-pound wheel load plus the deck dead load; wheel-load distribution width equals the plank width.

		Maximum effective span (in.)			
		Dimension lumber[a]		Full-sawn lumber[b]	
F_b	F_b'	4 by 10	4 by 12	4 by 10	4 by 12
1,900	1,814	22	26	28	32
1,850	1,766	22	25	28	32
1,800	1,718	22	24	27	31
1,750	1,623	21	24	27	30
1,700	1,623	21	24	26	30
1,650	1,527	21	23	26	29
1,600	1,527	20	23	25	28
1,550	1,480	20	22	25	28
1,500	1,432	19	22	24	27
1,450	1,384	19	21	24	27
1,400	1,336	19	21	23	26
1,350	1,289	18	20	22	25
1,300	1,240	18	20	22	25
1,250	1,193	18	20	21	24
1,200	1,146	17	19	21	23
1,150	1,098	17	19	20	23
1,100	1,050	16	18	20	22
1,050	1,002	16	18	19	22
1,000	955	16	17	19	21

[a] Plank sizes for 4 by 10 and 4 by 12 dressed lumber are 3-1/2 inches by 9-1/4 inches and 3-1/2 inches by 11-1/4 inches, respectively.

[b] Plank sizes for 4 by 10 and 4 by 12 full-sawn lumber are 4 inches by 10 inches and 4 inches by 12 inches, respectively.

$F_b' = F_b C_M$(modification factor for flatwise use) $= F_b(0.86)(1.11)$.

Table 7-20. - Approximate maximum effective span for transverse plank decks based on horizontal shear; loading from a 12,000-pound wheel load plus the deck dead load; wheel load distribution width equal to plank width.

		Maximum Effective Span (in.)			
		Dimension lumber[a]		Full-sawn lumber[b]	
F_v	F_v'	4 by 10	4 by 12	4 by 10	4 by 12
100	194	18	21	22	26
95	184	17	20	21	24
90	175	17	19	20	23
85	165	16	18	19	22
80	155	15	18	19	21
75	146	15	17	18	20
70	136	14	16	17	19
65	126	14	15	16	18
60	116	13	15	15	17

[a] Plank sizes for 4 by 10 and 4 by 12 dressed lumber are 3-1/2 inches by 9-1/4 inches and 3-1/2 inches by 11-1/4 inches, respectively.

[b] Plank sizes for 4 by 10 and 4 by 12 full-sawn lumber are 4 inches by 10 inches and 4 inches by 12 inches, respectively.

$F_v' = F_v C_M$(Shear stress modification factor) $= F_v(0.97)(2.0)$.

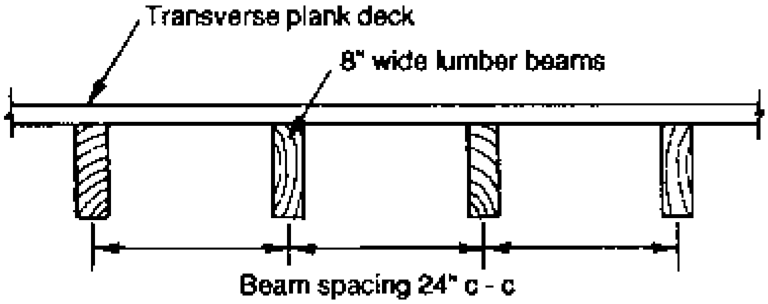
Example 7-13 - Transverse plank deck design; single-lane HS 15-44 loading

A longitudinal lumber beam superstructure carries AASHTO HS 15-44 loading and consists of a series of nominal &inch-wide lumber beams spaced 24 inches center-to-center. Design a transverse plank deck for this bridge assuming the following:

1. The deck is provided with a full-width lumber wearing surface constructed of nominal 2-inch planks.

2. All lumber, including the wearing surface, is dressed (S4S) Douglas Fir-Larch.

3. Deck live load deflection must be limited to *s/500.*



Solution

Define the Deck Span, Configuration, and Design Loads

The deck span is the clear distance between supporting beams plus one-half the width of one beam, but not greater than the clear span plus the deck thickness. From Table 16-2, the actual width of a dressed 8-inch-wide beam is 7.50 inches:

