CHAPTER 10
MISCELLANEOUS WOOD STRUCTURES

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10.1 WOOD BRIDGES

10.1.1 Introduction

Wood was probably the first material used by humans to construct a bridge. Although in the twentieth century concrete and steel replaced wood as the major material for bridge construction, wood is still widely used for short- and medium-span bridges. Of the bridges listed in the National Bridge Inventory with spans longer than 20 ft (6.10 m), approximately 8 percent, or 37,030 bridges, are made entirely of wood and 11 percent, or 51,422 bridges, use wood as one of the primary structural materials. The strength, lightweight, and energy-absorbing properties of wood are features that are desirable for bridge construction. Wood is capable of supporting short-term overloads without adverse effects, and contrary to popular belief, large wood members provide good fire-resistance qualities that meet or exceed those of other materials in severe fire exposures. From an economic standpoint, wood is competitive with other materials on a first-cost basis and shows advantages when life-cycle costs are compared. Wood bridges can be constructed in virtually any weather conditions without detriment to the material. The material is not damaged by continuous freezing and thawing and resists harmful effects of deicing agents, which cause deterioration in other bridge materials. Wood bridges do not require special equipment for installation and normally can be constructed without highly skilled labor. They also present a natural and aesthetically pleasing appearance, particularly in natural surroundings.

The misconception that wood provides a short service life has plagued wood as a construction material. Although wood is susceptible to decay or insect attack under specific conditions, it is inherently a very durable material when protected from moisture. Many covered bridges built during the nineteenth century have lasted over 100 years because they were protected from direct exposure to the

Note: In all example problems, SI units are provided only for given data and the final problem answer or answers. No intermediate values or data are expressed in SI units.

elements. In modem applications, it is seldom practical or economical to cover bridges; however, the use of wood preservatives has extended the life of wood used in exposed bridge applications. Using modem application techniques and preservative chemicals, wood can now be effectively protected from deterioration for periods of 50 years or longer. Treated wood requires little maintenance and no painting, which are distinct advantages over the life of the structure.

Another misconception about wood as a bridge material is that its use is limited to minor structures of no appreciable size. This belief is probably based on the fact that trees for commercial use are limited in size and are normally harvested before they reach maximum size. Although tree diameter limits the size of sawn lumber, the advent of glued-laminated timber (glulam timber) some 50 years ago provided designers with several compensating alternatives. Glulam timber, which is the most widely used modem wood bridge material, is manufactured by bonding sawn lumber laminations together with waterproof structural adhesives. Thus glulam timber members are virtually unlimited in depth, width, and length and can be manufactured in a wide range of shapes. Glulam timber provides higher design strengths than sawn lumber and provides better utilization of the available wood resource by permitting the manufacture of large wood structural elements from smaller lumber sizes. Technological advances in laminating over the past four decades have further increased the suitability and performance of wood for modem highway bridge applications.

This section addresses the design of two types of glulam timber bridges: beam bridges with transverse decks and longitudinal glulam deck (slab) bridges. The material presented in the section is based on the 1992 edition of the AASHTO *Standard Specifications for Highway Bridges* (AASHTO specifications),' including interim specifications through 1993. 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 section is not intended to serve as a substitute for current specifications.

10.12 Design of Glulam Timber Beam Bridges

Beam bridges consist of a series of longitudinal wood beams supporting a transverse wood deck. They are constructed of glulam timber or sawn lumber components and historically have been one of the most common and most economical types of timber bridge. For the past 25 years, beam bridges have been constructed almost exclusively from glulam timber because of the greater size and better performance characteristics glulam provides compared with sawn lumber systems. Sawn lumber beam bridges are still used to a limited degree on low-volume public and private road systems. The focus of this section will be on beam bridges constructed of glulam timber. For information regarding the design of sawn lumber beam bridges, refer to AASHTO specifications' and Ritter (1990).[6]

Glulam timber beam bridges consist of a series of transverse glulam timber deck panels supported on straight or slightly curved beams (Fig. 10.1). They are the most practical for clear spans of 20 to 100 ft (6.10 to 30.5 m) and are widely used on single- and multiple-lane roads and highways. Glulam timber has proved to be an excellent material for beam bridges because members are available in a range of

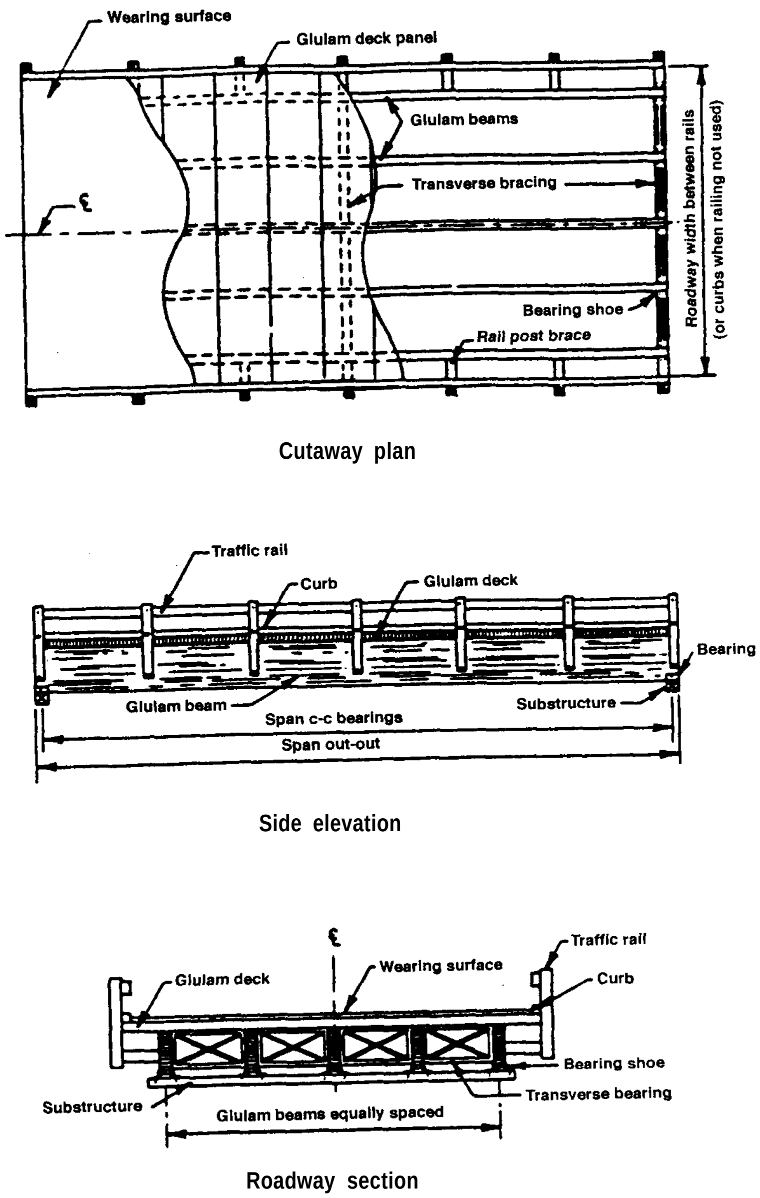


FIG. 10.1 Typical glulam beam bridge configuration.

sizes and grades and are easily adaptable to a modular or systems concept of design and construction. Although glulam timber 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.

10.1.3 Beam Design

Glulam timber bridge beams are horizontally laminated members designed from the bending combinations given in AASHTO specifications' and the NDS Supplement.[3] 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 timber 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 14½ in (76.2 to 362 mm) (Table 10.1) and in depth multiples of 1½ in (38.1 mm) for western species and $1^3/_8$ in (34.9 mm) for southern pine. Beam length is usually limited by treating and transportation considerations to a practical maximum of 110 to 120 ft (33.5 to 36.6 m), but longer members may be feasible in some areas.

Live-Load Distribution. Methods for determining the maximum moment, shear, and reactions for truck and lane loads are given in the AASHTO specifications.' For beam bridges, the designer also must 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 the beams. Load distribution also may be influenced by the type and spacing of beam transverse bracing. In view of the complexity of the theoretical analysis involved in determining lateral wheel-load distribution, the 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

TABLE 10.1 Standard Glulam Timber Beam Widths

	Net finished width (in)	
Nominal width (in)	Western species	Southern pine
4	$3\,^1/_8$	3
6	$5\,^1/_8$	5
8	6¾	6¾
10	8¾	8½
12	10¾	10½
14	12¼	12¼
16	14¼	14¼

Note: 1 in = 25.4 mm.

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, deflection shear, and reactions.

1. *Distribution for moment.* When computing bending moments in longitudinal beams, 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. 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 ft (0.610 m) from the face of the traffic railing or curb. 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 10.2). For glulam timber decks 6 in (152.4 mm) 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. In this case, wheel lines are laterally positioned in traffic lanes to produce the maximum beam reaction, but wheel lines in adjacent traffic lanes are separated by a minimum of 4 ft (1.219 m).

2. *Distribution for dejection.* Lateral load distribution for determining deflection is generally determined by using the same criteria specified for moment. However, AASHTO allows that for timber beam bridges with beams of equal stiffness and cross-bracing or diaphragms sufficient in depth and strength to ensure lateral distribution of loads, the deflection may be computed by considering all beams as acting together and having equal deflection.

3. *Distribution for shear.* Live-load horizontal shear in glulam timber beams is computed from the maximum vertical shear occurring at a distance from the beam

TABLE 10.2 Interior Beam Live-Load Distribution Factors for Glulam Timber Beams with Transverse Glulam Timber Decks

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

Note: 1 ft = 0.3048 m

[a]In this case. the distribution factor for each beam is the reuction of the wheel lines. assuming the deck between beams to act as a simple beam. S = average beam spacing (ft).

support equal to three times the beam depth (3d) or the span quarter point ($L/4$), whichever is less. 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 in the following equation:

$$V_{LL} = 0.5(0.6V_{LU} + V_{LD}) \tag{10.1}$$

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

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

4. *Distribution for reactions.* Live-load distribution for end reactions is computed assuming no longitudinal distribution of wheel loads. The distribution factor 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.

Beam Configuration. One of the most influential factors on the overall economy and performance of a glulam timber bridge is the beam configuration. For a given roadway width, the spacing of beams affects size and strength requirements for both beam and deck elements and significantly influences the cost for material, fabrication, and construction. Numerous combinations of beam size and spacing are possible, and the designer must select the most economical combination that provides the required structural capacity and meets serviceability requirements. 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.

1. *Site restrictions.* Efficient beam design favors a relatively narrow, deep section with a width-to-depth ratio of 4:1 to 7:1. In some cases, the optimal 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 (Fig. 10.2). In most cases, however, the longitudinal deck designs discussed in the following sections will provide a more economical design for restricted-depth crossings when span requirements permit.

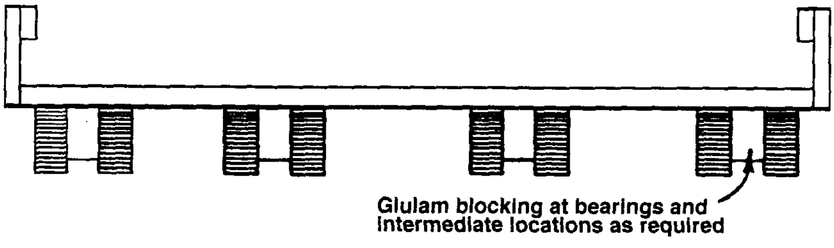


FIG. 10.2 Typical low-profile glulam beam configuration.

2. ***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 timber deck panels is based on standard member sizes that increase in depth in 1½ to 2-in (38.1- to 50.8-mm) increments. As a result, the load-carrying capacity and stiffness of a panel are adequate for a range of beam spacings. For example, a 6¾-in (171.4-mm) deck panel is used when the computed deck thickness is between 5 $^{1}/_{8}$ and 6¾ in (130.0 and 171.4 mm). The largest effect of beam spacing on the deck occurs when the panel thickness must be increased to the next thicker panel, e.g., from 6¾ to 8¾ in (171.4 to 222.2 mm). 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 timber decks supporting highway loads is between 4.5 and 6.5 ft (1.371 and 1.981 m). 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 ft (0.762 m). These values are based on deck stress and deflection considerations that may vary slightly for different panel combination symbols and configurations.

3. ***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 (DF), the greater is the load the beam must support. Thus the value of the distribution factor gives a good indication of relative beam size and grade requirements for different configurations. It is generally beneficial to have the distribution factor for interior beams and outside beams approximately equal so that the beam sizes are equal. If there is a significant difference between interior and outside beams, different beam sizes will be required. A summary of suggested beam configurations for various bridge widths that result in the same distribution factor to interior and outside beams is given in Table 10.3. For additional information, refer to Ritter (1990).[6]

10.1.4 Deck Design

Glulam timber decks are constructed of panels manufactured of vertically laminated lumber. The panels are placed transverse to supporting beams, and loads act parallel to the wide face of the laminations. Glulam timber decks 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 timber panels can be placed to provide a watertight deck, protecting the structure from the deteriorating effects of rain and snow. The two basic types of glulam timber decks are the noninterconnected deck and the doweled deck. Noninterconnected decks have no mechanical connection between adjacent panels. Doweled decks are interconnected with steel dowels to distribute loads between adjacent panels. Noninterconnected glulam timber decks are the most commonly used glulam timber deck. 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. Discussions in this section will be limited to noninterconnected decks, which are most common.

Noninterconnected glulam timber decks are designed from the axial glulam timber combinations given in the AASHTO specifications[1] and the NDS supplement.[3]

TABLE 10.3 Recommended Beam Spacing for Glulam Timber Beams with Transverse Glulam Timber Decks

Roadway width,* ft	Number of beams	Beam spacing, ft	Deck overhang,† ft	Moment DF, interior beams‡	Moment DF, outside beams§
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

Note: 1 ft. = 0.3048 m.
*Measured face to face of railings or of curbs when railing is not used.
†Measured from centerline of outside beam to face of railing or curbs.
‡For glulam decks 6 in or more in nominal thickness ($S/6$ for single-lane: $S/5$ for two or more lanes).
§Computed assuming the deck acts as a simple span between beams but not less than the interior beam DE

These combinations provide the most economical and efficient layups where primary loading is applied parallel to the wide face of the laminations. Deck panels are normally 5 $^1/_8$ in (130.2 mm) (5 in for southern pine) or 6¾ in (171.4 mm) thick. Increased thicknesses of 8¾ to 12¼ in (222.2 to 311.1 mm) are available but are seldom required. Panel width is a multiple of 1½ in (38.1 mm) for western species and 1 $^3/_8$ in (34.9 mm) for southern pine, the net width of the individual lumber laminations. The practical width of panels ranges from approximately 30 to 5.5 in (0.762 to 1.397 m); however, the designer should check local manufacturing and treating limitations before specifying widths over 48 in (1.219 m). 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.

Live-Load Distribution. Load distribution for glulam timber deck panels is specified in the AASHTO Specifications' as a function of the wheel load width and the deck thickness. Different criteria are used for moment and deflection, as well as for shear.

1. *Distribution for moment and deflection.* In the direction of the deck span s, perpendicular to traffic, the wheel load is assumed to be uniformly distributed over a width given by the following equation (see Fig. 10.3):

$$b_t = \sqrt{0.025P} \tag{10.2}$$

where b_t = wheel load distribution width in the direction of the deck span, in
P = maximum wheel load, lb

For a 12,000-lb (53.34-kN) wheel load, b_t = 17.32 in (0.440 m).

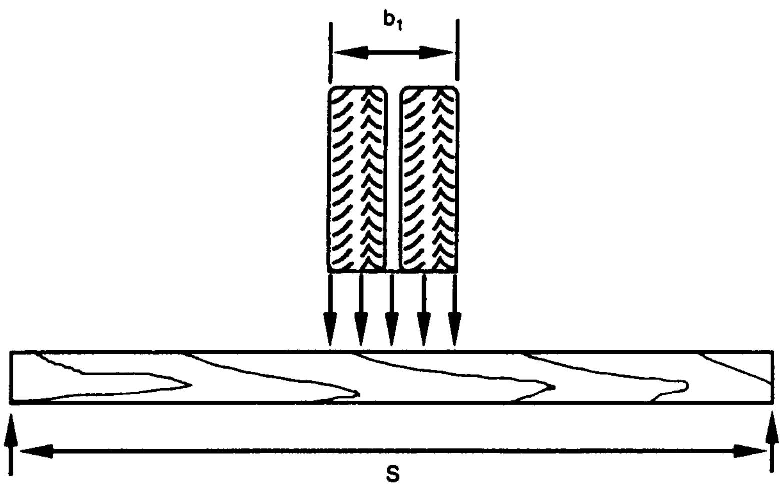


FIG. 10.3 Wheel load distribution parallel to deck span.

In the direction perpendicular to the deck span, parallel to traffic, the wheel load is distributed over an effective width equal to the deck thickness plus 15 in (381 mm), but not greater than the deck panel width (Fig. 10.4):

$$b_d = t + 15 \leq \text{actual panel width} \tag{10.3}$$

where b_d = wheel load distribution width perpendicular to the deck span, in
t = deck thickness, in

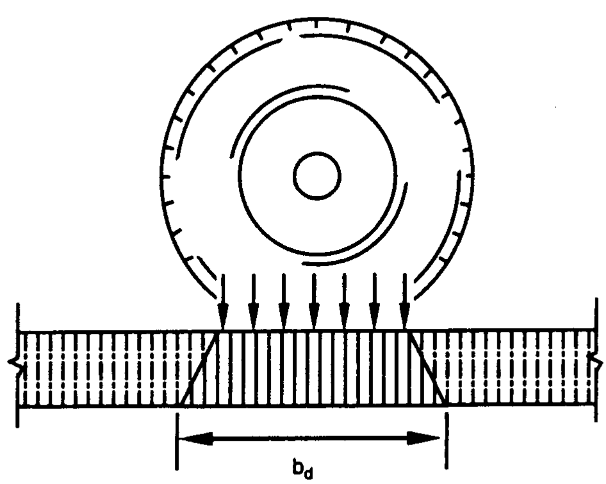


FIG. 10.4 Wheel load distribution perpendicular to deck span.

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. For analysis purposes, the deck is assumed to act as a simple span between beams. 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.

2. *Distribution for shear.* Live-load vertical shear is computed by placing the edge of the wheel load distribution width b_t a distance from the support equal to the deck thickness t. AASHTO does not specify the distribution width for shear, but it is common practice to assume that the entire panel width is effective for horizontal shear.

Deck Configuration. The performance and economy of glulam timber deck panels can be affected significantly 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 considerations for curb or railing systems with attachment points on every second or third panel. 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.

10.1.5 Deck Attachment

Glulam timber decks are attached to supporting glulam timber beams with mechanical fasteners such as bolts or lag screws. The attachments must hold the panels securely and transmit longitudinal and transverse forces from the deck to the beams. They also should 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. Large deflections cause attachments to loosen from vibrations and from panel rotation about the support. The larger the deflection, the more is significant the effect. Acceptable panel deflection is difficult to quantify and should be based on the best judgment of the designer. Recommended maximum deck deflections given in this section should provide acceptable attachment performance.

Glulam timber decks are placed directly on beams without material at the deck-beam interface. 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 glulam timber beam in a routed slot (Fig. 10.5). 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 timber suppliers and manufacturers, is the preferred attachment for glulam timber 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 (Fig. 10.6). 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

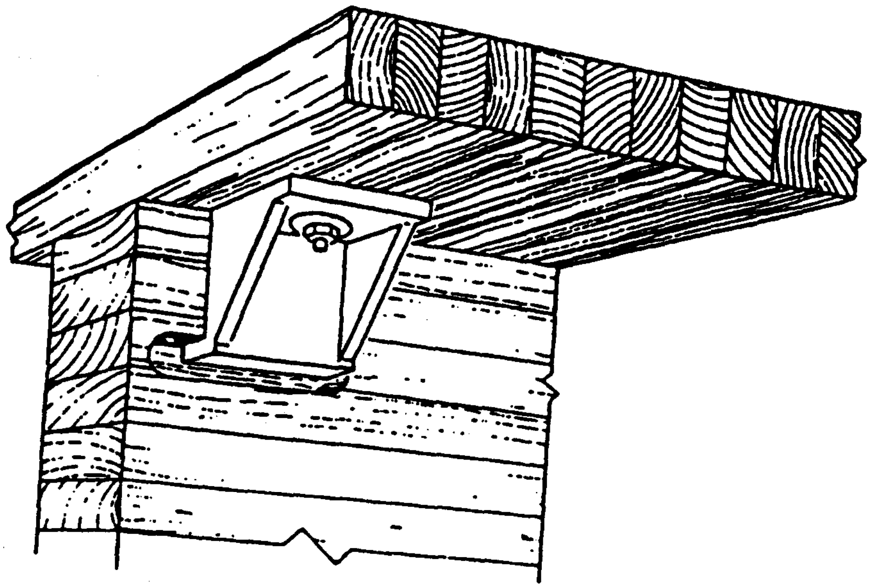


FIG. 10.5 Aluminium deck bracket for attaching glulam decks to glulam beams.