Clear distance between beams = 24 in. - 7.5 in. = 16.5 in.

$$s = 16.5 \text{ in.} + \frac{7.5}{2} \text{ in.} = 20.25 \text{ in.}$$

If a nominal 4-inch-thick plank is used (3.5 inches actual thickness), the deck span will be limited by the clear span plus the deck thickness:

s = 16.5 in. + 3.5 in. = 20 in.

For HS 15-44 loading, the deck will be designed for a 12,000-pound wheel load.

Estimate Plank Size and Determine Section Properties

Plank decks are generally constructed of 4- by 10-inch or 4- by 12-inch lumber. In this case, a dressed 4- by 12-inch plank is selected. Section properties are obtained from Table 16.2:

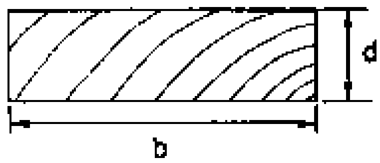
b = 11.25 in.

d = 3.50 in.

A = 39.38 in^2

S = 22.97 in^3

I = 40.20 in^4



Determine Wheel Distribution Widths

In the direction of the deck span, the wheel load is distributed over the tire width given by Equation 7-42:

$$b_t = \sqrt{0.025P} = \sqrt{0.025(12{,}000)} = 17.32 \text{ in.}$$

Normal to the deck span, the wheel load is distributed over the plank width of 11.25 inches.

Compute Dead Load, Dead Load Moment, and Live Load Moment

For a 3.5-inch deck and 1.5-inch timber wearing surface, the dead load is computed for the plank width:

$$w_{DL} = (11.25 \text{ in.}) \frac{\left[(3.5 \text{ in.} + 1.5 \text{ in.})(50 \text{ lb/ft}^3)\right]}{1{,}728 \text{ in}^3/\text{ft}^3} = 1.6 \text{ lb/in.}$$

$$M_{DL} = \frac{w_{DL} s^2}{8} = \frac{1.6(20)^2}{8} = 80 \text{ in-lb}$$

Live load moment is computed by Equation 7-44:

$$M_{LL} = 3{,}000s - 25{,}983 = 3{,}000(20) - 25{,}983 = 34{,}017 \text{ in-lb}$$

Compute Bending Stress and Select Plank Species and Grade

The deck is continuous over more than two spans, so bending stress is based on 80 percent of the simple span moment:

$$M = M_{DL} + M_{LL} = 80 + 34{,}017 = 34{,}097 \text{ in-lb}$$

$$f_b = \frac{0.80M}{S} = \frac{0.80(34{,}097)}{22.97} = 1{,}188 \text{ lb/in}^2$$

From Table 4A of the NDS, No. 2 Douglas Fir-Larch in the J&P size classification is chosen with the following tabulated values:

$$F_b = 1{,}250 \text{ lb/in}^2 \qquad C_M = 0.86$$

$$F_v = 95 \text{ lb/in}^2 \qquad C_M = 0.97$$

$$E = 1{,}700{,}000 \text{ lb/in}^2 \qquad C_M = 0.97$$

Footnotes to the NDS tabulated values also specify that bending stress may be increased by a factor of 1.11 for flatwise use:

$$F_b' = F_b C_M (1.11) = 1{,}250(0.86)(1.11) = 1{,}193 \text{ lb/in}^2$$

$f_b = 1{,}188$ lb/in$^2 < F_b' = 1{,}193$ lb/in^2, so the plank size and grade are satisfactory in bending.