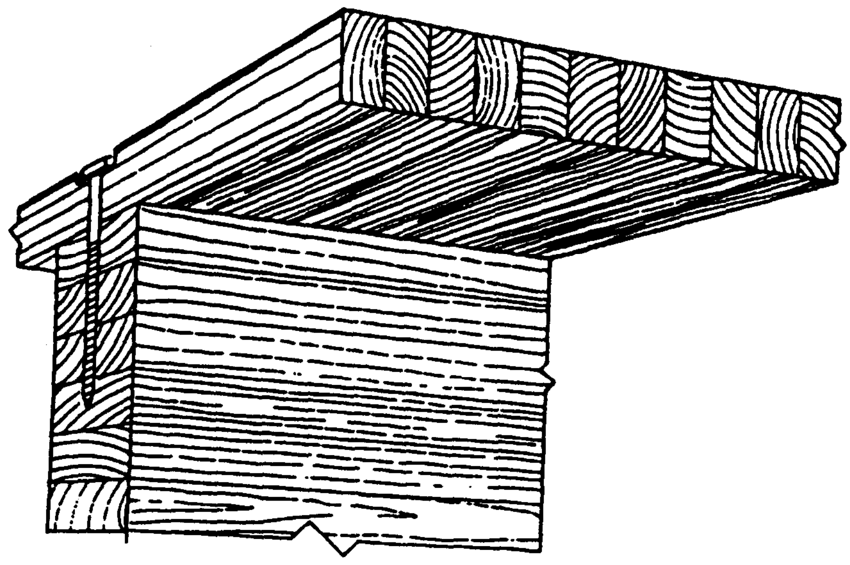


FIG. 10.6 Lag screw connection for attaching glulam decks to glulam beams.

boring increases the susceptibility to beam and deck decay, and they are not accessible for tightening if the deck is paved.

10.1.6 Design Examples

Sequential design examples are included in this section to familiarize the reader with the design procedures and requirements for glulam timber beam bridges with transverse glulam timber decks. The order of the examples is based on the most common design sequence and may vary for different applications. Specific site requirements and criteria are noted for each example. In addition, the following general criteria related to loads, materials, live-load deflection, and conditions of use are applicable.

Loads. Loads are based on the AASHTO Specification load requirements. 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. For wood deck design, AASHTO special provisions for HS 20-44 and H 20-44 loads allow the use of one-axle loads of 24,000 lb (106.8 kN) or two-axle loads of 16,000 lb (71.17 kN) each, spaced 4 ft (1.219 m) apart, instead of the standard 32,000-lb (142.3-kN) axle. For most glulam timber decks, the deck panel width is 4 ft or less and the single 24,000-lb axle load [12,000-lb (53.38-kN) wheel load] is used.

Materials. Tabulated values for glulam timber are taken from the AASHTO Specifications.' Glulam timber material specifications are given by combination symbol; however, glulam timber also can be specified by required design values. Visually graded combination symbols are used most often, with provisions for E-rated substitution at the option of the manufacturer. *All timber components are assumed to be properly pressure-treated with an oil-type preservative after fabrication.*

Live-Load Deflection. AASHTO Specifications' do not require a limit for deflection but recommend that the live-load deflection not exceed l/500 of the bridge span. Although it is recommended that these deflection guidelines be followed, deflection criteria should be based on specific design circumstances and are left to designer judgment. AASHTO does not give a recommended live-load deflection for glulam timber decks. In this section, deck deflection will be limited to a maximum value of 0.10 in (2.54 mm).

Conditions of Use. Tabulated values for glulam timber components must be adjusted for specific use conditions by all applicable adjustment factors given in the AASHTO Specifications.' The following criteria for adjustment factors have been used in this section.

1. *Duration of load.* Beam and deck design for combined dead load and vehicle live load are based on the 2-month load duration specified in AASHTO. Applicable tabulated design values are multiplied by a load-duration factor C_D of 1.15.

2. *Moisture content.* With the exception of glulam timber beams covered by a watertight deck, all design values for bridge components are adjusted for wet-use conditions. Based on industry recommendations, covered glulam timber 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 assumptions that a watertight deck sufficiently protects glulam timber beams from the elements and that superficial surface wetting does not cause significant increases in beam moisture content except at supports.

3. ***Temperature effects and fire-retardant treatment.*** Conditions requiring adjustments for temperature or fire-retardant treatment are rare in bridge applications. Design examples in this chapter do not include modification factors for temperature effect C_t or fire-retardant treatment C_R. The use of fire-retardant treatments is not permitted with glulam timber.

EXAMPLE 10.1: SIMPLE-SPAN GLULAM TIMBER BEAM BRIDGE, TWO-LANE HIGHWAY LOADING Design a glulam timber beam bridge for a length of 92 ft (28.04 m) with a center-to-center beam span of 90.5 ft (27.58 m). The bridge will have a transverse noninterconnected glulam timber deck and will carry two lanes of AASHTO HS 20-44 loading in 12 ft (3.658-m) lanes. The following provisions apply:

1. The deck wearing surface is a 3-in (76.2-mm) layer of asphalt pavement with a waterproof membrane.

2. Post and beam vehicle railing is provided with a dead load of 55 lb/ft (802 N/m). The railing extends approximately 1 ft (0.3048 m) inward from the post, and the post spacing must be 8 ft (2.438 m) or less.

3. The beam live-load deflection shall not exceed 1/500 of the beam span ($L/500$). Deck live-load deflection shall not exceed 0.10 in (2.54 mm).

4. Glulam timber is manufactured from visually graded Douglas fir.

SOLUTION: From the information given, a configuration of five beams spaced 5 ft (1.524 m) on center is selected from Table 10.3. An out-to-out deck width of 26 ft (7.925 m) will be used to accommodate the two 12-ft (3.658 m) traffic lanes and railing width (Fig. 10.7).

Beam Design. Beam design is an interactive process. 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 group I. Transverse or longitudinal loads may be significant in some cases and also should be checked.

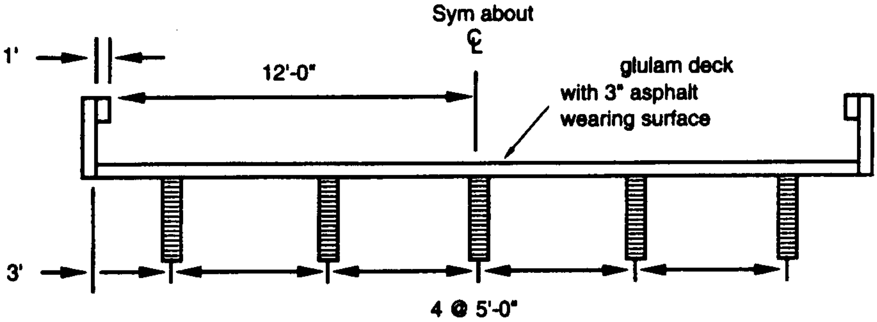


FIG. 10.7 Bridge cross section.

Select a Beam Combination Symbol. Several glulam timber combination symbols commonly used for bridge beams are given in Table 10.4. For this design, combination symbol 24F-V4, manufactured from visually graded Douglas fir, is selected for the beams. Tabulated values from the AASHTO Specifications' are as follows:

$$F_{bx} = 2{,}400 \text{ lb/in}^2 \text{ (16.55 MPa)}$$

$$F_{c\perp x} = 650 \text{ lb/in}^2 \text{ (4.481 MPa)}$$

$$F_{vx} = 165 \text{ lb/in}^2 \text{ (1.138 MPa)}$$

$$E_x = 1{,}800{,}000 \text{ lb/in}^2 \text{ (12.41 GPa)}$$

Determine Deck Dead Load and Dead-Load Moment. The dead load DL of the deck and wearing surface is computed in pounds per square foot based on AASHTO unit material weights of 50 lb/ft^3 (800.9 kg/m^3) for wood and 150 lb/ft^3 (2402 kg/m^3) for asphalt pavement. At this time, the deck thickness is unknown, so a thickness of 5 $^1/_8$ in is tentatively assumed:

$$DL = \frac{(5.125 \text{ in})(50 \text{ lb/ft}^3)}{12 \text{ in/ft}} + \frac{(3 \text{ in})(150 \text{ lb/ft}^3)}{12 \text{ in/ft}} = 59 \text{ lb/ft}^2$$

The deck dead load applied to each beam is equal to the tributary deck width supported by the beam. In this case, interior beams support 5 ft (1.524 m) of deck width. Outside beams support 5.5 ft (1.676 m) of deck plus 55 lb/ft (881.0 kg/m^3) of railing dead load.

For interior beams,

$$\text{Deck } W_{DL} = (5 \text{ ft})(59 \text{ lb/ft}^2) = 295 \text{ lb/ft}$$

$$\text{Deck } M_{DL} = \frac{W_{DL}L^2}{8} = \frac{(295 \text{ lb/ft})(90.5 \text{ ft})^2}{8} = 302{,}015 \text{ ft·lb}$$

For outside beams,

$$\text{Deck } W_{DL} = (5.5 \text{ ft})(59 \text{ lb/ft}^2) + 55 \text{ lb/ft} = 380 \text{ lb/ft}$$

$$\text{Deck } M_{DL} = \frac{(380 \text{ lb/ft})(90.5 \text{ ft})^2}{8} = 389{,}037 \text{ ft·lb}$$

TABLE 10.4 Glulam Timber Combination Symbols Commonly Used for Bridge Beams

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

Determine Live-Load Distribution for Moment. The live-load distribution factor *DF* for moment is determined in truck wheel lines *WL* per beam. From AASHTO tables (see Table 10.2), the distribution factor for interior beams is *S*/5.0, where *S* is the average beam spacing in feet:

$$DF_{\text{interior}} = \frac{S}{5} = \frac{5 \text{ ft}}{5} = 1.0WL/\text{beam}$$

For outside beams, the live-load distribution factor is determined as the reaction at the outside beam, assuming that the truck wheel line is 2 ft (0.610 m) from the rail face and the deck acts as a simple span between supports (Fig. 10.8). By examination, $DF_{\text{outside}} = 1.0$ *WL*/beam.

Determine Live-Load Moment. The maximum moment for one wheel line of the design vehicle is computed by statics or obtained from design tables given in the AASHTO specifications' and other publications. For a span of 90.5 ft (27.58 m), the maximum HS 20-44 moment is 676,665 ft·lb. The live-load moment per beam is determined by multiplying the maximum moment for one wheel line by the distribution factor *DF*. For interior and outside beams,

$$M_{LL} = 676{,}670 \text{ ft·lb } (1.0W\,L/\text{beam}) = 676{,}670 \text{ ft·lb}$$

Determine Beam Size Based on Bending. Beams must be designed to satisfy the following requirement:

$$F'_b \leq \frac{M}{S_x} \qquad \text{or} \qquad S_x \geq \frac{M}{F'_{bx}}$$

The allowable beam bending stress F'_b is determined by multiplying the tabulated

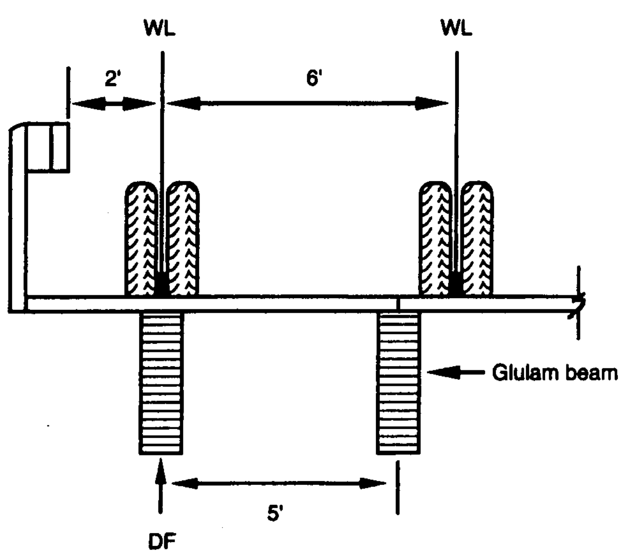


FIG. 10.8 Wheel load placement for outside beam.

bending stress F_{bx} by the wet-service factor C_M, load-duration factor C_D, and the more restrictive of the volume factor C_v or beam-stability factor C_L:

$$F'_b = F_{bx}C_MC_D(C_v \text{ or } C_L)$$

In most cases, the volume factor will control in bridge design. Thus the beam will be designed initially assuming that C_v controls, and then C_L will be checked. Per the AASHTO Specifications,[1] C_D = 1.15 will be used for the combination of vehicle live load and dead load. Because the glulam timber deck and asphalt wearing surface provide an essentially watertight system, protecting the beams from direct exposure to the elements, C_M = 1.0.

$$F'_b = F_{bx}C_MC_DC_v = 2400 \text{ lb/in}^2(1.15)(1.0)(C_v) = 2760 \text{ lb/in}^2(C_v)$$

$$C_v = (21/L)^{1/x}(12/d)^{1/x}(5.125/b)^{1/x} \le 1.0$$

where x = 10 in C_v calculations for Douglas fir.

Initial beam design will be based on the outside beams, which have a greater required moment capacity. Beam size is unknown, and values for C_v and beam dead load must be estimated. An iterative process is followed to arrive at an acceptable beam size.

$$M = 389{,}037 \text{ ft·lb} + 676{,}670 \text{ ft·lb} + \text{beam } M_{DL} = 1{,}065{,}707 \text{ ft·lb} + \text{beam } M_{DL}$$

Generally, a beam depth-to-width ratio of 7:1 or less is preferable for bridge applications. A design aid for estimating initial values for C_v and beam weight is given in Fig. 10.9.

Assuming a beam width of 10¾ in (273 mm), C_v = 0.67 per Fig. 10.7 for span of 90.5 ft, and a beam for western species glulam. Similar design charts can be developed for southern pine or other species which may have a different exponent for x in the volume effect equation weight of 250 lb/ft, an initial beam size is determined:

$$F'_b = 2760(0.67) = 1849 \text{ lb/in}^2$$

$$\text{Beam } M_{DL} = \frac{(250 \text{ lb/ft})(90.5)^2}{8} = 255{,}945 \text{ ft·lb}$$

$$M = 1{,}065{,}707 + 255{,}945 = 1{,}321{,}652 \text{ ft·lb}$$

$$S_x \text{ required} = \frac{(1{,}321{,}652 \text{ ft·lb})(12 \text{ in/ft})}{1849 \text{ lb/in}^2} = 8578 \text{ in}^3$$

Entering the glulam timber beam tables (Reference Data Table A.3) for western species, a beam size of 10¾ × 70.5 in (273 × 1791 mm) provides an S_x value of

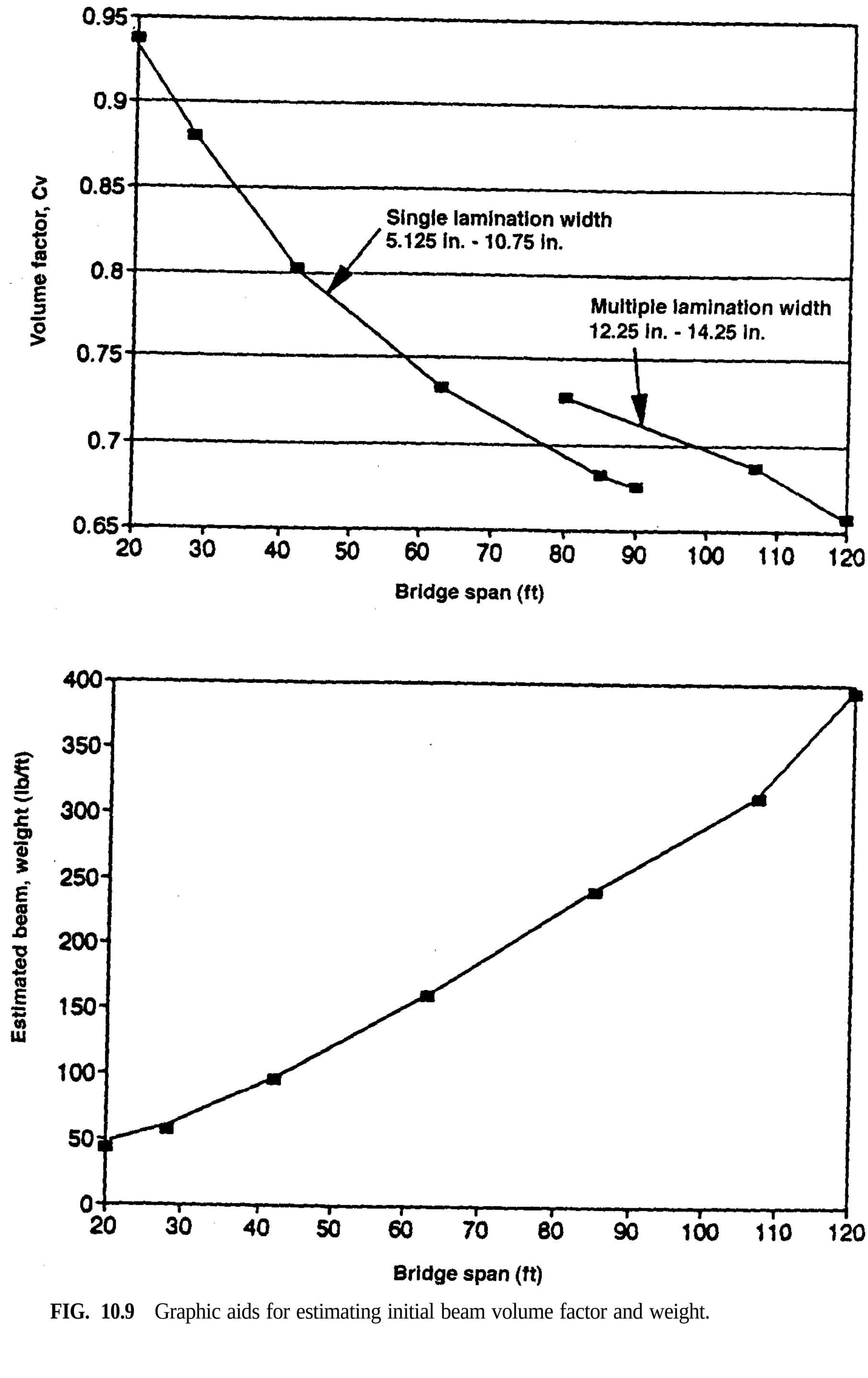


FIG. 10.9 Graphic aids for estimating initial beam volume factor and weight.