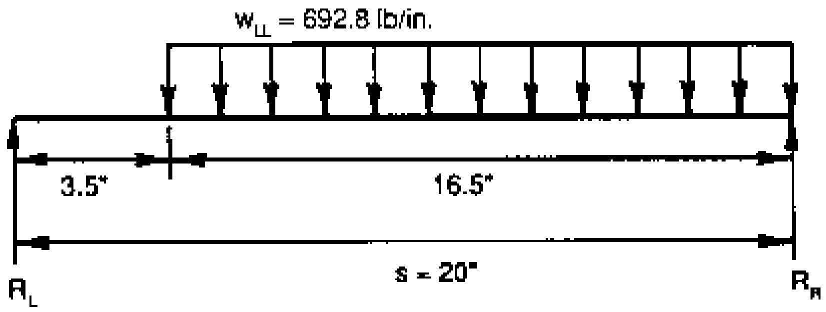
Check Horizontal Shear

Dead load vertical shear is computed at a distance t from the support. By Equation 7-49 for w_{DL} = 1.6 lb/in,

$$V_{DL} = w_{DL}\left(\frac{s}{2} - t\right) = 1.6\left(\frac{20}{2} - 3.5\right) = 10.4 \text{ lb}$$

Live load vertical shear is computed by placing the edge of the wheel load distribution width (b_t) a distance t from the support. In this case, the remaining span is less than b_t and the wheel load is converted to a uniform load:

$$w_{LL} = \frac{P}{b_t} = \frac{12{,}000}{17.32} = 692.8 \text{ lb/in.}$$



$$V_{LL} = R_L = \frac{(692 \text{ lb/in.})(16.5 \text{ in.})(16.5 \text{ in.}/2)}{20 \text{ in.}} = 4{,}715 \text{ lb}$$

$$V = V_{DL} + V_{LL} = 10.4 + 4{,}715 = 4{,}725 \text{ lb}$$

$$f_v = \frac{1.5\,V}{A} = \frac{1.5\,(4{,}725)}{39.38} = 180 \text{ lb/in}^2$$

By Equation 7-50,

$$F_v' = F_v C_M \text{(shear stress modification factor)}$$

For planks treated with oil-type preservatives, a 2.0 shear stress modification factor is used (Table 7-17):

$$F_v' = 95(0.97)(2.0) = 184 \text{ lb/in}^2$$

f_v = 180 lb/in^2 < F_v' = 184 lb/in^2, so the deck is satisfactory in horizontal shear.

Check Live Load Deflection

Maximum deflection for a 12,000-pound wheel load and 6-foot track width on a simple span is computed by Equation 7-48. Because the deck is continuous over more than two spans, 80 percent of the simple span deflection is used to account for span continuity:

$E' = EC_M = 1{,}700{,}000(0.97) = 1{,}649{,}000 \text{ lb/in}^2$

$$\Delta_{LL} = (0.80)\left[\frac{1.80}{E'I}\left(138.8s^3 - 20{,}780s + 90{,}000\right)\right]$$

$$\Delta_{LL} = (0.80)\left[\frac{1.80\left[(138.8)(20)^3 - 20{,}780(20) + (90{,}000)\right]}{1{,}649{,}000\,(40.20)}\right] = 0.02 \text{ in.}$$

A deflection of 0.02 inch = s/1,000 < s/500, so live load deflection is acceptable.

Summary

The deck will consist of surfaced 4-inch by 12-inch Douglas Fir-Larch planks, visually graded No. 2 or better in the J&P size classification. Stresses and deflection are as follows:

$f_b = 1{,}188 \text{ lb/in}^2$

$F_b' = 1{,}193 \text{ lb/in}^2$

$\Delta_{LL} = 0.02 \text{ in.} = s/1{,}000$

$f_v = 180 \text{ lb/in}^2$

$F_v' = 184 \text{ lb/in}^2$

7.10 SELECTED REFERENCES

1. American Association of State Highway and Transportation Officials. 1983. Standard specifications for highway bridges. 13th ed. Washington, DC: American Association of State Highway and Transportation Officials. 394 p.
2. American Association of State Highway and Transportation Officials. 1983. Manual for maintenance inspection of bridges. Washington, DC: American Association of State Highway and Transportation Officials. 50 p.
3. American Association of State Highway and Transportation Officials. 1982. AASHTO materials: pt. 1, specifications. Washington, DC: American Association of State Highway and Transportation Officials. 1094 p.
4. American Institute of Timber Construction in conjunction with the Virginia Highway Research Council. [1973]. Typical timber bridge design and details. Englewood, CO: American Institute of Timber Construction. 10 p.

5. American Institute of Timber Construction. 1987. Design standard specifications for structural glued laminated timber of softwood species. AITC 117-87-Design. Englewood, CO: American Institute of Timber Construction. 28 p.
6. American Institute of Timber Construction. 1974. Glulam bridge systems plans and details. Englewood, CO: American Institute of Timber Construction. 16 p.
7. American Institute of Timber Construction. 1988. Glulam bridge systems. Vancouver, WA: American Institute of Timber Construction. 33 p.
8. American Institute of Timber Construction. 1973. Modem timber highway bridges, a state of the art report. Englewood, CO: American Institute of Timber Construction. 79 p.
9. American Institute of Timber Construction. 1985. Timber construction manual. 3d ed. New York: John Wiley and Sons, Inc. 836 p.
10. American Society of Civil Engineers. 1975. Wood structures, a design guide and commentary. New York: American Society of Civil Engineers. 416 p.
11. American Wood-Preservers' Association. 1987. Book of standards. Stevensville, MD: American Wood-Preservers' Association. 240 p.
12. Anderson, L.O.; Heebink, T.B.; Oviatt, A.E. 1971. Construction guide for exposed wood decks. Portland, OR: U.S. Department of Agriculture, Forest Service, Pacific Northwest Forest and Range Experiment Station. 78 p.
13. Barnhart, J.E. 1986. Ohio's experiences with treated timber for bridge construction. In: Trans. Res. Rec. 1053. Washington, DC: National Academy of Sciences, National Research Council, Transportation Research Board: 56-58.
14. Bell, L.C.; Yoo, C.H. 1984. Seminar on fundamentals of timber bridge construction. Course notes; 1984 May 22-25; Auburn University, AL. Auburn University. [150 p.].
15. Better Roads. 1976. Glulam helping to solve America's bridge problem. Better Roads 46(5): 36-37.
16. Bohannan, B. 1972. FPL timber bridge deck research. Journal of the Structural Division, American Society of Civil Engineers 98(ST3): 729-740.
17. Boomsliter, G.P.; Cather, C.H.; Worrell, D.T. 1951. Distribution of wheel loads on a timber bridge floor. Res. Bull. 24. Morgantown, WV: West Virginia University, Engineering Experiment Station. 31 p.
18. Canadian Institute of Timber Construction. 1970. Modem timber bridges, some standards and details. 3d ed. Ottawa, Can.: Canadian Institute of Timber Construction. 48 p.
19. Commonwealth of Pennsylvania, Department of Transportation. 1984. Standard plans for low cost bridges. Series BLC-540, timber spans. Pub. No. 130. [Pittsburgh, PA]: Commonwealth of Pennsylvania, Department of Transportation. 28 p.