8905 in^3 and a beam weight of 263 lb/ft. Values for F'_b and M are computed and the beam size is checked:

$$C_V = (21/90.5)^{1/10}(12/70.5)^{1/10}(5.125/10.75)^{1/10} = 0.67$$

$$F'_b = 2760 \text{ ft·lb}(C_V) = 2760(0.67) = 1849 \text{ lb/in}^2$$

$$M_{DL} = \frac{(263)(90.5)^2}{8} = 269{,}254 \text{ ft·lb}$$

$$M_{TL} = 1{,}065{,}707 \text{ ft·lb} + 269{,}254 \text{ ft·lb} = 1{,}334{,}961 \text{ ft·lb}$$

$$S_x \text{ required} = \frac{(1{,}334{,}961 \text{ ft·lb})(12 \text{ in/ft})}{1849 \text{ in}^3} = 8664 \text{ in}^3$$

Examining the glulam timber beam tables (Table A.3), the next lower beam depth of 69 in will not meet design requirements. Therefore, a 10¾- × 70½-in (273 × 1790 mm) beam is selected, and the actual bending stress is computed:

$$f_b = \frac{(1{,}334{,}961 \text{ ft·lb})(12 \text{ in/ft})}{8905 \text{ in}^3} = 1799 \text{ lb/in}^2 \leq F'_b = 1849 \text{ lb/in}^2$$

The selected beam size is next evaluated for interior beam loading:

$$M = 302{,}015 + 676{,}670 + 269{,}254 = 1{,}247{,}939 \text{ ft·lb}$$

$$S_x \text{ required} = \frac{(1{,}247{,}939 \text{ ft·lb})(12 \text{ in/ft})}{1849 \text{ lb/in}^2} = 8099 \text{ in}^3$$

From the glulam timber beam tables (Table A.3), the beam depth for interior beams could be reduced by two laminations to a 10¾- by 67½-in (273 × 1714 mm) member. If this is done, provisions must be made at the bearings to account for the depth difference between interior and outside beams. In this case, a uniform beam depth is considered more economical, and a 10¾- by 70½-in (273 × 1790 mm) beam will be used for all beams. Bending stress for interior beams is computed:

$$f_b = \frac{(1{,}247{,}939 \text{ ft·lb})(12 \text{ in/ft})}{8905 \text{ in}^3} = 1682 \text{ lb/in}^2 \leq F'_b = 1849 \text{ lb/in}^2$$

The beam must next be checked for lateral stability using AASHTO requirements. Assuming a maximum spacing between points of lateral support, transverse beam bracing will be provided at the bearings and at the quarter points.

$$\ell_u = \frac{L}{4} = \frac{90.5 \text{ ft}}{4} = 22.63 \text{ ft}$$

$$\frac{\ell_u}{d} = \frac{(22.63 \text{ ft})(12 \text{ in/ft})}{70.5 \text{ in}} = 3.85 \qquad \frac{\ell_u}{d} < 7$$

$$\ell_e = 2.06\ell_u = (2.06)(22.63 \text{ in})(12 \text{ in/ft}) = 559 \text{ in}$$

$$R_B = \sqrt{\frac{(\ell_e)(d)}{b^2}} = \sqrt{\frac{(559 \text{ in})(70.5 \text{ in})}{(10.75 \text{ in})^2}} = 18.47$$

The beam stability factor C_L is determined using $K_{bE} = 0.609$ for glulam timber. As with bending stress, $C_M = 1.0$ is assumed for E.

$$C_L = \frac{1 + (F_{bE}/F_b^*)}{1.90} - \sqrt{\frac{[1 + (F_{bE}/F_b^*)]^2}{3.61} - \frac{(F_{bE}/F_b^*)}{0.95}}$$

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

$$F_{bE} = \frac{K_{bE}E'}{(R_B)^2} = \frac{(0.609)(1{,}800{,}000 \text{ lb/in}^2)}{(18.47)^2} = 3213 \text{ lb/in}^2$$

$$\frac{F_{bE}}{F_b^*} = \frac{3213 \text{ lb/in}^2}{2760 \text{ lb/in}^2} = 1.16$$

Thus

$$C_L = \frac{1 + 1.16}{1.90} - \sqrt{\frac{(1 + 1.1)^2}{3.61} - \frac{1.16}{0.95}} = 0.87$$

As assumed, the volume factor $C_v = 0.67$ will control over the beam stability factor $C_L = 0.87$.

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. Deflection can be computed through static analysis or from coefficients provided in design tables. Using a deflection coefficient DC from Ritter (1990),[6] the beam moment of inertia I_x and deflection are computed for one HS 20-44 wheel line on a span of 90.5 ft (27.6 m):

$$I_x = \frac{bd^3}{12} = \frac{(10.75 \text{ in})(70.5 \text{ in})^3}{12} = 313{,}902 \text{ in}^4$$

$$\Delta_{LL} = \frac{DC}{E'I_x} = \frac{9.06 \times 10^{11}}{E'I_x} = \frac{9.06 \times 10^{11}}{(1{,}800{,}000 \text{ lb/in}^2)(313{,}903 \text{ in}^4)}$$

$$= 1.60 \text{ in } (40.6 \text{ mm}) = L/679$$

$L/679 < L/500$, so live-load deflection is acceptable.

Check Horizontal Shear. From bending calculations, the total dead load for outside beams is 380 lb/ft (5.55 kN/m) for the deck and railing and 263 lb/ft (3.48 kN/m) for the beam, for a total of 643 lb/ft (9.39 kN/m). Neglecting loads within

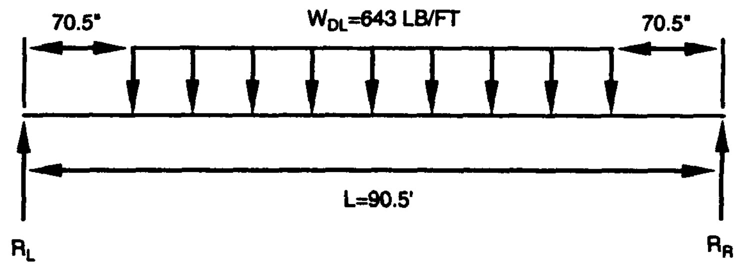


FIG. 10.10 Dead load placement for vertical shear.

a distance d = 70.5 in (1790 mm) from the supports, dead-load vertical shear is computed (Fig. 10.10):

$$V_{DL} = w_{DL}\left(\frac{L}{2} - d\right) = 643 \text{ lb/ft}\left(\frac{90.5 \text{ ft}}{2} - \frac{70.5 \text{ in}}{12 \text{ in/ft}}\right) = 25{,}318 \text{ lb}$$

Live-load vertical shear is computed from the maximum vertical shear occurring at the lesser of $3d$ or $L/4$ from the support. $3d$ controls (see below), and maximum vertical shear at that location due to one wheel line of an HS 20-44 truck (V_{LU}) is determined (Fig. 10.11):

$$3d = \frac{3(70.5 \text{ in})}{12 \text{ in/ft}} = 17.63 \text{ ft} \qquad \frac{L}{4} = \frac{90.5 \text{ ft}}{4} = 22.63 \text{ ft}$$

$$V_{LU} = R_L = 25{,}270 \text{ lb}$$

The AASHTO Specifications' require that live-load vertical shear be based on the maximum vertical shear due to undistributed wheel loads V_{LU} and wheel loads distributed laterally as specified for moment V_{LD}, as given by the Eq. (10.1). In this case, the distribution factor for moment equals 1.0, so the values for both V_{LU} and V_{LD} are 25,270 lb.

$$V_{LL} = 0.50[(0.60)(25{,}270 \text{ lb}) + 25{,}270 \text{ lb}] = 20{,}216 \text{ lb}$$

$$\text{Total vertical shear} = V_{DL} + V_{LL} = 25{,}318 \text{ lb} + 20{,}216 \text{ lb} = 45{,}534 \text{ lb}$$

Stress in horizontal shear is computed in accordance with AASHTO requirements:

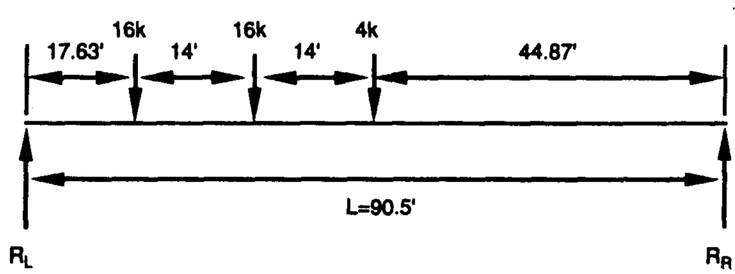


FIG. 10.11 Live load placement for vertical shear.

$$f_v = \frac{3V}{2bd} = \frac{(3)(45{,}534 \text{ lb})}{(2)(10.75 \text{ in})(70.5 \text{ in})} = 90 \text{ lb/in}^2$$

The allowable stress in horizontal shear is computed using $C_M = 1.0$ and $C_D = 1.15$:

$$F'_v = F_{vx} C_M C_D = (165 \text{ lb/in}^2)(1.0)(1.15) = 190 \text{ lb/in}^2$$

$F'_v = 190 \text{ lb/in}^2 > f_v = 90 \text{ lb/in}^2$, so the beam is satisfactory in horizontal shear.

Check Bearing Length and Stress. From the information provided, the bridge length is 92 ft (28.0 m) with a center-to-center beam span of 90.5 ft (27.6 m). Thus the bearing length at each beam end is 18 in. The bearing length and stress will be checked for the outside beams, which carry the greatest load.

For a unit dead load w_{DL} to outside beams of 380 lb/ft for the deck and railing and 263 lb/ft for the beam, the beam dead load reaction R_{DL} is computed:

$$R_{DL} = \frac{w_{DL} L}{2} = \frac{(380 \text{ lb/ft} + 263 \text{ lb/ft})(92 \text{ ft})}{2} = 29{,}578 \text{ lb}$$

The live-load beam reaction R_{LL} is the product of the maximum reaction for one wheel line of the design vehicle and the reaction *DF*. For outside beams, the reaction *DF* is the same as the moment *DF* of 1.0. From AASHTO Specification tables, the maximum reaction for one wheel line of an HS 20-44 truck on a span of 90.5 ft is 32,290 lb:

$$R_{LL} = R(DF) = (32{,}290 \text{ lb})(1.0) = 32{,}290 \text{ lb}$$

The bearing stress in compression perpendicular to grain $f_{c\perp}$ is computed for the bearing area *A*, which is the product of the beam width and the bearing length:

$$f_{c\perp} = \frac{R_{DL} + R_{LL}}{A} = \frac{(29{,}578 \text{ lb}) + (32{,}290 \text{ lb})}{(10.75 \text{ in})(18 \text{ in})} = 320 \text{ lb/in}^2$$

The allowable stress in compression perpendicular to grain $F'_{c\perp}$ is determined by multiplying the tabulated stress in compression perpendicular to grain $F_{c\perp x}$ by the wet-service factor C_M of 0.53:

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

$F'_{c\perp} = 345 \text{ lb/in}^2 > f_{c\perp} = 320 \text{ lb/in}^2$, so the bearing stress is acceptable. A bearing configuration shown in Fig. 10.12 will be used.

Determine Camber Per the AASHTO Specifications,' beam camber should be a minimum of three times the dead-load deflection but not less than ½ in. Dead-load deflection A,, will be based on $w_{DL} = 643$ lb/ft for outside beams, but the same camber will be placed in both interior and outside beams.

From the glulam timber beam tables (Table A.3), the moment of inertia I for a 10¾- by 70½-in glulam timber beam is 313,902 in^4. Beam dead-load deflection is computed by the following equation:

$$\Delta_{DL} = \frac{5(w_{DL})(L^4)}{384(E')(I_x)} = \frac{(5)(643 \text{ lb/ft})(90.5 \text{ ft} \times 12 \text{ in/ft})^4}{(384)(1{,}800{,}000 \text{ lb/in}^2)(313{,}902 \text{ in}^4)(12 \text{ in/ft})} = 1.72 \text{ in}$$

The beams will be cambered 5 (127 mm) in at midspan.

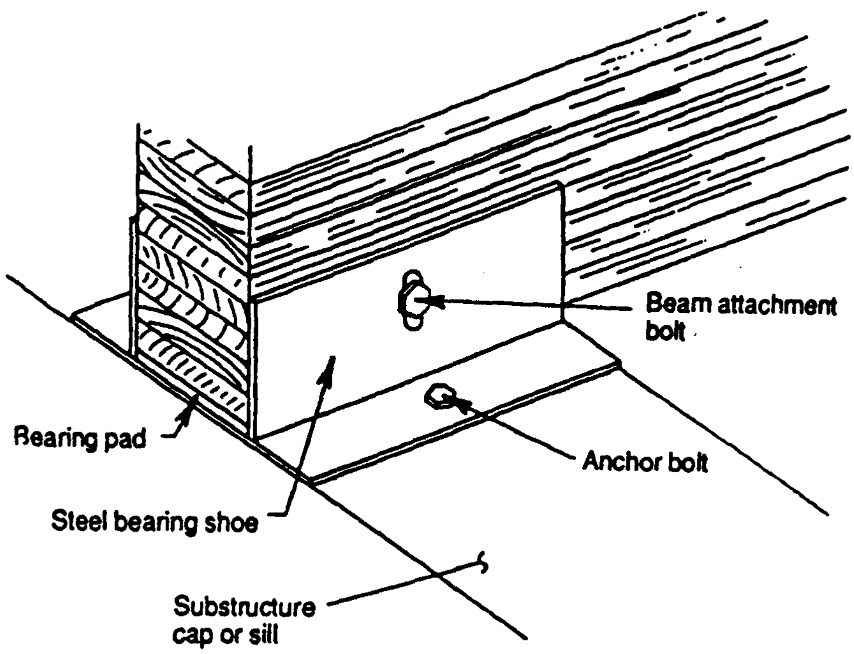


FIG. 10.12 Typical fixed-bearing configuration for glulam beams.

Deck Design. Noninterconnected glulam timber 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 typically designed for bending and then checked for deflection and shear. Although deflection rather than bending may control in many applications, the acceptable level of deflection is established by the designer and may vary for different applications.

Determine the Deck Span, Design Loads, and Panel Size. Based on previous assumptions, an initial deck thickness of $5^1/_8$ in will be used. Per the AASHTO Specifications,[1] the 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:

$$\text{Clear distance between beams} = 60 \text{ in} - 10.75 \text{ in} = 49.25 \text{ in}$$

$$s = 49.25 + \frac{10.75 \text{ in}}{2} = 54.63 \text{ in}$$

$$\text{Clearspan} + \text{deck thickness} = 49.25 \text{ in} + 5.125 \text{ in} = 54.38 \text{ in}$$

$s = 54.38$ in will be used for design.

For HS 20-44 loading, AASHTO special provisions apply, and the deck will be designed for a 12,000-lb wheel load. Panel width for an out-to-out bridge length of 92 ft (28.0 m) will be based on an alternating repetition of panels to allow standardized panel configurations. In this case, 48-in-wide (1.22 m) panels will be used. Rail posts will be placed at the center of end panels and at the center of every second panel for a post spacing of 8 ft (2.44 m) (Fig. 10.13).

Determine Wheel Distribution Widths and Effective Deck Section Properties. The wheel load distribution width in the direction of the deck span b_t is computed using Eq. (10.2):

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

Normal to the deck span, the wheel load distribution width b_d is computed using Eq. (10.3):

$$b_d = t + 15 \text{ in} = 5.125 \text{ in} + 15 \text{ in} = 20.125 \text{ in}$$

The deck will be designed with a beam of width b_d and depth t to resist the forces and deflection of one wheel load. Section modulus S_y, and moment of inertia I_y are computed using the following equations:

$$S_y = \frac{b_d t^2}{6} = \frac{(20.125 \text{ in})(5.125 \text{ in})^2}{6} = 88 \text{ in}^3 \ (1.44 \text{ Kcm}^3)$$

$$I_y = \frac{b_d t^3}{12} = \frac{(20.125 \text{ in})(5.125 \text{ in})^3}{12} = 226 \text{ in}^4 \ (9.41 \text{ Kcm}^4)$$

Determine Deck Dead Load. For a $5\,{}^1/_8$-in deck and 3-in asphalt wearing surface, dead-load unit weight w_{DL} and dead-load moment M_{DL} over the effective distribution width of 20.125 in are computed:

$$w_{DL} = 20.125 \text{ in}\left[\frac{(5.125 \text{ in})(50 \text{ lb/ft}^3) + (3 \text{ in})(150 \text{ lb/ft}^3)}{1728 \text{ in}^3/\text{ft}^3}\right] = 8.2 \text{ lb/in}$$

$$M_{DL} = \frac{w_{DL}s^2}{8} = \frac{(8.2 \text{ lb/in})(54.38 \text{ in})^2}{8} = 3031 \text{ in·lb}$$

Determine Live-Load Moment. For an effective deck span less than 122 in, maximum live-load moment M_{LL} is computed for a 6-ft track width and 12,000-lb wheel load by the following equation (see Ritter[6]):

$$M_{LL} = 3000s - 25{,}983 = 3000(54.38 \text{ in}) - 25{,}983 = 137{,}157 \text{ in·lb}$$

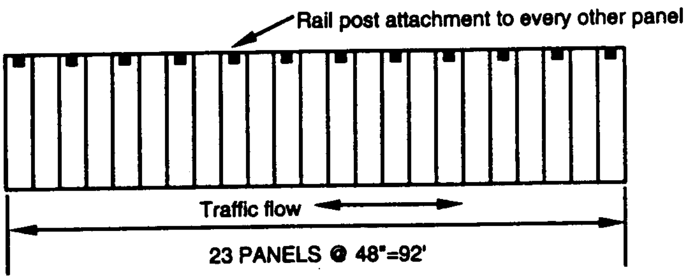


FIG. 10.13 Deck panel layout and rail post placement.

Compute Bending Stress and Select a Deck Combination Symbol. The deck is continuous over more than two spans, so bending stress f_b is based on 80 percent of the simple span moment:

$$M_{TL} = M_{LL} + M_{DL} = 3031 \text{ in·lb} + 137{,}157 \text{ in·lb} = 140{,}188 \text{ in·lb}$$

$$f_b = \frac{0.80M}{S_y} = \frac{0.80(140{,}188 \text{ in·lb})}{88 \text{ in}^3} = 1274 \text{ lb/in}^2$$

From the AASHTO glulam timber design tables for axial combinations, Douglas fir combination No. 2 is selected with the following tabulated values and wet-service factors C_M:

$$F_{by} = 1{,}800 \text{ lb/in}^2 \qquad C_M = 0.80$$

$$F_{vy} = 165 \text{ lb/in}^2 \qquad C_M = 0.875$$

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

From the AASHTO Specifications,' the allowable bending stress F'_b is equal to the tabulated bending stress F_{by} times the wet-service factor C_M, load-duration factor C_D, and bending size factor C_F:

$$F'_b = F_{by}C_DC_F$$

Per the AASHTO Specifications,' the value of C_D is 1.15 and C_F is computed by the following equation:

$$C_F = \left(\frac{12}{t}\right)^{1/9} = \left(\frac{12}{5.125 \text{ in}}\right)^{1/9} = 1.10$$

The allowable deck bending stress is computed:

$$F'_b = F_{by}C_MC_DC_F = (1800 \text{ 1b/in}^2)(0.80)(1.15)(1.10) = 1822 \text{ lb/in}^2$$

f_b = 1,274 lb/in^2 is substantially less than F'_b = 1,822 lb/in^2, and a lower grade glulam timber combination symbol may be feasible. However, no changes will be made until the live-load deflection is checked.

Check Live-Load Deflection. To determine the deck live-load deflection, the allowable modulus of elasticity E' is computed by multiplying the tabulated modulus of elasticity E_y by the wet-service factor C_M:

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

From Ritter (1990),[6] the live-load deflection for a 12,000-lb wheel load and 6-ft track width Δ_{WL} is computed by the following equation:

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

$$= \frac{1.80[(138.8)(54.38 \text{ in})^3 - (20{,}780)(54.38 \text{ in}) + 90{,}000)]}{(1{,}416{,}100 \text{ lb/in}^2)(226 \text{ in}^4)} = 0.12 \text{ in}$$

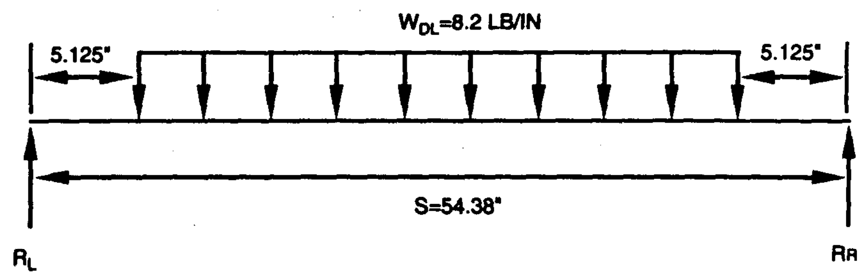


FIG. 10.14 Dead load placement for vertical shear.

The deck is continuous over more than two spans, and per AASHTO Specifications,[1] 80 percent of the simple span deflection is used to account for span continuity:

$$\Delta_{LL} = 0.80(0.12\ \text{in}) = 0.10\ \text{in}$$

The computed deflection of 0.10 in equals the maximum allowable, so the deck thickness and combination symbol are acceptable for live-load deflection. A reduction in the glulam timber combination symbol grade will reduce E', which will result in greater deck deflection. Therefore, a combination symbol No. 2 will be retained.

Check Horizontal Shear. Dead-load vertical shear is computed at a distance from the support equal to the deck thickness t, and loads acting within the distance t are neglected. For w_{DL} = 8.2 lb/in (Fig. 10.14),

$$V_{DL} = w_{DL}\left(\frac{s}{2} - t\right) = 8.2\ \text{lb/in}\left(\frac{54.38\ \text{in}}{2} - 5.125\ \text{in}\right) = 181\ \text{lb}$$

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-lb wheel load acts through the center of the distribution width, and live-load vertical shear V_{LL} is computed by statics (Fig. 10.15):

$$V_{LL} = R_L = \frac{(12{,}000\ \text{lb})(8.66\ \text{in} + 31.94\ \text{in})}{54.38\ \text{in}} = 8{,}959\ \text{lb}$$

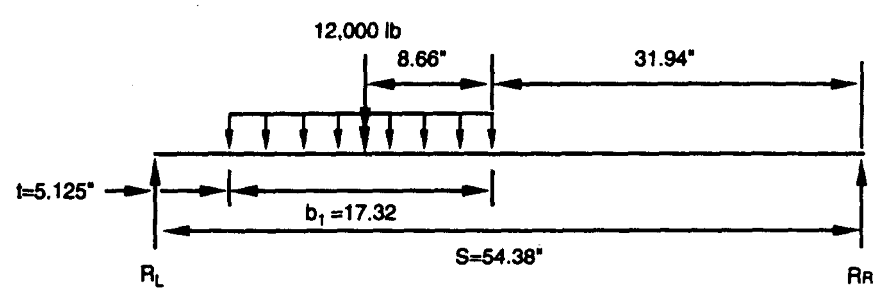


FIG. 10.15 Live load placement for vertical shear.

The applied stress in horizontal shear f_v is computed using AASHTO equations, assuming that the entire panel cross-sectional area A_p is effective in shear distribution:

$$V_{TL} = V_{DL} + V_{LL} = 181 \text{ lb} + 8959 \text{ lb} = 9140 \text{ lb}$$

$$A_p = (\text{panel width})(t) = (48 \text{ in})(5.125 \text{ in}) = 246 \text{ in}^2$$

$$f_v = \frac{1.5V}{A_p} = \frac{(1.5)(9140 \text{ lb})}{246 \text{ in}^2} = 56 \text{ lb/in}^2$$

Per AASHTO requirements, the allowable stress in horizontal shear F'_v is equal to the tabulated stress in horizontal shear F_{vy} times the wet-service factor C_M and the load-duration factor C_D. Values of C_M and C_D are 0.875 and 1.15, respectively.

$$F'_v = F_{vy}C_MC_D = (165 \text{ lb/in}^2)(0.875)(1.15) = 166 \text{ lb/in}^2$$

$f_v = 56$ lb/in^2 < $F'_v = 166$ lb/in^2, so the panel is satisfactory in horizontal shear.

Summary. The bridge superstructure will consist of five Douglas fir glulam timber beams, combination symbol 24F-V4. The beams will measure 10¾ in (273 mm) wide, 70½ in (1790 mm) deep, and 92 ft (28.04 m) long. The span center to center of bearings will be 90½ ft (27.58 m). Transverse bracing will be provided for lateral support at the bearings and at the beam quarter points. The deck will consist of 23 combination No. 2 Douglas fir glulam timber deck panels measuring 5 $^1/_8$ in (130.2 mm) thick by 48 in (1.219 m) wide by 26 ft (7.925 m) long. Stresses and live load deflection are given in Table 10.5.

EXAMPLE 10.2: SIMPLE-SPAN GLULAM TIMBER BEAM BRIDGE, ONE-LANE HIGHWAY LOADING Design a glulam timber beam bridge with a span of 47 ft (14.32 m) measured center to center of bearings and a length of 48 ft (14.63 m). The bridge will have a transverse noninterconnected glulam timber deck and will carry AASHTO HS 20-44 loading in a single lane, 14 ft (4.267 m) wide. The following provisions apply:

1. The deck wearing surface is a 3-in (76.2-mm) layer of wood plank.

TABLE 10.5 Summary of Design Values for Example 10.1

Design value	Outside beams	Interior beams	Deck
f_b	1799 lb/in^2	1682 lb/in^2	1274 lb/in^2
F'_b	1849 lb/in^2	1849 lb/in^2	1822 lb/in^2
Δ_{LL}	1.60 in = $L/679$	1.60 in = $L/679$	0.10 in
f_v	90 lb/in^2	—	56 lb/in^2
F'_v	190 lb/in^2	—	166 lb/in^2
$f'_{c\perp}$	320 lb/in^2	—	N/A
$F'_{c\perp}$	345 lb/in^2	—	N/A
Δ_{DL}	1.72 in	—	N/A
Camber	6 in	6 in	N/A

Note: 1 in = 25.4 mm; 1 lb/in^2 = 6.895 kPa.

2. Post and beam vehicle railing is provided with a dead load of 45 lb/ft (656.7 N/m). The railing extends approximately 1 ft (0.3048 m) inward from the deck edge.

3. The beam live-load deflection shall not exceed 1/360 of the beam span ($L/360$). Deck live-load deflection shall not exceed 0.10 in (2.54 mm).

4. Glulam timber is manufactured from visually graded southern pine.

SOLUTION: From the given information, a configuration of three beams spaced 5.5 ft on center is selected from Table 10.3. An out-to-out deck width of 16 ft will be used to accommodate the traffic lane and railing width (Fig. 10.16).

Beam Design. Beam design will follow the same basic procedure illustrated in the preceding example.

Select a Beam Combination Symbol. A southern pine combination symbol 24F-V3 is initially selected for the beams. Tabulated values and wet-service factors are obtained from the AASHTO specifications':

$$F_{bx} = 2400 \text{ lb/in}^2 \text{ (16.55 MPa)} \qquad C_M = 0.80$$

$$F_{c\perp x} = 650 \text{ lb/in}^2 \text{ (4.481 MPa)} \qquad C_M = 0.53$$

$$F_{vx} = 200 \text{ lb/in}^2 \text{ (1.380 MPa)} \qquad C_M = 0.875$$

$$E_x = 1{,}800{,}000 \text{ lb/in}^2 \text{ (12.41 GPa)} \qquad C_M = 0.833$$

Determine Deck Dead Load and Dead-Load Moment. The dead load DL of the deck and wearing surface is computed in pounds per square foot based on the AASHTO unit material weight for wood of 50 lb/ft^3. At this time, the deck thickness is unknown, so a thickness of 6¾ in is assumed:

$$DL = \frac{(3 \text{ in} + 6.75 \text{ in})(50 \text{ lb/ft}^3)}{12 \text{ in/ft}} = 41 \text{ lb/ft}^2$$

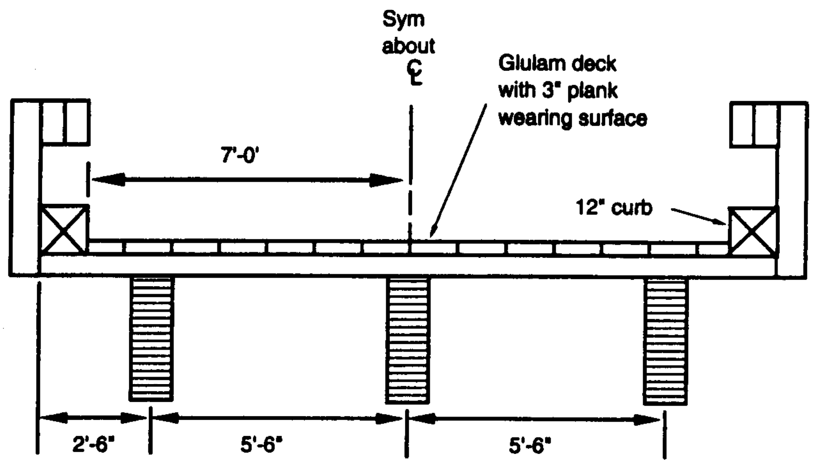


FIG. 10.16 Bridge cross section.

The deck dead load applied to each beam is equal to the tributary deck width supported by the beam. The interior beam supports 5.5 ft of deck width. Outside beams support 5.25 ft of deck plus 45 lb/ft of railing dead load.

For the interior beam,

$$\text{Deck } w_{DL} = (5.5 \text{ ft})(41 \text{ lb/ft}^2) = 226 \text{ lb/ft}$$

$$\text{Deck } M_{DL} = \frac{w_{DL}L^2}{8} = \frac{(226 \text{ lb/ft})(47 \text{ ft})^2}{8} = 62{,}404 \text{ ft·lb}$$

For outside beams,

$$\text{Deck } w_{DL} = (5.25 \text{ ft})(41 \text{ lb/ft}^2) + 45 \text{ lb/ft} = 260 \text{ lb/ft}$$

$$\text{Deck } M_{DL} = \frac{(260 \text{ lb/ft})(47 \text{ ft})^2}{8} = 71{,}793 \text{ ft·lb}$$

Determine Live-Load Distribution Factors and Live-Load Moment. From Table 10.2, the live-load distribution factor for both the interior beams and the outside beams is 0.92 *WL*/beam. From AASHTO Specification tables, the maximum moment for one wheel line of an HS 20-44 truck on a span 47 ft long is 287,170 ft·lb. The live-load moment per beam is determined by multiplying the maximum moment for one wheel line by the distribution factor *DF*. For interior and outside beams,

$$M_{LL} = 287{,}170 \text{ ft·lb } (0.92WL/\text{beam}) = 264{,}196 \text{ ft·lb}$$

Determine Beam Size Based on Bending. The allowable beam bending stress F'_b is determined by multiplying the tabulated bending stress F_{bx} by the wet-service factor C_M, load-duration factor C_D, and the more restrictive of the volume factor C_v or beam stability factor C_L:

$$F'_b = F_{bx}C_MC_D(C_V \text{ or } C_L)$$

Per the AASHTO Specifications,[1] C_D = 1.15 will be used for the combination of vehicle live load and dead load. Because the glulam timber deck has a wood plank wearing surface, a wet-use value of $C_M = 0.80$ will be used:

$$F'_b = F_{bx}C_MC_DC_V = 2400 \text{ lb/in}^2 \ (1.15)(0.80)(C_V) = 2208 \text{ lb/in}^2 \ (C_V)$$

$$C_V = (21/L)^{1/x}(12/d)^{1/x}(5.125/b)^{1/x} \leq 1.0$$

where $x = 20$ in C_v calculations for southern pine.

Initial beam design will be based on the outside beams, which have a slightly greater required moment capacity:

$$M_{TL} = 71{,}793 \text{ ft·lb} + 264{,}196 \text{ ft·lb} + \text{beam } M_{DL} = 335{,}989 \text{ ft·lb} + \text{beam } M_{DL}$$

Assume a beam width of 8½ in, $C_v = 0.78$, and a beam weight of 120 lb/ft. An initial beam size is computed:

$$F'_b = 2{,}208(0.78) = 1722 \text{ lb/in}^2$$

$$\text{Beam } M_{DL} = \frac{(120 \text{ lb/ft})(47)^2}{8} = 33{,}135 \text{ ft·lb}$$

$$M = 335{,}989 + 33{,}135 = 369{,}124 \text{ ft·lb}$$

$$S_x \text{ required} = \frac{(369{,}124 \text{ ft·lb})(12 \text{ in/ft})}{1722 \text{ lb/in}^2} = 2572 \text{ in}^3$$

Entering the glulam timber beam tables for southern pine, a beam size of 8½ × 42⅝ in provides an S_x value of 2574 in^3 and a beam weight of 126 lb/ft. Values for F'_b and M are computed, and the beam size is checked:

$$C_V = (21/47)^{1/20}(12/42.625)^{1/20}(5.125/8.5)^{1/20} = 0.88$$

$$F'_b = 2208(C_V) = 2208(0.88) = 1943 \text{ lb/in}^2$$

$$M_{DL} = \frac{(126)(47)^2}{8} = 34{,}792 \text{ ft·lb}$$

$$M_{TL} = 335{,}989 \text{ ft·lb} + 34{,}792 \text{ ft·lb} = 370{,}781 \text{ ft·lb}$$

$$S_x \text{ Required} = \frac{(370{,}781 \text{ ft·lb})(12 \text{ in/ft})}{1943 \text{ in}^3} = 2290 \text{ in}^3$$

Entering the glulam timber beam tables, a revised smaller beam size of 8½ × 41¼ in will meet design requirements with an S_x value of 2411 in^3 and beam weight of 122 lb/ft:

$$C_V = (21/47)^{1/20}(12/41.25)^{1/20}(5.125/8.5)^{1/20} = 0.88$$

$$F'_b = 2208 \text{ ft·lb}(C_V) = 2208(0.88) = 1943 \text{ lb/in}^2$$

$$M_{DL} = \frac{(122)(47)^2}{8} = 33{,}687 \text{ ft·lb}$$

$$M_{TL} = 335{,}989 \text{ ft·lb} + 33{,}687 \text{ ft·lb} = 369{,}676 \text{ ft·lb}$$

$$S_x \text{ required} = \frac{(369{,}676 \text{ ft·lb})(12 \text{ in/ft})}{1943 \text{ lb/in}^2} = 2283 \text{ in}^3$$

Examining the glulam timber beam tables, the next lower beam depth will not meet design requirements. Therefore, an 8½- × 41¼-in beam is selected, and the actual bending stress is computed:

$$f_b = \frac{(369{,}676 \text{ ft·lb})(12 \text{ in/ft})}{2411 \text{ in}^3} = 1840 \text{ lb/in}^2 \le F'_b = 1943 \text{ lb/in}^2$$

The beam must next be checked for lateral stability using the AASHTO Specification requirements. Assuming a maximum spacing between points of lateral support, transverse beam bracing will be provided at the bearings and at centerspan:

$$\ell_u = \frac{L}{2} = \frac{47\text{ ft}}{2} = 23.5\text{ ft}$$

$$\frac{\ell_u}{d} = \frac{(23.5\text{ ft})(12\text{ in/ft})}{41.25\text{ in}} = 6.84 \qquad \frac{\ell_u}{d} < 7$$

$$\ell_e = 2.06\ell_u = (2.06)(23.5\text{ ft})(12\text{ in/ft}) = 581\text{ in}$$

$$R_B = \sqrt{\frac{(\ell_e)(d)}{b^2}} = \sqrt{\frac{(581\text{ in})(41.25\text{ in})}{(8.5\text{ in})^2}} = 18.21$$

The beam stability factor C_L is determined using $K_{bE} = 0.609$ for glulam timber. As with bending stress, wet-use conditions ($C_M = 0.833$) are assumed for E.

$$C_L = \frac{1 + (F_{bE}/F_b^*)}{1.90} - \sqrt{\frac{[1 + (F_{bE}/F_b^*)]^2}{3.61} - \frac{(F_{bE}/F_b^*)}{0.95}}$$

$$E' = E_x C_M = 1{,}800{,}000\text{ lb/in}^2\,(0.833) = 1{,}499{,}400\text{ lb/in}^2$$

$$F_{bE} = \frac{K_{bE}E'}{(R_B)^2} = \frac{(0.609)(1{,}499{,}400\text{ lb/in}^2)}{(18.21)^2} = 2754\text{ lb/in}^2$$

$$\frac{F_{bE}}{F_b^*} = \frac{2754\text{ lb/in}^2}{2208\text{ lb/in}^2} = 1.25$$

Thus
$$C_L = \frac{1 + 1.25}{1.90} - \sqrt{\frac{[1 + 1.25]^2}{3.61} - \frac{1.25}{0.95}} = 0.89$$

The volume factor $C_v = 0.88$ will control over the beam stability factor $C_L = 0.89$.

Check Live-Load Deflection. The beam moment of inertia I_x is computed, and the deflection for one HS 20-44 wheel line Δ_{WL} is computed using a deflection coefficient DC from Ritter (1990).[6] The live-load deflection Δ_{LL} is the product of A, and the distribution factor DF:

$$I_x = \frac{bd^3}{12} = \frac{(8.5\text{ in})(41.25\text{ in})^3}{12} = 49{,}718\text{ in}^4$$

$$\Delta_{WL} = \frac{DC}{E'I_x} = \frac{1.09 \times 10^{11}}{E'I_x} = \frac{1.09 \times 10^{11}}{(1{,}499{,}400\text{ lb/in}^2)(49{,}718\text{ in}^4)} = 1.46\text{ in}$$

$$\Delta_{LL} = \Delta_{WL}(DF) = (1.46\text{ in})(0.92) = 1.34\text{ in }(34.0\text{ mm}) = L/421$$

$L/421 < L/360$, so live-load deflection is acceptable.

Check Horizontal Shear. From bending calculations, the total dead load for outside beams is 260 lb/ft for the deck and railing and 122 lb/ft for the beam, for a total of 382 lb/ft. Neglecting loads within a distance d = 41.25 in from the supports, dead-load vertical shear is computed (Fig. 10.17):

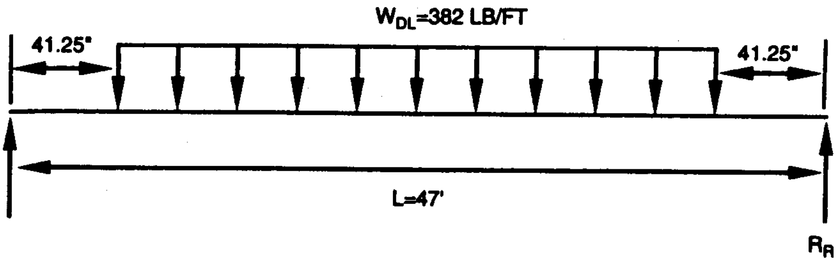


FIG. 10.17 Dead load placement for vertical shear.

$$V_{DL} = w_{DL}\left(\frac{L}{2} - d\right) = 382\ \text{lb/ft}\left(\frac{47\ \text{ft}}{2} - \frac{41.25\ \text{in}}{12\ \text{in/ft}}\right) = 7664\ \text{lb}$$

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

$$3d = \frac{3(41.25\ \text{in})}{12\ \text{in/ft}} = 10.31\ \text{ft} \qquad \frac{L}{4} = \frac{47\ \text{ft}}{4} = 11.75\ \text{ft}$$

3*d* = 10.31 ft controls, and maximum vertical shear at that location due to one wheel line of an HS 20-44 truck V_{LU} is determined (Fig. 10.18):

$$V_{LU} = R_L = 20{,}954\ \text{lb}$$

The distributed shear V_{LD} is the product of V_{LU} and the moment *DF*:

$$V_{LD} = V_{LU}(DF) = 20{,}954\ \text{lb}(0.92) = 19{,}278\ \text{lb}$$

$$V_{LL} = 0.50[(0.60)(20{,}954\ \text{lb}) + 19{,}278\ \text{lb}] = 15{,}925\ \text{lb}$$

$$\text{Total vertical shear} = V_{DL} + V_{LL} = 7664\ \text{lb} + 15{,}925\ \text{lb} = 23{,}589\ \text{lb}$$

Stress in horizontal shear is computed in accordance with AASHTO requirements. The allowable stress in horizontal shear is computed using $C_M = 0.875$ and $C_D = 1.15$:

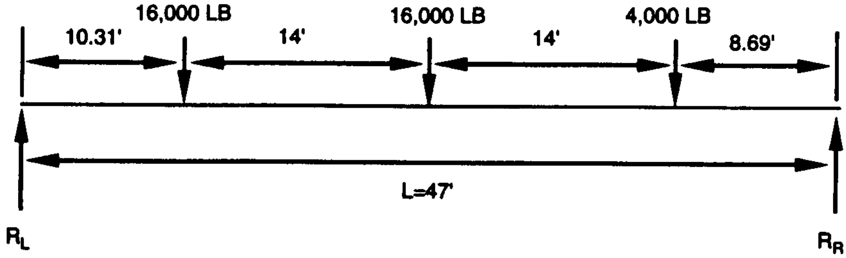


FIG. 10.18 Live load placement for vertical shear.

$$f_v = \frac{3V}{2bd} = \frac{(3)(23{,}589\ \text{lb})}{(2)(8.5\ \text{in})(41.25\ \text{in})} = 101\ \text{lb/in}^2$$

$$F'_v = F_{vx}C_MC_D = (200\ \text{lb/in}^2)(0.875)(1.15) = 201\ \text{lb/in}^2$$

F'_v = 201 lb/in^2 > f_v = 102 lb/in^2, so the beam is satisfactory in horizontal shear.