20. Erickson, E.C.O.; Romstad, K.M. 1965. Distribution of wheel loads on timber bridges. Res. Pap. FPL 44. Madison, WI: U.S. Department of Agriculture, Forest Service, Forest Products Laboratory. 62 p.
21. Freas, A.D. 1952. Laminated timber permits flexibility of design. Civil Engineering 22(9): 173-175.
22. Gurfinkel, G. 1981. Wood engineering. 2d ed. Dubuque, IA: Kendall/ Hunt Publishing Co. 552 p.
23. Gutkowski, R.M.; McCutcheon, W.J. 1984. Comparative performance of experimental timber bridges. Madison, WI: U.S. Department of Agriculture, Forest Service, Forest Products Laboratory. 27 p.
24. Gutkowski, R.M.; Williamson, T.G. 1983. Timber bridges: state-of-the-art Journal of Structural Engineering 109(9): 2175-2191.
25. Hale, C.Y. 1975. Field test of a 40-ft span, two-lane Weyerhaeuser panelized wood bridge. Rep. No. RDR 045-1092. Tacoma, WA: Weyerhaeuser Co. 22 p.
26. Hockaday, E. 1980. Preliminary study of duration of load for glulam wood bridges used in rural areas. St. Paul, MN: Weyerhaeuser Co. 4p.
27. Leviasky, S. 1982. Timber bridges. In: Arches and short span bridges. New York: Chapman and Hall Ltd. 48-83.
28. Manesh, A.A.S. 1977. Design formulas for slab-stringer bridges. Morgantown, WV: West Virginia University, Department of Civil Engineering. 53 p.
29. McCutcheon, W.J.; Tuomi, R.L. 1973. Procedure for design of glued-laminated orthotropic bridge decks. Res. Pap. FPL 210. Madison, WI: U.S. Department of Agriculture, Forest Service, Forest Products Laboratory. 42 p.
30. McCutcheon, W.J.; Tuomi, R.L. 1974. Simplified design procedure for glued-laminated bridge decks. Res. Pap. FPL 233. Madison, WI: U.S. Department of Agriculture, Forest Service, Forest Products Laboratory. 8 p.
31. Mielke, K.F. 1977. Experimental project for glued-laminated timber deck panels on highway bridges. Juneau, AK: State of Alaska, Department of Highways. [50 p.].
32. Ministry of Transportation and Communications. 1986. Ontario highway bridge design code updates. Downsview, ON, Can.: Ministry of Transportation and Communications. 28 p.
33. Ministry of Transportation and Communications. 1983. Ontario highway bridge design code. Downsview, ON, Can.: Ministry of Transportation and Communications. 357 p.
34. Ministry of Transportation and Communications. 1983. Ontario highway bridge design code commentary. Downsview, ON, Can,: Ministry of Transportation and Communications. 279 p.
35. Muchmore, F.W. 1986. Designing timber bridges for long life. In: Trans. Res. Rec. 1053. Washington, DC: National Academy of Sciences, National Research Council, Transportation Research Board: 12-17.

36. Nagy, M.M.; Trebett, J.T.; Wellburn, G.V. 1980. Log bridge construction handbook. Vancouver, Can.: Forest Engineering Research Institute of Canada. 421 p.
37. National Forest Products Association. 1986. Design values for wood construction. A supplement to the national design specification for wood construction. Washington, DC: National Forest Products Association. 34 p.
38. National Forest Products Association. 1986. National design specification for wood construction. Washington, DC: National Forest Products Association. 87 p.
39. Nowak, A.S.; Taylor, R.J. 1986. Ultimate strength of timber deck bridges. In: Trans. Res. Rec. 1053. Washington, DC: National Academy of Sciences, National Research Council, Transportation Research Board: 26-30.
40. Ou, F.L. 1986. An overview of timber bridges. In: Trans. Res. Rec. 1053. Washington, DC: National Academy of Sciences, National Research Council, Transportation Research Board: 1-12.
41. Parry, J.D. 1986. A prefabricated modular timber bridge. In: Trans. Res. Rec. 1053. Washington, DC: National Academy of Sciences, National Research Council, Transportation Research Board: 49-55.
42. Sanders, W.W. 1984. Distribution of wheel loads on highway bridges. National Cooperative Highway Research Program Synthesis of Highway Practice. No. 3. Washington, DC: National Academy of Sciences, National Research Council, Transportation Research Board. 22 p.
43. Sanders, W.W., Jr. 1980. Load distribution in glulam timber highway bridges. Report ISU-ERI-AMES-80124. Ames, IA: Iowa State University, Engineering Research Institute. 21 p.
44. Sanders, W.W., Jr.; Elleby, H.A. 1970. Distribution of wheel loads on highway bridges. Cooperative Highway Research Program. Rep. 83. Washington, DC: National Academy of Sciences, National Research Council, Highway Research Board. 56 p.
45. Scales, W.H. 1959. Standard treated timber bridges. In: Standardization of highway bridges. Bull. No. 244. Washington, DC: American Road Builders' Association: 22-26.
46. Scarisbrick, R.G. 1976. Laminated timber logging bridges in British Columbia. Journal of the Structural Division, American Society of Civil Engineers 102(ST1): [10 p.].
47. Selbo, M.L. 1966. Laminated bridge decking (progress report). Madison, WI: U.S. Department of Agriculture, Forest Service, Forest Products Laboratory. 32 p.
48. Selbo, M.O.; Knauss, A.C.; Worth, H.E. 20 years of service prove durability of pressure-treated glulam bridge members. Wood Preserving News 44(3): 5-8.
49. Southern Forest Products Association. 1985. Southern Pine use guide. New Orleans, LA: Southern Forest Products Association. 13 p.