Check Bearing Length and Stress. From the information provided, the bridge length is 48 ft with a center-to-center beam span of 47 ft. Thus the bearing length at each beam end is 12 in. The bearing length and stress will be checked for the outside beams, which carry a slightly greater load.

For a unit dead load w_{DL} to outside beams of 260 lb/ft for the deck and railing and 122 lb/ft for the beam, the beam dead-load reaction R_{DL} is computed:

$$R_{DL} = \frac{w_{DL}L}{2} = \frac{(260\ \text{lb/ft} + 122\ \text{lb/ft})(48\ \text{ft})}{2} = 9168\ \text{lb}$$

From AASHTO Specification tables, the maximum reaction for one wheel line R_{WL} of an HS 20-44 truck is 28,850 lb. The live-load reaction R_{LL} is computed:

$$R_{LL} = R_{WL}(DF) = (28{,}850\ \text{lb})(0.92) = 26{,}542\ \text{lb}$$

Bearing stress in compression perpendicular to grain $f_{c\perp}$ is computed for the bearing area A:

$$f_{c\perp} = \frac{R_{DL} + R_{LL}}{A} = \frac{(9168\ \text{lb}) + (26{,}542\ \text{lb})}{(8.5\ \text{in})(12\ \text{in})} = 350\ \text{lb/in}^2$$

The allowable stress in compression perpendicular to grain $F'_{c\perp}$ is determined by multiplying the tabulated stress in compression perpendicular to grain $F_{c\perp x}$ by the wet-service factor C_M of 0.53:

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

$F'_{c\perp}$ = 345 lb/in^2 < $f_{c\perp}$ = 350 lb/in^2, so the bearing stress exceeds the allowable by approximately 1.5 percent. In most cases, this minor difference is acceptable and within roundoff error. However, the bearing length will be extended to 13 in. The minor change in span length will have a slight conservative effect on values previously computed but will be negligible. The applied stress is computed for the revised bearing length of 13 in:

$$f_{c\perp} = \frac{R_{DL} + R_{LL}}{A} = \frac{(9168\ \text{lb}) + (26{,}542\ \text{lb})}{(8.5\ \text{in})(13\ \text{in})} = 323\ \text{lb/in}^2$$

Determine Camber: Per the AASHTO Specifications,' beam camber will be a minimum of three times the dead-load deflection. Dead-load deflection A,, will be based on w_{DL} = 382 lb/ft for outside beams, but the same camber will be placed in the interior and outside beams.

$$\Delta_{DL} = \frac{5(w_{DL})(L^4)}{384(E')(I_x)} = \frac{(5)(382\ \text{lb/ft})(47\ \text{ft} \times 12\ \text{in/ft})^4}{(384)(1{,}499{,}400\ \text{lb/in}^2)(49{,}718\ \text{in}^4)(12\ \text{in/ft})} = 0.56\ \text{in}$$

The beams will be cambered 1¾ in (44.4 mm) at centerspan.

Deck Design. The deck will be assumed to act as a simple span between beams and will be designed for bending and then checked for deflection and shear.

Determine the Deck Span, Design Loads, and Panel Size. For the beam spacing of 5.5 ft, an initial deck thickness of 6¾ in will be used. Per the AASHTO Specifications,' the 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:

$$\text{Clear distance between beams} = 66\text{ in} - 8.5\text{ in} = 57.5\text{ in}$$

$$s = 57.5 + \frac{8.5\text{ in}}{2} = 61.75\text{ in}$$

$$\text{Clear span} + \text{deck thickness} = 57.5\text{ in} + 6.75\text{ in} = 64.25\text{ in}$$

$s = 61.75$ in will be used for design.

For HS 20-44 loading, AASHTO special provisions apply, and the deck will be designed for a 12,000-lb wheel load. Panel width for an out-to-out bridge length of 48 ft will be 11 panels 48 $^1/_8$ in wide and 1 panel 6 $^5/_8$ in wide (Fig. 10.19).

Determine Wheel Distribution Widths and Effective Deck Section Properties. The wheel load distribution width in the direction of the deck span is computed using Eq. (10.2):

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

Normal to the deck span, the wheel load distribution width is computed using Eq. (10.3):

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

Section modulus S_y and moment of inertia I_y are computed using the following equations:

$$S_y = \frac{b_d t^2}{6} = \frac{(21.75\text{ in})(6.75\text{ in})^2}{6} = 165\text{ in}^3$$

$$I_y = \frac{b_d t^3}{12} = \frac{(21.75\text{ in})(6.75\text{ in})^3}{12} = 557\text{ in}^4$$

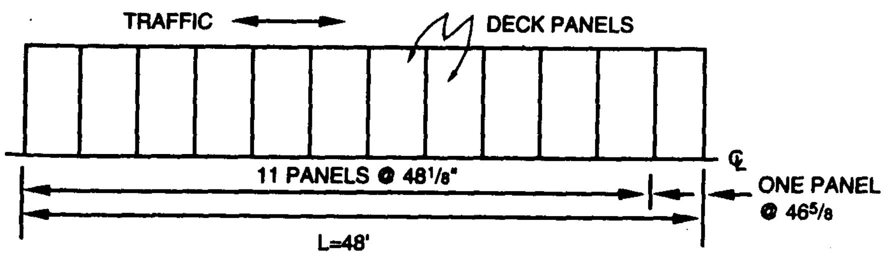


FIG. **10.19** Deck panel layout.

Determine Deck Dead Load. For a 6¾-in deck and 3-in wood plank wearing surface, dead-load unit weight w_{DL} and dead-load moment M_{DL} over the effective distribution width of 21.75 in are computed:

$$w_{DL} = 21.75 \text{ in}\left[\frac{(6.75 \text{ in} + 3 \text{ in})(50 \text{ lb/ft}^3)}{1728 \text{ in}^3/\text{ft}^3}\right] = 6.1 \text{ lb/in}$$

$$M_{DL} = \frac{w_{DL}s^2}{8} = \frac{(6.1 \text{ lb/in})(61.75 \text{ in})^2}{8} = 2907 \text{ in·lb}$$

Determine Live-Load Moment. For an effective deck span of less than 122 in, maximum live-load moment M_{LL} is computed for a 6-ft track width and 12,000-lb wheel load by the following equation (see Ritter[6]):

$$M_{LL} = 3000s - 25{,}983 = 3000(61.75 \text{ in}) - 25{,}983 = 159{,}267 \text{ in·lb}$$

Compute Bending Stress and Select a Deck Combination Symbol. The deck is not continuous over more than two spans, so bending stress f_b is based on the simple-span moment:

$$M_{TL} = M_{LL} + M_{DL} = 2907 \text{ in·lb} + 159{,}267 \text{ in·lb} = 162{,}174 \text{ in·lb}$$

$$f_b = \frac{M}{S_y} = \frac{162{,}174 \text{ in·lb}}{165 \text{ in}^3} = 983 \text{ lb/in}^2$$

From the AASHTO glulam timber design tables for axial combinations, southern pine combination No. 47 is selected with the following tabulated values and wet-service factors C_M:

$$F_{by} = 1{,}750 \text{ lb/in}^2 \qquad C_M = 0.80$$

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

$$E_y = 1{,}400{,}000 \text{ lb/in}^2 \qquad C_M = 0.833$$

The allowable bending stress F'_b is equal to the tabulated bending stress F_{by} times the wet-service factor C_M, load-duration factor C_D, and bending size factor C_F:

$$F'_b = F_{by}C_MC_DC_F$$

Per the AASHTO specifications,[1] the value of C_D is 1.15 and C_F is computed by the following equation:

$$C_F = \left(\frac{12}{t}\right)^{1/9} = \left(\frac{12}{6.75 \text{ in}}\right)^{1/9} = 1.07$$

The allowable deck bending stress is computed:

$$F'_b = F_{by}C_MC_DC_F = (1750 \text{ lb/in}^2)(0.80)(1.15)(1.07) = 1723 \text{ lb/in}^2$$

$f_b = 983$ lb/in^2 is substantially less than $F'_b = 1723$ lb/in^2, so a lower-grade glulam timber combination symbol or deck thickness may be feasible. However, no changes will be made until the live-load deflection is checked.

Check Live-Load Deflection. The allowable modulus of elasticity E' is computed by multiplying the tabulated modulus of elasticity E_y by the wet-service factor C_M:

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

From Ritter (1990)[6] the live-load deflection Δ_{LL} for a 12,000-lb wheel load and 6-ft track width is computed by the following equation:

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

$$= \frac{1.80[(138.8)(61.75\ \text{in})^3 - (20{,}780)(61.75\ \text{in}) + 90{,}000)]}{(1{,}166{,}200\ \text{lb/in}^2)(557\ \text{in}^4)} = 0.09\ \text{in}$$

The computed deflection of 0.09 in (2.29 mm) is less than the maximum allowable, so the deck thickness and combination symbol are acceptable for live-load deflection. A reduction in the glulam timber combination symbol grade or deck thickness will result in excessive deflection. Therefore, a combination symbol No. 47 will be retained.

Check Horizontal Shear. Dead-load vertical shear is computed at a distance from the support equal to the deck thickness t, and loads acting within the distance t are neglected. For w_{DL} = 6.1 lb/in (Fig. 10.20):

$$V_{DL} = w_{DL}\left(\frac{s}{2} - t\right) = 6.1\ \text{lb/in}\left(\frac{61.75\ \text{in}}{2} - 6.75\ \text{in}\right) = 147\ \text{lb}$$

Live-load vertical shear V_{LL} is computed by placing the edge of the wheel load distribution width b_t a distance t from the support (Fig. 10.21):

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

The applied stress in horizontal shear f_v is computed using AASHTO equations, assuming that the entire panel cross-sectional area A_p of the narrowest panel is effective in shear distribution:

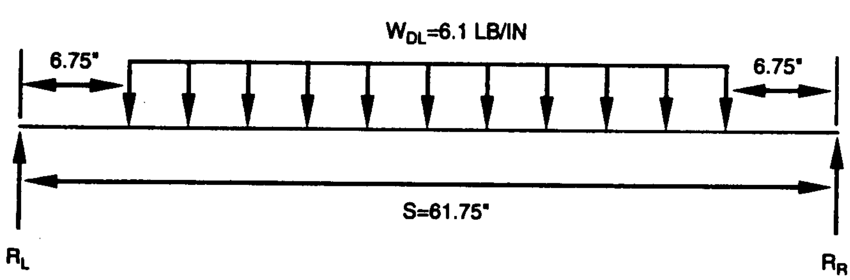


FIG. 10.20 Dead load placement for vertical shear.

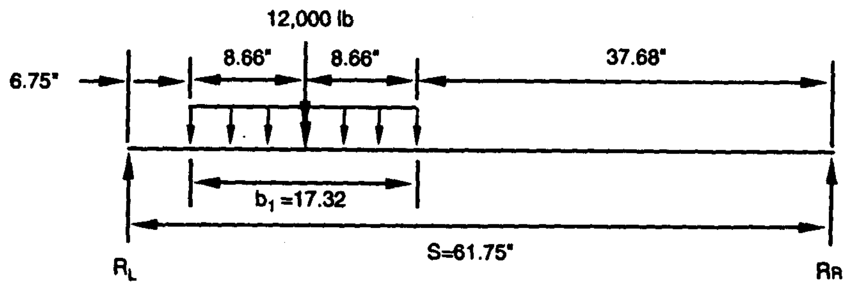


FIG. 10.21 Live load placement for vertical shear.

$$V = V_{DL} + V_{LL} = 147 \text{ lb} + 9005 \text{ lb} = 9152 \text{ lb}$$

$$A_p = (\text{panel width})(t) = (46.625 \text{ in})(6.75 \text{ in}) = 315 \text{ in}^2$$

$$f_v = \frac{1.5V}{A_p} = \frac{(1.5)(9152 \text{ lb})}{315 \text{ in}^2} = 44 \text{ lb/in}^2$$

The allowable stress in horizontal shear F'_v is equal to the tabulated stress in horizontal shear F_{vy} times the wet-service factor C_M and the load-duration factor C_D. Values of C_M and C_D are 0.875 and 1.15, respectively.

$$F'_v = F_{vy} C_M C_D = (175 \text{ lb/in}^2)(0.875)(1.15) = 176 \text{ lb/in}^2$$

$f_v = 44$ lb/in^2 < $F'_v = 176$ lb/in^2, so the panel is satisfactory in horizontal shear.

Summary. The bridge superstructure will consist of three southern pine glulam timber beams, combination symbol 24F-V3. The beams will measure 8½ in (215.9 mm) wide, 41¼ (1048 mm) deep, and 48 ft (14.63 m) long. The span center to center of bearings will be 47 ft (14.33 m). Transverse bracing will be provided for lateral support at the bearings and at the beam centerspan. The deck will consist of 12 combination No. 47 southern pine glulam timber deck panels 6¾ in (171.4 mm) thick by 16 ft (4.877 m) long. Eleven panels will be 48 $^1/_8$ in (1.23 m) wide, and one panel will be 46 $^5/_8$ in (1.184 m) wide. Stresses and live-load deflection are given in Table 10.6.

10.1.7 Design of Glulam Timber Longitudinal Deck Bridges

Deck Panel Design. Longitudinal glulam timber deck bridges consist of a series of glulam timber panels placed edge to edge across the deck width (Fig. 10.22). The lumber laminations are oriented parallel to traffic, and loads are applied parallel to the wide face of the laminations. The deck provides all structural support for the roadway, without the aid of beams or other components. However, stiffener beams are placed transverse to the panels on the deck underside to transfer loads between panels and give continuity to the system. Longitudinal glulam timber decks are practical for clear spans up to approximately 35 ft (10.67 m) and are equally adaptable to single- and multiple-lane crossings. The low deck profile makes them especially suitable for short-span applications where clearance below the structure is limited. The same basic configuration also can be used over transverse floorbeams

TABLE 10.6 Summary of Design Values for Example 10.2

Design value	Outside beams	Deck
f_b	1840 lb/in²	983 lb/in²
F'_b	1943 lb/in²	1723 lb/in²
Δ_{LL}	1.34 in. = $L/421$	0.09 in
f_v	101 lb/in²	44 lb/in²
F'_v	201 lb/in²	176 lb/in²
$f_{c\perp}$	323 lb/in²	N/A
$F_{c\perp}$	345 lb/in²	N/A
Δ_{DL}	0.56 in	N/A
Camber	2 in	N/A

Note: 1 in = 25.4 mm; 1 lb/in² = 6.895 kPa.

for the construction or rehabilitation of other superstructure types. As with glulam timber beam bridges, longitudinal glulam timber deck bridges can be prefabricated in a modular system that is pressure-treated with preservatives after all required cuts and holes are made. This improves the bridge economy and longevity and reduce field erection time.

Longitudinal glulam timber deck panels are manufactured from visually graded glulam timber axial combinations given in the AASHTO specifications.’ Combination symbols with a tabulated bending stress F_b of 1800 lb/in² (12.41 MPa) or less are most economical and are most commonly used. Panels are 42 to 54 in (1.067 to 1.372 m) wide in increments equal to the net lamination thickness of 1½ in (38.1 mm) for western species and $1\,^3/_8$ in (34.9 mm) for southern pine. Panels can be manufactured in any length subject to local pressure-treating and transportation restrictions and are available in thicknesses that correspond to the standard glulam timber beam widths given in Table 10.1. Deck thicknesses up to 10¾ in (273.0 mm) are generally manufactured from full-width laminations (Fig. 10.23). Deck thicknesses of 12¼ and 14¼ in (311.1 and 362.0 mm) are also available but typically require multiple-piece laminations, which may be bonded or unbonded depending on specific design requirements in horizontal shear. With unbonded edge joints, tabulated horizontal shear values are assumed to be 50 percent of those for comparable panels with bonded joints.

The design criteria for glulam timber longitudinal deck bridges are given in the AASHTO Specifications[1] and are based on research and development work conducted at Iowa State University.[7] The primary emphasis of the Iowa State University studies dealt with the lateral live-load distribution characteristics for deck panel design. Empirical methods for stiffener beam design also were developed based on limitations placed on design parameters within the load-distribution studies. Additional experimental data obtained by Iowa State University subsequent to development of the load-distribution criteria should eventually provide a basis for more explicit stiffener beam design criteria rather than the empirical methods currently given in the AASHTO Specifications’ and presented in this section.

Live-Load Distribution. Deck panels for longitudinal glulam timber deck bridges are designed as individual beams of rectangular cross section. Design is based on the maximum forces and deflection produced by the design vehicle, assuming that

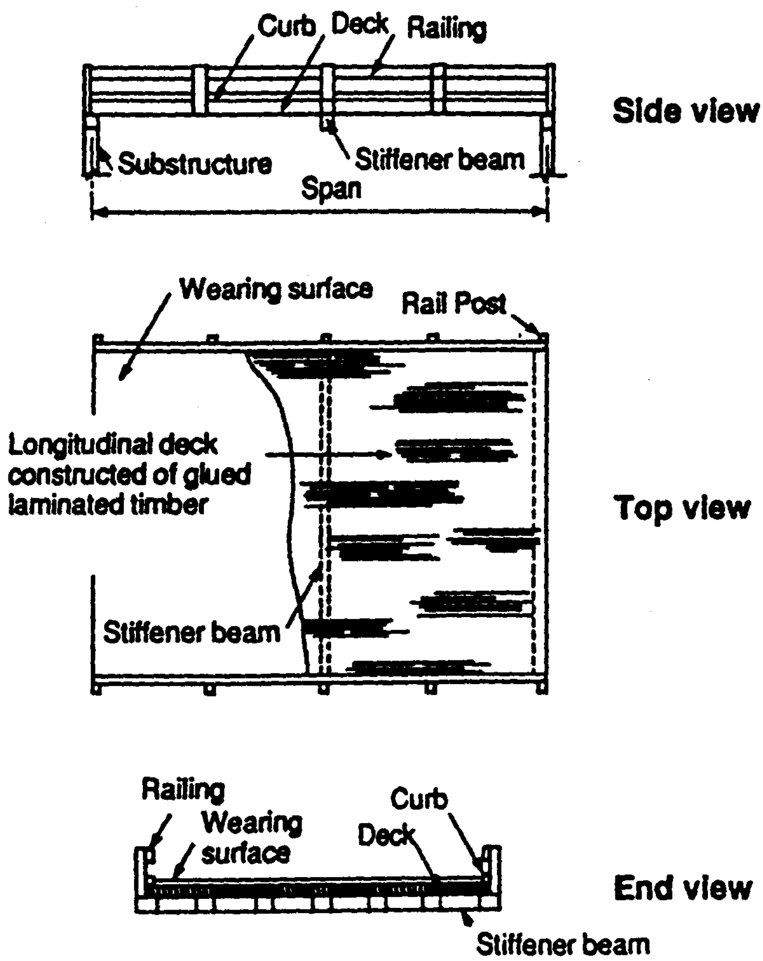


FIG. 10.22 Typical configuration for a single-lane longitudinal deck bridge.

wheel loads act as point loads in the direction of the deck span. The portion of the vehicle wheel line distributed transversely to each panel is computed as a wheel load fraction *WLF,* which is similar in application to the distribution factors used for beam design. The *WLF* represents the portion of one vehicle wheel line that is assumed to be distributed to one deck panel. The procedures for determining the *WLF* for bending, deflection, shear, and reactions follow.

1. ***Distribution for moment and deflection.*** For live-load moment and deflection, the *WLF* is based on the panel width and span in feet. The AASHTO Specifications' give *WLF* equations for bridges designed for two or more traffic lanes and for bridges designed for one traffic lane.

For bridges designed for two or more traffic lanes,

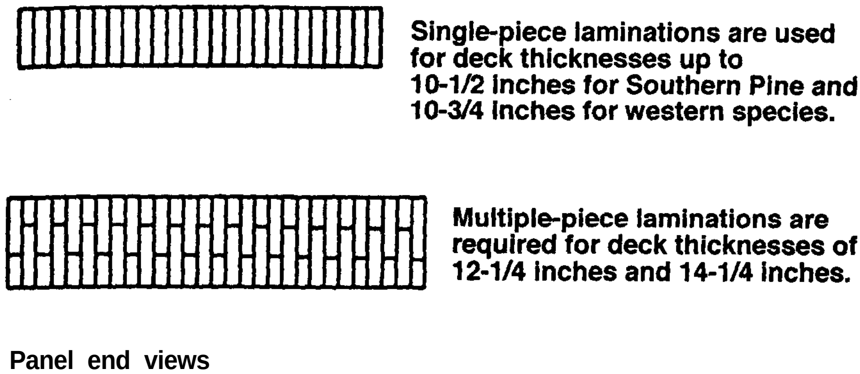


FIG. 10.23 Laminating patterns for longitudinal glulam deck panels.

$$WLF = \frac{W_p}{3.75 + (L/28)} \quad \text{or} \quad \frac{W_p}{5.00} \quad \text{whichever is greater} \tag{10.4}$$

For bridges designed for one traffic lane,

$$WLF = \frac{W_p}{4.25 + (L/28)} \quad \text{or} \quad \frac{W_p}{5.50} \quad \text{whichever is greater} \tag{10.5}$$

where WLF = the portion of the maximum wheel line moment or deflection distributed to one deck panel
W_p = panel width, ft
L = length of span for simple-span bridges or length of the shortest span for continuous-span bridges, measured center to center of bearings, ft

2. ***Distribution for shear.*** Live-load vertical shear is based on the maximum vertical shear occurring at a distance from the support equal to three times the deck thickness ($3t$) or the span quarter point ($L/4$), whichever is less. At this location, WLF for shear is the same as that specified for moment and deflection. Horizontal shear is normally not a controlling factor in longitudinal glulam timber deck design because of the relatively large panel area.

3. ***Distribution for reactions.*** The WLF for live-load reactions at the supports of multiple-lane and single-lane bridges is given by the following equation:

$$WLF = \frac{W_p}{4} \geq 1.0 \tag{10.6}$$

Stiffener Beam Design. Transverse stiffener beams are placed transversely across the underside of longitudinal glulam timber decks to distribute loads and deflections among the individual panels. Stiffener beams typically consist of horizontally laminated glulam timber beams or shallow steel shapes (Fig. 10.24). The AASHTO Specifications' require that a stiffener beam be placed at midspan for all deck spans and at intermediate spacings not to exceed 10 ft (3.048 m). A more restrictive intermediate stiffener beam spacing of 8 ft (2.438 m) is recommended by industry

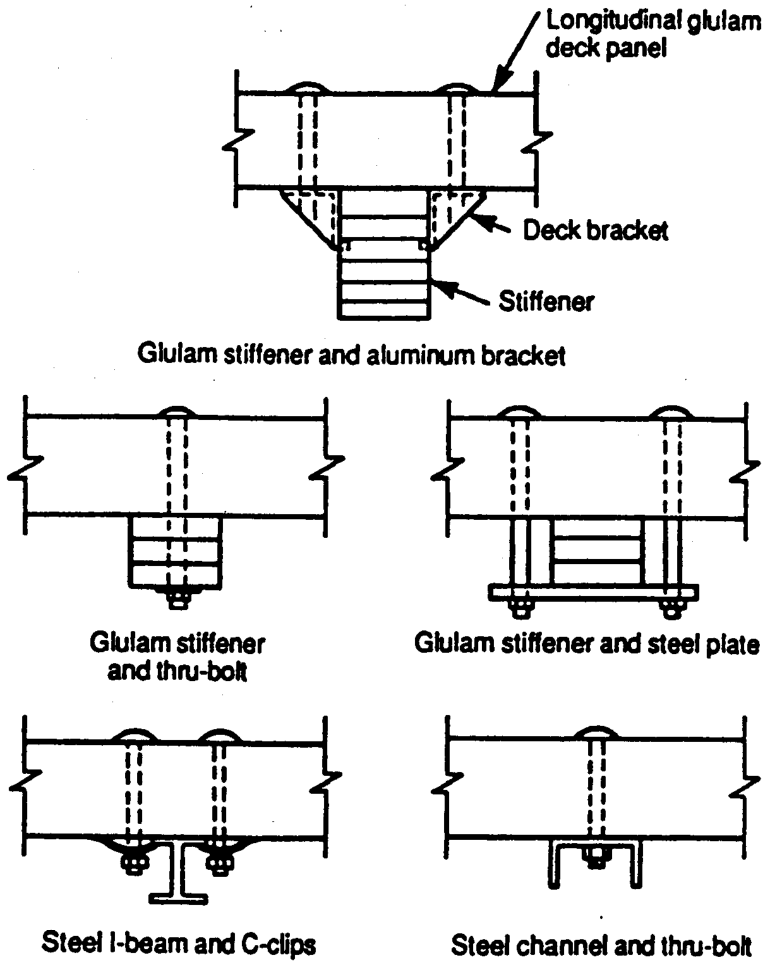


FIG. 10.24 Types of transverse stiffener-beam configurations for longitudinal glulam deck panels.

and is used in this section. Stiffener beam design consists of sizing the beam so that the stiffness factor $E'I$ of the member is not less than 80,000 k-in^2 (551.6 GN·m^2); however, this is an approximate value that should not be significantly exceeded. Experimental and analytical tests at Iowa State University have shown that the connection may be overstressed if the stiffness factor is very large, on the order of twice the minimum value. Load distribution between panels is more effectively improved by decreasing stiffener beam spacing rather than by increasing the beam size substantially above the required minimum.

Connections between the stiffener beam and the deck panels are placed approximately 6 in from each panel edge (Fig. 10.25). The type of connection depends on the stiffener-beam material and configuration. Through-bolting is most common for glulam timber beams and steel channels. Deck brackets or steel plates are also used for glulam timber beams, and C clips are used for steel I beams. A minimum bolt diameter of 3/4 in (19.05 mm) is recommended for single through-bolt connections, while a minimum 5/8-in (15.88-mm)-diameter bolt is used for bracket connections. The type of connection is left to designer judgment, since all connector types shown in Fig. 10.24 were modeled in the Iowa State University study. However, experimental results indicate that the through-bolt type of connections provide more favorable load distribution in the panels and reduce the potential for localized stress conditions in the region of the connection to the stiffener beams. They are also more effective in reducing interpanel displacements that occur between stiffener-beam locations.

10.1.8 Design Examples

Sequential design examples are included in this section to familiarize the reader with the design procedures and requirements for longitudinal glulam timber deck bridges. These examples are based on AASHTO requirements and are valid for panels that are 42 to 54 in (1.067 to 1.372 m) wide and are provided with transverse

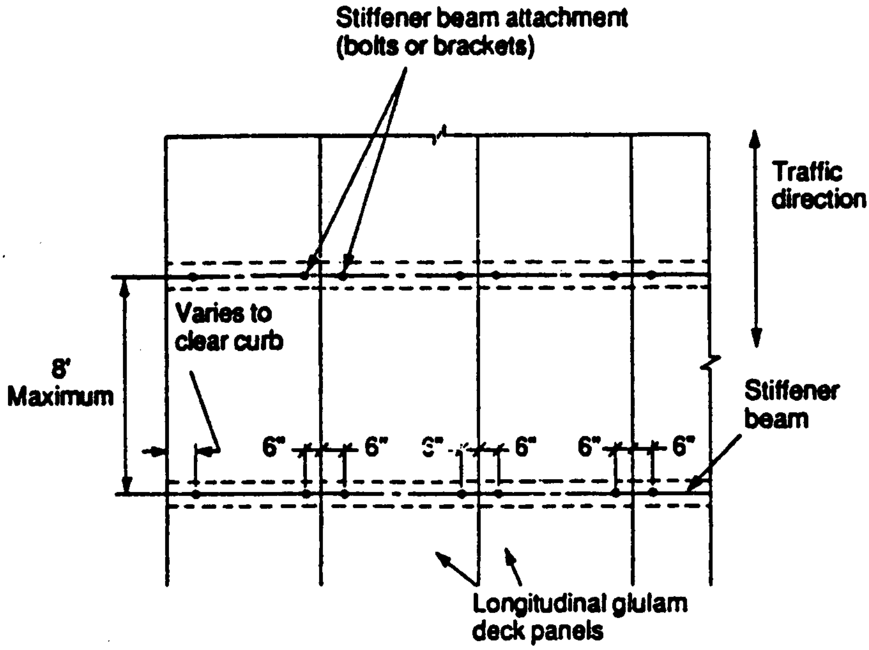


FIG. 10.25 Stiffener-beam attachment for longitudinal glulam decks.

stiffener beams. The basic sequence assumes that deck panels are initially designed for deflection and then checked for bending and shear. The process is iterative in nature if panel dimensions are changed at any point during the design process. After a suitable panel size and grade are determined, stiffener beams and bearings are designed.

Specific site requirements and criteria are noted for each example. In addition the following general criteria related to loads, materials, live-load deflection, and conditions of use are applicable.

Loads. Loads are based on the AASHTO load requirements and are illustrated for AASHTO group I, where design is routinely controlled by a combination of structure dead load and vehicle live load. Other loads and load combinations also should be checked depending on site-specific conditions. AASHTO special provisions regarding reduced wheel loads for H 20-44 and HS 20-44 trucks previously discussed for transverse glulam timber decks do not apply to longitudinal decks.

Materials. Tabulated values for glulam timber are taken from tables for axial glulam timber combinations given in the AASHTO Specifications.' *All timber components are assumed to be properly pressure-treated with an oil-type preservative after fabrication.*

Live-Load Deflection. The AASHTO Specifications' do not require a limit for deflection but recommend that the live-load deflection not exceed 1/500 of the bridge span. Although it is recommended that these deflection guidelines be followed, deflection criteria should be based on specific design circumstances and are left to designer judgment. However, it is recommended that maximum panel deflection not exceed $L/360$. Because continuity from panel to panel is provided only at stiffener-beam locations, relative panel displacements do occur at locations between these beams. At this time, there is no accurate method for predicting the interpanel displacements between stiffener beams; however, with a maximum panel live-load deflection of $L/360$, Iowa State University studies indicate that the interpanel displacement will not exceed approximately 0.10 in (2.54 mm) in most applications. The 0.10-in limit on relative panel displacement is considered the maximum allowable for acceptable asphalt wearing surface performance. A further reduction in deflection is desirable to reduce the potential for minor asphalt cracks at the panel joints or when the bridge includes a pedestrian walkway.

Conditions of Use. Tabulated values for glulam timber components must be adjusted for specific use conditions by all applicable adjustment factors given in the AASHTO Specifications.' The following criteria for adjustment factors have been used in this chapter:

1. ***Duration of load.*** Beam and deck design for combined dead load and vehicle live load are based on the 2-month load duration specified in the AASHTO Specifications.' Applicable tabulated design values are multiplied by a load-duration factor C_D of 1.15.

2. ***Moisture content.*** With the exception of transverse stiffener beams used with watertight glulam timber decks, all deck components are designed for wet-use conditions. Based on industry recommendations, stiffener beams that are treated with oil-type preservatives and are located under a watertight glulam timber deck are assumed to remain within the range of dry-use conditions.

3. ***Temperature effects and fire-retardant treatment.*** Conditions requiring adjustments for temperature or fire-retardant treatment are not applicable in the design examples.

EXAMPLE 10.3: SIMPLE-SPAN LONGITUDINAL DECK GLULAM TIMBER BRIDGE, TWO-LANE HIGHWAY LOADING Design a longitudinal deck glulam timber bridge with a length of 26 ft (7.924 m) and a center-to-center span of 25 ft (7.62 m). The bridge will carry two lanes of AASHTO HS 20-44 loading in 12-ft (3.658-m) lanes. The following provisions apply:

1. The deck wearing surface is a 3-in (76.2-mm) layer of asphalt pavement.

2. Railing dead load is 50 lb/ft (729.6 N/m) along each deck edge, and the railing extends approximately 1 ft (0.3048 m) inward from the deck edge.

3. The deck live-load deflection shall not exceed 1/500 of the deck span ($L/500$).

4. Glulam timber is visually graded southern pine.

SOLUTION: The design of longitudinal glulam timber deck panels involves an iterative process similar to that used for beam design. In this case, with a maximum live-load deflection of $L/500$, it is likely that deflection will control the design. Therefore, the deck will be designed initially based on deflection and then checked for bending and shear.

Define the Deck Configuration and Deck Panel Width. The deck span L is the distance measured center to center of the bearings. Deck width is the roadway width plus any additional width required for curb and railing systems. With two 12-ft (3.658-m)-wide traffic lanes and a railing that projects 1 ft (0.3048 m) inward from each deck edge, an out-to-out bridge width of 26 ft (7.924 m) is required.

Panel width depends on the out-to-out structure width. Panels are 42 to 54 in (1.067 to 1.372 m) wide in multiples of 1½ in (38.1 mm) for western species or $1\,^3/_8$ in (34.9 mm) for southern pine. The panels are normally designed to be of equal width, obtained by dividing the bridge width by a selected number of panels. Using southern pine glulam timber with $1\,^3/_8$-in laminations, a configuration of six panels, each 52¼ in (1.327 m) wide, is selected (Fig. 10.26).

Select a Deck Panel Combination Symbol. The southern pine glulam timber combination symbols most commonly used for longitudinal decks are No. 47 and No. 48, with tabulated modulus of elasticity E values of 1,400,000 lb/in^2 (9.652 GPa) and 1,700,000 lb/in^2 (11.72 GPa), respectively. Because of the restrictive $L/500$ live-load deflection requirement for this bridge, the No. 48 combination symbol is selected initially because of the higher E value. Tabulated values and wet-service

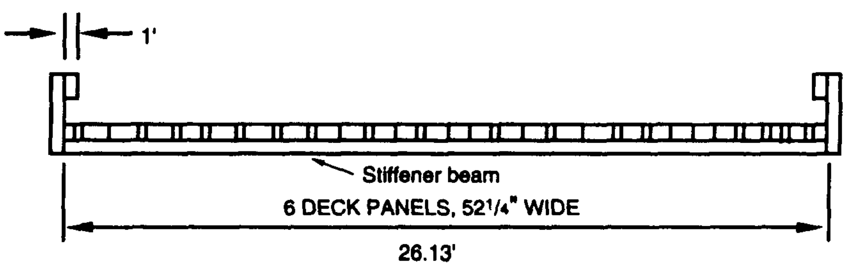


FIG. 10.26 Bridge cross section.

factors C_M for E and bending stress F_{by} are obtained from AASHTO glulam timber design tables for axial combinations:

$$F_{by} = 2000 \text{ lb/in}^2 \text{ (12.41 MPa)} \qquad C_M = 0.80$$

$$E_y = 1{,}700{,}000 \text{ lb/in}^2 \text{ (11.72 GPa)} \qquad C_M = 0.833$$

Determine the Wheel Load Fraction for Live-Load Distribution. The wheel load fraction *WLF* in wheel lines *WL* per panel is computed for a two-lane bridge using Eq. (10.4):

$$WLF = \frac{W_P}{3.75 + L/28} \quad \text{or} \quad WLF = \frac{W_P}{5} \quad \text{whichever is greater}$$

$$\frac{W_P}{3.75 + L/28} = \frac{52.25 \text{ in/12 in/ft}}{3.75 + (25 \text{ ft}/28)} = 0.94WL/\text{panel}$$

$$\frac{W_P}{5} = \frac{52.25 \text{ in/(12 in/ft)}}{5} = 0.87WL/\text{panel}$$

WLF = 0.94 *WL*/panel will be used for design.

Estimate Panel Thickness and Check Live-Load Deflection. Standard dimensions for glulam timber deck panel thickness are the same as the standard beam widths given in Table 10.1. For deflection computations, a deck panel thickness t must be estimated. An initial thickness of 12¼ in is selected, and the panel moment of inertia I_y is computed based on t and the panel width w_p in inches:

$$I_y = \frac{w_p(t)^3}{12} = \frac{52.25 \text{ in (12.25 in)}^3}{12} = 8004 \text{ in}^4$$

The allowable modulus of elasticity E' is computed per AASHTO requirements:

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

The deflection due to one wheel line Δ_{WL} of an HS 20-44 truck is computed by statics or by coefficients given in design tables. In this case, a deflection coefficient *DC* is used[6]:

$$\Delta_{WL} = \frac{DC}{E'I_y} = \frac{1.11 \times 10^{10}}{(1{,}416{,}100 \text{ lb/in}^2)(8004 \text{ in}^4)} = 0.98 \text{ in}$$

The deck live-load deflection Δ_{LL} is computed by multiplying Δ_{WL} by the *WLF*:

$$\Delta_{LL} = \Delta_{WL}(WLF) = (0.98 \text{ in})(0.94) = 0.92 \text{ in (23.4 mm)} = L/326$$

The live-load deflection of $L/326$ exceeds the allowable $L/500$, so the panel thickness must be increased. A revised panel thickness of 14¼ in is checked using the same approach:

$$I_y = \frac{w_p(t)^3}{12} = \frac{52.25 \text{ in } (14.25 \text{ in})^3}{12} = 12{,}600 \text{ in}^4$$

$$\Delta_{WL} = \frac{DC}{E'I_y} = \frac{1.11 \times 10^{10}}{(1{,}416{,}100 \text{ lb/in}^2)(12{,}600 \text{ in}^4)} = 0.62 \text{ in}$$

$$\Delta_{LL} = \Delta_{WL}(WLF) = (0.62 \text{ in})(0.94) = 0.58 \text{ in } (14.7 \text{ mm}) = L/517$$

The live-load deflection of $L/517$ is less than the allowable $L/500$, so the panel thickness is acceptable.

Compute Panel Dead Load and Dead-Load Moment. The dead load DL of the deck and asphalt wearing surface is computed in pounds per square foot based on AASHTO unit material weights of 50 lb/ft^3 for wood and 150 lb/ft^3 for asphalt pavement:

$$DL = \frac{(14.25 \text{ in})(50 \text{ lb/ft}^3)}{12 \text{ in/ft}} + \frac{(3 \text{ in})(150 \text{ lb/ft}^3)}{12 \text{ in/ft}} = 97 \text{ lb/ft}^2$$

Railing dead load of 100 lb/ft is distributed equally over the entire deck width. An additional dead load of 10 lb/ft is added for the distributor beams and miscellaneous hardware for a total load of 110 lb/ft. The dead load per panel w_{DL} is computed for a span of 25 ft:

$$w_{DL} = \frac{(97 \text{ lb/ft}^2)(52.25 \text{ in})}{12 \text{ in/ft}} + \frac{110 \text{ lb/ft}}{6 \text{ panels}} = 441 \text{ lb/ft}$$

Panel deal-load moment M_{DL} is computed for the uniformly distributed load:

$$M_{DL} = \frac{w_{DL}L^2}{8} = \frac{(441 \text{ lb/ft})(25 \text{ ft})^2}{8} = 34{,}453 \text{ ft·lb}$$

Compute Live-Load Moment. Live-load moment M_{LL} is the product of the WLF and the moment produced by one wheel line of the design vehicle M_{WL}. From AASHTO Specification tables, the maximum moment from one wheel line of an HS 20-44 truck on a 25-ft span is 103,680 ft·lb:

$$M_{LL} = M_{WL}(WLF) = 103{,}680 \text{ ft·lb}(0.94) = 97{,}459 \text{ ft·lb}$$

Check Bending Stress. The total moment M is the sum of the dead-load and live-load moments:

$$M_{TL} = M_{DL} + M_{LL} = 34{,}450 \text{ ft·lb} + 97{,}459 \text{ ft·lb} = 131{,}909 \text{ ft·lb}$$

The deck panel section modulus S_y and the applied bending stress f_b are computed:

$$S_y = \frac{w_p(t)^2}{6} = \frac{(52{,}25 \text{ in})(14.25 \text{ in})^2}{6} = 1768 \text{ in}^3$$

$$f_b = \frac{M}{S_y} = \frac{(131{,}909 \text{ ft·lb})(12 \text{ in/ft})}{1768 \text{ in}^3} = 895 \text{ lb/in}^2$$

The allowable bending stress F'_b is computed per AASHTO requirements by multiplying the tabulated bending stress F_{by} by the wet-service factor C_M, load-duration factor C_D, and the size factor C_F. Assuming wet-service conditions and a controlling load combination of dead load and vehicle live load, the values for C_M and C_D are 0.80 and 1.15, respectively. The values of C_F and F'_b are computed:

$$C_F = \left(\frac{12}{t}\right)^{1/9} = \left(\frac{12}{14.25 \text{ in}}\right)^{1/9} = 0.98$$

$$F'_b = F_{by}C_MC_DC_F = (1800 \text{ lb/in}^2)(0.80)(1.15)(0.98) = 1803 \text{ lb/in}^2$$

$f_b = 895$ lb/in^2 $< F'_b = 1803$ lb/in^2, so the deck thickness and combination symbol are satisfactory for bending.