50. Sprinkel, M.M. 1978. Glulam timber deck bridges. VHTRC 79-R26. Charlottesville, VA: Virginia Highway and Transportation Research Council. 33 p.
51. Sprinkel, M.M. 1985. Prefabricated bridge elements and systems. National Cooperative Highway Research Program, Synthesis of Highway Practice 119. Washington, DC: National Academy of Sciences, National Research Council, Transportation Research Board. 75 p.
52. Stacey, W.A. 1935. The design of laminated timber bridge floors. Wood Preserving News 13(4): 44-46, 55-56.
53. Stanton, J.F.; Roeder, C.W. 1982. Elastomeric bearing design. Nat. Coop. Highw. Res. Program Rep. 248. Washington, DC: National Academy of Sciences, National Research Council, Transportation Research Board. 82 p.
54. Stone, M.F. [1975]. New concepts for short span panelized bridge design of glulam timber. Tacoma, WA: Weyerhaeuser Co. 8 p.
55. Sunset Foundry Company, Inc. [1970]. Deck brackets for treated timber bridges. AIA File 17F. Kent, WA: Sunset Foundry. 4 p.
56. Tuomi, R.L. 1972. Advancements in timber bridges through research and engineering. In: Proceedings, 13th annual Colorado State University bridge engineering conference; Ft. Collins, CO. Colorado State University: 34-61.
57. Tuomi, R.L. 1976. Erection procedure for glued-laminated timber bridge decks with dowel connectors. Res. Pap. FPL 263. Madison, WI: U.S. Department of Agriculture, Forest Service, Forest Products Laboratory. 15 p.
58. Tuomi, R.L.; McCutcheon, W.J. 1973. Design procedure for glued laminated bridge decks. Forest Products Journal 23(6): 36-42.
59. U.S. Department of Agriculture, Forest Service, Northern Region. 1985. Bridge design manual. Missoula, MT: U.S. Department of Agriculture, Forest Service, Northern Region. 299 p.
60. U.S. Department of Transportation, Federal Highway Administration. 1979. Standard plans for highway bridges. Timber bridges. Washington, DC: U.S. Department of Transportation, Federal Highway Administration. Vol. 3. 19 p.
61. Weyerhaeuser Company. 1976. Weyerhaeuser bridge deck bracket. SL-495. Tacoma, WA: Weyerhaeuser Co. 4 p.
62. Weyerhaeuser Company. 1980. Weyerhaeuser glulam wood bridge systems. Tacoma, WA: Weyerhaeuser Co.; 1980. 114 p.
63. Weyerhaeuser Company. 1975. Weyerhaeuser panelized bridge system for secondary roadways, highways & footbridges. SL-1318. Tacoma, WA: Weyerhaeuser Co. 4 p.
64. Weyerhaeuser Company. 1974. Weyerhaeuser panelized bridge system. SL-1318. Tacoma, WA: Weyerhaeuser Co. 4 p.