Check Horizontal Shear. Dead-load vertical shear V_{DL} is computed at a distance t from the support, and loads acting within a distance t of the supports are neglected (Fig. 10.27):

$$V_{DL} = w_{DL}\left(\frac{L}{2} - t\right) = 441 \text{ lb/ft}\left(\frac{25 \text{ ft}}{2} - \frac{14.25 \text{ in}}{12 \text{ in/ft}}\right) = 4989 \text{ lb}$$

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

$$3t = \frac{3(14.25 \text{ in})}{12 \text{ in/ft}} = 3.56 \text{ ft} \qquad \frac{L}{4} = \frac{25 \text{ ft}}{4} = 6.25 \text{ ft}$$

$3t = 3.56$ ft controls, and maximum live-load vertical shear V_{LL} at that location is computed by multiplying the shear due to one wheel line of an HS 20-44 truck V_{WL} by the WLF (Fig. 10.28):

$$V_{WL} = R_L = \frac{(16{,}000 \text{ lb})(7.44 \text{ ft} + 21.44 \text{ ft})}{25 \text{ ft}} = 18{,}483 \text{ lb}$$

$$V_{LL} = V_{WL}(WLF) = 18{,}483 \text{ lb}(0.94) = 17{,}374 \text{ lb}$$

The stress in horizontal shear f_v is computed per AASHTO requirements:

$$V = V_{DL} + V_{LL} = 4989 \text{ lb} + 17{,}374 \text{ lb} = 22{,}363 \text{ lb}$$

$$f_v = \frac{3V}{2w_pt} = \frac{(3)(22{,}363 \text{ lb})}{(2)(52.25 \text{ in})(14.25 \text{ in})} = 45 \text{ lb/in}^2$$

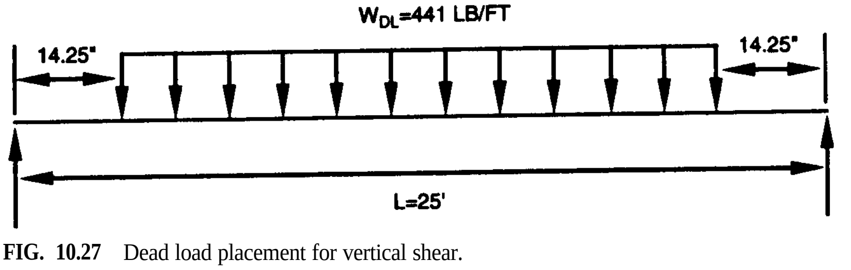


FIG. 10.27 Dead load placement for vertical shear.

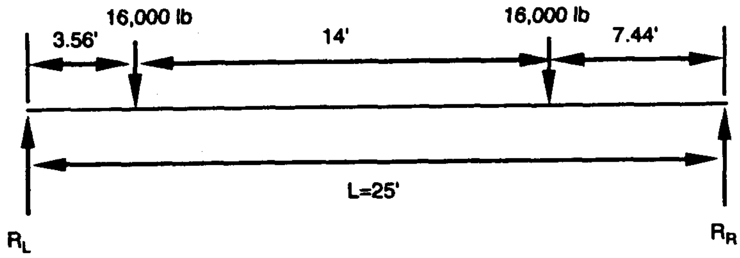


FIG. 10.28 Live load placement for vertical shear.

For a 14¼-in deck thickness, an edge layup will be used for the glulam timber deck panels. From the AASHTO tables for southern pine glulam timber axial combinations, the tabulated horizontal shear stress F_{vy} for combination No. 48 is 90 lb/in^2 if edge joints are not bonded and 175 lb/in^2 if edge joints are bonded. For this case, f_v is low, and the unbonded value will be used.

The allowable stress in horizontal shear F'_v is computed by multiplying F_{vy} by $C_M = 0.875$ and $C_D = 1.15$:

$$F'_v = F_{vy}C_MC_D = (90 \text{ lb/in}^2)(0.875)(1.15) = 91 \text{ lb/in}^2$$

$F'_v = 91 \text{ lb/in}^2 > f_v = 45 \text{ lb/in}^2$, so the deck panel is satisfactory in horizontal shear.

Determine Stiffener Beam Spacing and Configuration. The maximum recommended spacing for stiffener beams is 8 ft. For this bridge, glulam timber stiffener beams through-bolted to the deck panels (see Fig. 10.24) will be used at the span quarter points for a spacing of 6.25 ft. The size and stiffness of the stiffener beam must be sufficient to provide a minimum *EI* value of 80,000 k-in^2. Select a southern pine combination symbol No. 48 glulam timber stiffener, 5 in wide and 5½ in deep (dry-use conditions may be used for glulam timber stiffener beams if they are protected by a watertight deck):

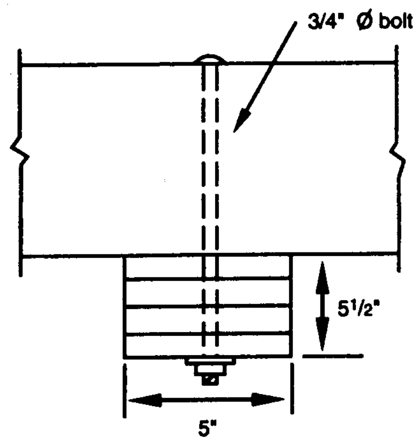
$$E' = E_yC_M = 1{,}700{,}000 \text{ lb/in}^2 \,(1.0) = 1{,}700{,}000 \text{ lb/in}^2$$

$$I = \frac{bd^3}{12} = \frac{(5 \text{ in})(5.5 \text{ in})^3}{12} = 69 \text{ in}^4$$

$$E'I = \frac{1{,}700{,}000 \text{ lb/in}^2}{1{,}000 \text{ lb/k}}(69 \text{ in}^4) = 117{,}300 \text{ k-in}^2$$

117,300 k-in^2 > 80,000 k-in^2, so 5- by 5½-in stiffener beams are satisfactory. The beams will be attached to the deck with 3/4-in-diameter bolts located 6 in from the panel edge (Fig. 10.29).

Determine Bearing Configuration and Check Bearing Stress. Bearings for longitudinal glulam timber decks are designed to resist the vertical and lateral forces in the same manner previously discussed for glulam timber beams. However, for longitudinal deck bridges, the required bearing length is normally controlled by considerations for bearing configuration rather than stress in compression perpendicular to gram. From a practical standpoint, a bearing length of 10 to 12 in is



recommended for stability and deck attachment. In this case, a bearing length of 12 in is used.

The dead-load reaction R_{DL} is determined by assuming the panel acts as a simple beam between supports. For an out-out panel length of 26 ft,

$$R_{DL} = \frac{w_{DL}L}{2} = \frac{(441 \text{ lb/ft})(26 \text{ ft})}{2} = 5733 \text{ lb}$$

The live-load reaction R_{LL} is computed by multiplying the maximum reaction for one wheel line R_{WL} by the reaction WLF. From the AASHTO Specifications.[1] the maximum reaction for one wheel line of an HS 20-44 truck is 23,040 lb.

$$WLF = \frac{W_p}{4} = \frac{(52.25 \text{ in})(12 \text{ in/ft})}{4} = 1.09WL/\text{panel}$$

$$R_{LL} = WLF(R_{WL}) = 1.09(23{,}040 \text{ lb}) = 25{,}114 \text{ lb}$$

The applied stress in compression perpendiculat to grain $f_{c\perp}$ is computed for a length of bearing ℓ_b of 12 in:

$$f_{c\perp} = \frac{R_{DL} + R_{LL}}{w_p(\ell_b)} = \frac{5733 \text{ lb} + 25{,}114 \text{ lb}}{52.25 \text{ in } (12 \text{ in})} = 49 \text{ lb/in}^2$$

The allowable stress in compression perpendicular to grain $F'_{c\perp}$ is computed in accordance with AASHTO requirements:

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

$f_{c\perp} = 49$ lb/in^2 < $F'_{c\perp} = 345$ lb/in^2, so a bearing length of 12 in is satisfactory. The bearing configuration shown in Fig. 10.30 will be used.

Summary. The bridge will consist of six combination No. 48 southern pine glulam timber deck panels. Each panel will measure 14¼ in thick, 52¼ in wide, and 26 ft long. Stiffener beams are combination No. 48 southern pine glulam timber,

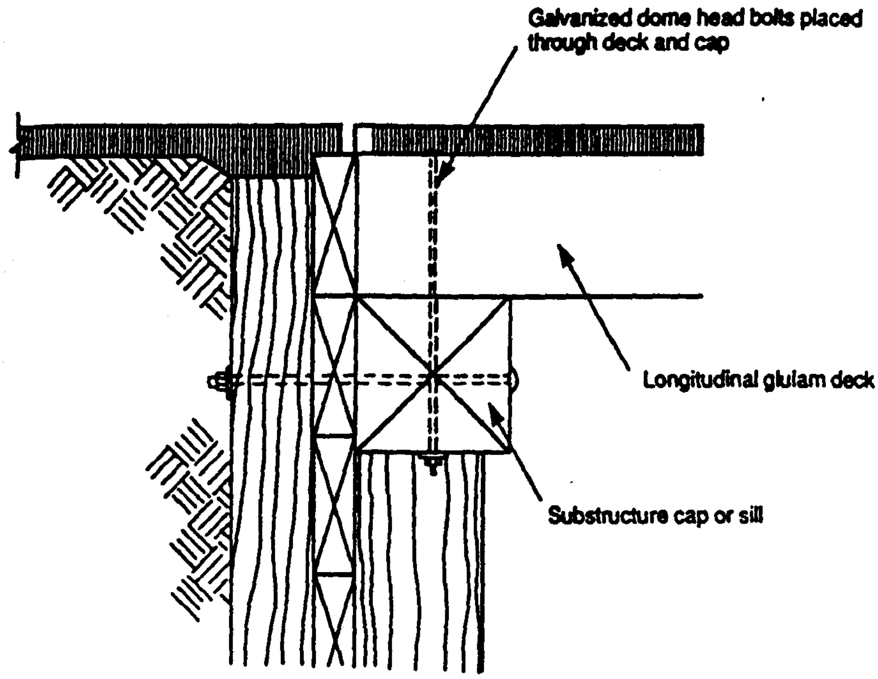


FIG. 10.30 Typical bearing configuration for longitudinal glulam timber decks.

5 in wide by 5½ in deep, placed at span quarter points. Stresses and deflection are as follows:

$$f_b = 895 \text{ lb/in}^2$$

$$F'_b = 1803 \text{ lb/in}^2$$

$$\Delta_{LL} = 0.58 \text{ in} = L/517$$

$$f_v = 45 \text{ lb/in}^2$$

$$F'_v = 91 \text{ lb/in}^2$$

$$f_{c\perp} = 49 \text{ lb/in}^2$$

$$F'_{c\perp} = 345 \text{ lb/in}^2$$

Note: 1 in = 25.4 mm; 1 lb/in^2 = 6.895 kPa.

EXAMPLE 10.4: SIMPLE-SPAN LONGITUDINAL DECK GLULAM TIMBER BRIDGE, SINGLE-LANE HIGHWAY LOADING Design a single-lane longitudinal deck glulam timber bridge with a span of 21 ft (6.401 m) center to center of bearings. The bridge is located on a low-volume road and will carry one lane of AASHTO HS 20-44 loading in a 14-ft (4.267-m) lane. The following provisions apply:

1. The deck wearing surface is a 3-in (76.2-mm) layer of wood plank.
2. A wood curb on scupper blocks 12 in (0.3048 m) wide is provided along each deck edge with a dead load of 30 lb/ft (437.8 N/m).
3. The deck live-load deflection shall not exceed 1/360 of the deck span ($L/360$).
4. Glulam timber is visually graded western species.

SOLUTION: The design of this bridge will follow the same basic sequence described in the preceding example. The deck panels will be initially designed for live-load deflection and then checked for bending, shear, and compression perpendicular to grain.

Define the Deck Configuration and Deck Panel Width. With a 14-ft (4.267-m)-wide lane and curbs that extend 1 ft (0.3048 m) inward from the deck edges, an out-to-out bridge width of 16 ft (4.877 m) is required. Using western species glulam timber with 1½-in (38.1.mm) laminations, a configuration of four panels, each 48 in (1.219 m) wide, is selected (Fig. 10.31).

Select* a *Deck Panel Combination Symbol. The western species glulam timber combination symbols used most commonly for longitudinal decks are No. 2 and No. 3, with E values of 1,700,000 lb/in^2 (11.72 GPa) and 1,800,000 lb/in^2 (12.41 GPa), respectively. In this case, combination symbol No. 2 is selected initially, and tabulated-values and wet service factors are obtained from AASHTO glulam timber design tables for axial combinations:

$$F_{by} = 1{,}800 \text{ lb/in}^2 \text{ (12.41 MPa)} \qquad C_M = 0.80$$

$$E_y = 1{,}700{,}000 \text{ lb/in}^2 \text{ (11.72 GPa)} \qquad C_M = 0.833$$

$$F_{c\perp} = 560 \text{ lb/in}^2 \text{ (3.861 MPa)} \qquad C_M = 0.53$$

Determine the Wheel Load Fraction for Live-Load Distribution. The *WLF* in wheel lines, *WL*, per panel is computed for a one-lane bridge using Eq. (10.5):

$$WLF = \frac{W_p}{4.25 + L/28} \qquad \text{or} \qquad WLF = \frac{W_p}{5.5} \qquad \text{whichever is greater}$$

$$\frac{W_p}{4.25 + L/28} = \frac{4 \text{ ft}}{4.25 + (21 \text{ ft}/28)} = 0.80 WL/\text{panel}$$

$$\frac{W_p}{5.5} = \frac{4 \text{ ft}}{5.5} = 0.73 WL/\text{panel}$$

$WLF = 0.80$ *WL*/panel will be used for design.

Estimate Panel Thickness and Check Live-Load Defection. An initial thickness of 12¼ in is estimated, and the panel moment of inertia I_y is computed based on t and the panel width w_p in inches:

$$I_y = \frac{w_p(t)^3}{12} = \frac{48 \text{ in } (12.25 \text{ in})^3}{12} = 7353 \text{ in}^4$$

The allowable modulus of elasticity E' is computed per AASHTO requirements:

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

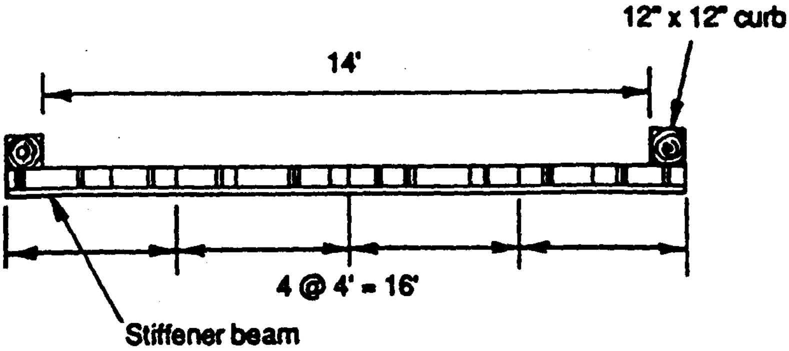


FIG. 10.31 Bridge cross section.

The deflection due to one wheel line Δ_{WL} of an HS 20-44 truck is computed by a deflection coefficient, *DC*, given in design tables[6]:

$$\Delta_{WL} = \frac{DC}{E'I_y} = \frac{5.33 \times 10^9}{(1{,}416{,}100 \text{ lb/in}^2)(7353 \text{ in}^4)} = 0.51 \text{ in}$$

The deck live-load deflection Δ_{LL} is computed by multiplying A, by the *WLF*:

$$\Delta_{LL} = \Delta_{WL}(WLF) = (0.51 \text{ in})(0.80) = 0.41 \text{ in } (10.4 \text{ mm}) = L/615$$

The live-load deflection of *L*/615 is considerably less than the allowable *L*/360. The deck thickness will be reduced to 10¾ in, and the live-load deflection checked:

$$I_y = \frac{w_p(t)^3}{12} = \frac{48 \text{ in } (10.75 \text{ in})^3}{12} = 4969 \text{ in}^4$$

$$\Delta_{WL} = \frac{DC}{E'I_y} = \frac{5.33 \times 10^9}{(1{,}416{,}100 \text{ lb/in}^2)(4969 \text{ in}^4)} = 0.76 \text{ in}$$

$$\Delta_{LL} = \Delta_{WL}(WLF) = (0.76 \text{ in})(0.80) = 0.61 \text{ in } (15.5 \text{ mm}) = L/413$$

The live-load deflection of *L*/413 is less than the allowable *L*/360, so the panel thickness is acceptable.

Compute Panel Dead Load and Dead-Load Moment. The dead load *DL* of the deck and plank wearing surface is computed in pounds per square foot based on the AASHTO unit material weights of 50 lb/ft^3 for wood:

$$DL = \frac{(10.75 \text{ in} + 3 \text{ in})(50 \text{ lb/ft}^3)}{12 \text{ in/ft}} = 57.3 \text{ lb/ft}^2$$

Curb dead load of 60 lb/ft is distributed equally over the entire deck width. An additional dead load of 10 lb/ft is added for the distributor beams and miscellaneous hardware, for a total load of 70 lb/ft. The dead load per panel w_{DL} is computed for a span of 21 ft:

$$w_{DL} = \frac{(57.3\ \text{lb/ft}^2)(48\ \text{in})}{12\ \text{in/ft}} + \frac{70\ \text{lb/ft}}{4\ \text{panels}} = 247\ \text{lb/ft}$$

M_{DL} is computed for the uniformly distributed load:

$$M_{DL} = \frac{w_{DL}L^2}{8} = \frac{(247\ \text{lb/ft})(21\ \text{ft})^2}{8} = 13{,}616\ \text{ft·lb}$$

Compute Live-Load Moment. From AASHTO Specification tables, the maximum moment from one wheel line of an HS 20-44 truck on a 21-ft span is 84,000 ft·lb. Live-load moment M_{LL}. is the product of the *WLF* and the moment produced by one wheel line of the design vehicle M_{WL}:

$$M_{LL} = M_{WL}(WLF) = 84{,}000\ \text{ft·lb}(0.80) = 67{,}200\ \text{ft·lb}$$

Check Bending Stress. The total moment M is the sum of the dead-load and live-load moments:

$$M = M_{DL} + M_{LL} = 13{,}616\ \text{ft·lb} + 67{,}000\ \text{ft·lb} = 80{,}816\ \text{ft·lb}$$

S_y and the applied bending stress f_b are computed:

$$S_y = \frac{w_p(t)^2}{6} = \frac{(48\ \text{in})(10.75\ \text{in})^2}{6} = 925\ \text{in}^3$$

$$f_b = \frac{M}{S_y} = \frac{(88{,}816\ \text{ft·lb})(12\ \text{in/ft})}{925\ \text{in}^3} = 1078\ \text{lb/in}^2$$

The allowable bending stress F'_b is computed per AASHTO requirements by multiplying the tabulated bending stress F_{by} by the wet-service factor C_M, load-duration factor C_D, and the size factor C_F. Assuming wet-service conditions and a controlling load combination of dead load and vehicle live load, the values for C_M and C_D are 0.80 and 1.15, respectively. The values of C_F and F'_b are computed:

$$C_F = \left(\frac{12}{t}\right)^{1/9} = \left(\frac{12}{10.75\ \text{in}}\right)^{1/9} = 1.01$$

$$F'_b = F_{by}C_MC_DC_F = (1800\ \text{lb/in}^2)(0.80)(1.15)(1.01) = 1673\ \text{lb/in}^2$$

$f_b = 1048\ \text{lb/in}^2 < F'_b = 1673\ \text{lb/in}^2$, so the deck thickness and combination symbol are satisfactory for bending.

Check Horizontal Shear. Dead-load vertical shear V_{DL} is computed at a distance t from the support, and loads acting within a distance t of the supports are neglected.

$$V_{DL} = w_{DL}\left(\frac{L}{2} - t\right) = 247\ \text{lb/ft}\left(\frac{21\ \text{ft}}{2} - \frac{10.75\ \text{in}}{12\ \text{in/ft}}\right) = 2372\ \text{lb}$$

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

$$3t = \frac{3(10.75\ \text{in})}{12\ \text{in/ft}} = 2.69\ \text{ft} \qquad \frac{L}{4} = \frac{21\ \text{ft}}{4} = 5.25\ \text{ft}$$

$3t = 2.69$ ft controls, and maximum live-load vertical shear V_{LL} at that location is computed by multiplying the shear due to one wheel line V_{WL} of an HS 20-44 truck by the *WLF* (Fig. 10.32):

$$V_{WL} = R_L = \frac{(16{,}000\ \text{lb})(4.31\ \text{ft} + 18.31\ \text{ft})}{21\ \text{ft}} = 17{,}234\ \text{lb}$$

$$V_{LL} = V_{WL}(WLF) = 17{,}234\ \text{lb}(0.80) = 13{,}787\ \text{lb}$$

The stress in horizontal shear f_v is computed per AASHTO requirements:

$$V_{TL} = V_{DL} + V_{LL} = 2372\ \text{lb} + 13{,}787\ \text{lb} = 16{,}159\ \text{lb}$$

$$f_v = \frac{3V}{2w_p t} = \frac{(3)(16{,}159\ \text{lb})}{(2)(48\ \text{in})(10.75\ \text{in})} = 47\ \text{lb/in}^2$$

For a 10¾-in deck thickness, single-piece laminations will be used for the glulam timber deck panels. From the AASHTO tables for western species glulam timber axial combinations, the tabulated horizontal shear stress F_{vy} for combination No. 2 is 145 lb/in^2. The allowable stress in horizontal shear F'_v is computed by multiplying F_{vy} by $C_M = 0.875$ and $C_D = 1.15$:

$$F'_v = F_{vy}C_MC_D = (145\ \text{lb/in}^2)(0.875)(1.15) = 146\ \text{lb/in}^2$$

$F'_v = 146$ lb/in^2 $> f_v = 47$ lb/in^2, so the deck panel is satisfactory in horizontal shear.

Determine Stiffener Beam Spacing and Configuration. Glulam timber stiffener beams through-bolted to the deck panels will be used at the span third points for a spacing of 7 ft. Select a western species combination symbol No. 2 glulam timber stiffener, 5 $^1/_8$ in wide and 6 in deep, and apply $C_M = 0.833$;

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

$$I = \frac{bd^3}{12} = \frac{(5.125\ \text{in})(6\ \text{in})^3}{12} = 92\ \text{in}^4$$

$$E'I = \frac{1{,}416{,}100\ \text{lb/in}^2}{1000\ \text{lb/k}}(92\ \text{in}^4) = 130{,}281\ \text{k-in}^2$$

130,281 k-in^2 > 80,000 k-in^2, so 5 $^1/_8$ by 6-in stiffener beams are satisfactory. The

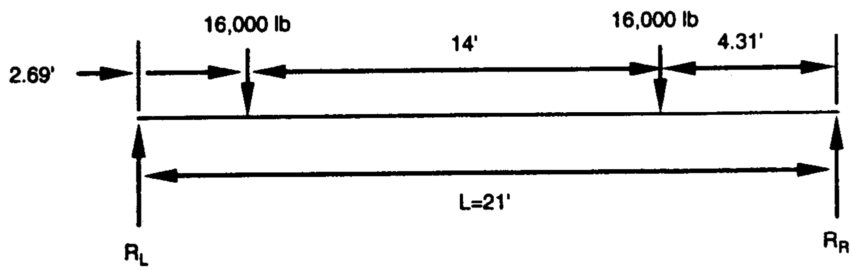


FIG. 10.32 Live load placement for vertical shear.

beams will be attached to the deck with 3/4-in-diameter bolts located 6 in from the panel edge.

Determine Bearing Configuration and Check Bearing Stress. For the given span of 21 ft, a bearing length of 12 in is selected. The dead load reaction R_{DL} is determined for a panel length of 23 ft by assuming that the panel acts as a simple beam between supports:

$$R_{DL} = \frac{w_{DL}L}{2} = \frac{(247 \text{ lb/ft})(23 \text{ ft})}{2} = 2841 \text{ lb}$$

From the AASHTO Specifications,' the maximum reaction for one wheel line R_{WL} of an HS 20-44 truck is 21,330 lb. The live-load reaction R_{LL} is the product of R_{WL} and the *WLF*:

$$WLF = \frac{W_p}{4} = \frac{(48 \text{ in})/(12 \text{ in/ft})}{4} = 1.00WL/\text{panel}$$

$$R_{LL} = WLF(R_{WL}) = 1.00(21{,}330 \text{ lb}) = 21{,}330 \text{ lb}$$

The applied stress in compression perpendicular $f_{c\perp}$ to gram is computed for a length of bearing ℓ_b of 12 in:

$$f_{c\perp} = \frac{R_{DL} + R_{LL}}{w_p(\ell_b)} = \frac{2841 \text{ lb} + 21{,}330 \text{ lb}}{48 \text{ in (12 in)}} = 42 \text{ lb/in}^2$$

$F'_{c\perp}$ is computed in accordance with AASHTO requirements:

$$F'_{c\perp} = F'_{c\perp}(C_M) = 560 \text{ lb/in}^2(0.53) = 297 \text{ lb/in}^2$$

$f_{c\perp} = 42 \text{ lb/in}^2 < F'_{c\perp} = 297 \text{ lb/in}^2$, so a bearing length of 12 in is satisfactory. The bearing configuration shown in Fig. 10.33 will be used.

Summary. The bridge will consist of four combination No. 2 western species glulam timber deck panels. Each panel will measure 10¾ in thick, 48 in wide, and 23 ft long. Stiffener beams are combination No. 2 western species glulam timber, 5 $^1/_8$ in wide by 6 in deep, placed at span third points. Stresses and deflection are as follows:

$$f_b = 1048 \text{ lb/in}^2$$

$$F'_b = 1673 \text{ lb/in}^2$$

$$\Delta_{LL} = 0.61 \text{ in} = L/413$$

$$f_v = 47 \text{ lb/in}^2$$

$$F'_v = 146 \text{ lb/in}^2$$

$$f_{c\perp} = 42 \text{ lb/in}^2$$

$$F'_{c\perp} = 297 \text{ lb/in}^2$$

Note: 1 in = 25.4 mm; 1 lb/in^2 = 6.895 kPa.

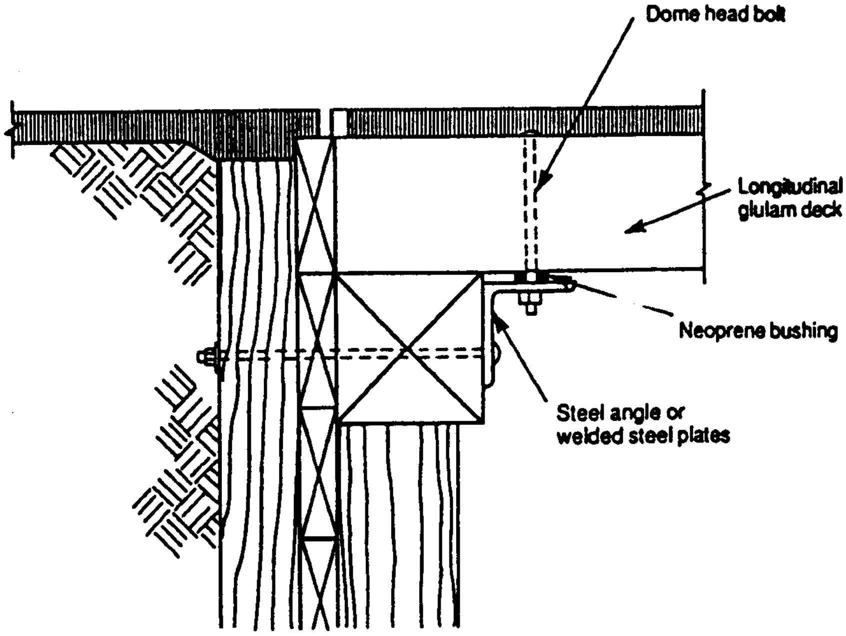


FIG. 10.33 Typical bearing configurations for longitudinal glulam decks.

10.2 CONCRETE FORMWORK

Although for multiple reuse applications wood formwork has been replaced by steel formwork to some degree and a large share of scaffolding supports are now tubular steel frames, wood is still used for a large part of the concrete forming in the United States because of its availability and ease of fabrication. As with steel, wood forms can be reused if dismantled carefully, and the material can be salvaged for future use. All formwork must be designed to support both the weight or pressure of wet concrete and all temporary construction loads, including lateral loads, until the concrete reaches a strength level at which it becomes self-supporting. The forms also must be watertight.

Formwork, particularly when used for floors and roofs, should be designed so that portions of the formwork such as beam sides or some slab soffits can be removed early, leaving the beam soffit forms with their shores or slab shores to support the main floor loads until the concrete gains greater strength. Flat slab floor forms must be removable in a sequence such that reshores can be properly placed to support the slab, control early deflections of the slab, and support additional levels of forming.

Deflections may control the spacing of the plywood supports and ties as well as the sizes of walers and joists. The deflections of concrete flatwork must be considered in determining both the camber to be built into the forms and the jacking clearance needed to allow the formwork to be decentered for removal without damaging the concrete. The following example illustrates a problem which occurred when proper camber was not used. A lightweight concrete fill was to be placed on a floor composed of plywood sheathing over wood/plywood I-beam joists supported by glulam beams. No camber was specified or furnished for the glulam beams. Normal tolerance on camber for the span involved is ± ¼ in (6 mm). Wood/plywood I beams are not typically cambered in manufacturing. The screeds for the specified 1½-in (38-mm)-thick concrete slabs were set at the column lines for a level floor. Due to the lack of camber of the glulam beam system and dead-load deflections, the resulting concrete slab was over 3 in (76 mm) thick at some midspans as a result of dead-load deflections. This was a considerable overrun in material quantities, and a further problem resulted because the actual dead load considerably exceeded the design dead load.

One of the most common causes of formwork failure is the lack of adequate lateral bracing. This can result in either local instability of an individual shore or instability of the entire formwork system. A line of bracing is not effective unless it is anchored to a solid support. For the stability of the system as a whole, the method used to transport the concrete is very important. The use of power buggies requires that consideration be given to the forces which result from the simultaneous action of the rapid braking of the buggy and the dumping of the concrete. This may produce large horizontal and vertical dynamic forces, which must be resisted.

The type and amount of vibration used for compacting the concrete can change the effective fluid depth on the forms. Therefore, it is recommended that the contractor plan a program for vibrating so that its effect can be included in the design loads for the forms.

Another problem often found in formwork is that shoring is not adequately supported either horizontally or vertically at its base. Adequate bearing under the shores must be provided, and the possibility of wet ground conditions at the base of the shore which can reduce the bearing capacity of the supporting material must not be neglected. Wet conditions could be due to the weather, water from form or truck washing, leakage from the forms, and so on.

Formwork must be as watertight as possible to prevent surface marring of the concrete and to prevent possible reduction in strength of the concrete due to mortar loss. Special surfaces may be required to obtain a desired architectural finish.

Once the loads are determined, the design of the formwork normally follows accepted engineering practice. Recommended design stresses for wood members in wall formwork can reasonably be increased by 25 percent, since the duration of the full concrete pressure will not exceed 7 days. For slab formwork, a 15 percent increase in allowable stresses is appropriate if the wood is to be used only once, since it would be supporting the concrete, depending on the weather and the stripping schedule, for a maximum of 14 to 28 days. For repetitive use, cumulative loading effects will eliminate any increase in allowable design stresses which would otherwise apply for a shorter single-use duration of load. If the formwork is to be exposed to moisture for any extended period of time, it is strongly recommended that the allowable design stresses be reduced to account for the wet condition of use. This is especially true for bearing stresses perpendicular to the grain.

Shoring systems will range from posts at close spacing to elaborate truss and tower systems for structures such as concrete domes, arches, and bridges. Many of the large concrete arches, hypars, and similar items built in the early 1950s used a system of wood trusses and towers for formwork support. Some of these were reused several times on the same project, and others were reused on duplicate projects.

In addition, all formwork designs must include the loads that result from construction equipment and construction workers placing and finishing the concrete. This will vary but should be at least 50 lb/ft^2 (245 kg/m^2). The formwork also may have to support other material that is to be attached to the concrete, such as stone facing or other aesthetic finishes.

Tables 10.7 through 10.10 are taken from the American Concrete Institute publication SP4.[8] Tables 10.7 and 10.8 give the maximum internal lateral pressures to be expected for column and wall forms, respectively, and are based on test results and practical experience. Tables 10.9 and 10.10 give the minimum lateral forces for wall and slab form bracing, respectively.

TABLE 10.7 Maximum Lateral Pressure for Design of Column Forms*

Rate of place-ment R, ft/h	Maximum lateral pressure p,†‡ lb/ft^2					
	90°F	80°F	70°F	60°F	50°F	40°F
1						
2		600 psf minimum governs				
3					690	825
4			664	750	870	1050
5	650	742	793	900	1050	1275
6	750	825	921	1050	1230	1500
7	850	938	1050	1200	1410	1725
8	950	1050	1178	1350	1590	1950
9	1050	1163	1307	1500	1770	2175
10	1150	1275	1436	1650	1950	2400
11	1250	1388	1564	1800	2130	2625
12	1350	1500	1693	1950	2310	2850
13	1450	1613	1822	2100	2490	3000
14	1550	1725	1950	2250	2670	
16	1750	1950	2207	2550	3000	
18	1950	2175	2464	2850		
20	2150	2400	2721	3000		
22	2350	2625	2979			
24	2550	2850	3000			
26	2750	3000				
28	2950					
30	3000					

Note: 1 ft = 0.305 m, 1 lb/ft^2 = 47.88 Pa, C° = (F° - 32)5/9.

*For concrete with type I cement having a unit weight of 150 lb/ft^3 and slump ≦ 4 in.

†Do not use design pressure in excess of 3000 lb/ft^2 (144 kPa) or 150 lb/ft^3 (23.6 kN/m^3) × height of fresh concrete in forms in ft (m), whichever is less.

‡3000 lb/ft^2 maximum governs.

Source: From SP4, *Formwork for Concrete*, 6th ed., American Concrete Institute, 1995,[8] by permission of the publisher. Based on ACI Committee 347 concrete pressure formula.

TABLE 10.8 Maximum Lateral Pressure for Design of Wall Forms*

Rate of placement R, ft/h	Maximum lateral pressure p,† lb/ft^2					
	90°F	80°F	70°F	60°F	50°F	40°F
1	600 psf minimum governs					
2						
3					690	825
4			664	750	870	1050
5	650	712	793	900	1050	1275
6	750	825	921	1050	1230	1500
7	850	938	1050	1200	1410	1725
8	881	973	1090	1246	1466	1795
9	912	1008	1130	1293	1522	1865
10	943	1043	1170	1340	1578	1935

Note: 1 ft = 0.305 m, 1 lb/ft^2 = 47.88 Pa, C° = (F° - 32)5/9.

*For concrete with type I cement having a unit weight of 150 lb/ft^3. no admixtures or pozzolans, vibration depth limit of 4 ft or less and slump ≦ 4 in.

†Do not use design pressure in excess of 2000 lb/ft^2 (96 kPa) or 150 lb/ft^3 (23.6 kN/m^3) × height of fresh concrete in forms in ft (m), whichever is less.

Source: From SP4, *Formwork for Concrete,* 6th ed., American Concrete Institute 1995,[8] by permission of the publisher. Based on ACI Committee 347 concrete pressure formula.

EXAMPLE 10.5 Design the wall forms for an 8-in (203-mm)-thick by 10-ft (3.05-m)-high concrete pour. The forms are to be filled at the rate of 5 ft (1.5 m) per hour when the air temperature is 70°F (21°C). A vibrator is to be used, but it will not be extended down into the fresh concrete more than 4 ft (1.22 m). Assume that ¾-in (19-mm) Plyform is placed on studs which are 16 in (406 mm) on center. A single plate will be used at the bottom of the form and anchored to previously poured concrete. Ties rated by the manufacturer at 2000 lb (8.9 kN) each will be used (Fig. 10.34*a*).

SOLUTION: ***Tie Design.*** Try ties at 24 in on center horizontally and 16 in on center vertically. The distribution of pressure on the form is obtained by using Table 10.8. From this table the tabular lateral pressure is 793 lb/ft^2. However, according to the footnote of the table, the maximum pressure used need not exceed 150 × 5 =750 lb/ft^2, which is shown in Fig. 10.34*b*.

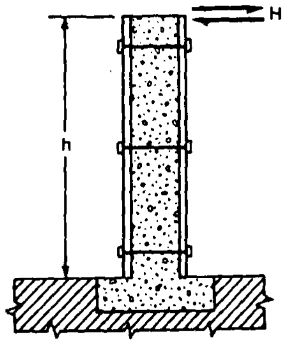
Check Maximum Tie Force. The force per tie is

$$750 \times 1.33 \times 2.0 = 1995 \text{ lb} < 2000 \text{ lb} \qquad \text{OK}$$

Waler Design. Assume walers are to be placed at 16-in centers vertically with studs on 16-in centers horizontally, with ties spaced at 24-in centers horizontally (Fig. 10.34*c*). The Load on the waler from the stud is 750 × 1.33 × 1.33 = 1327 lb.

TABLE 10.9 Minimum Lateral Force for Design of Wall Forms

Minimum lateral force, lb/ft, applied at top of wall form in either direction.



Wall height (above grade) *h*, ft	Force *H*, lb/ft: Committee 347 minimum: 100 lb/ft or 15-lb/ft^2 wind	Wind force prescribed by local code,* lb/ft^2: 10	20	25	30
(Above grade)					
4 or less	30	20	40	50	60
6	45	30	60	75	90
8	100	100	100	100	120
10	100	100	100	125	150
12	100	100	120	150	180
14	105	100	140	175	210
16	120	100	160	200	240
18	135	100	180	225	270
20	150	100	200	250	300
22 or more	7.5*h*	5.0*h*	10.0*h*	12.5*h*	15.0*h*
Walls below grade					
8 or less	Brace to maintain alignment				
More than 8	100 psf minimum, or brace for any lateral forces which are greater.				

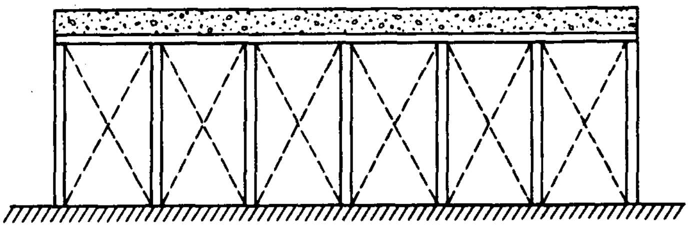
Note: 1 ft = 0.305 m, 1 lb/ft^2 = 47.88 N/m^2, 1 lb/ft = 14.6 N/m.

*Wind force prescribed by local code shall be used whenever it would require a lateral force for design greater then the minimum shown.

Source: From Sp4, *Formwork for Concrete*, 6th ed., American Concrete Institute, 1995,[8] by permission of the publisher.

TABLE 10.10 Minimum Lateral Force for Design of Slab Form Bracing

Minimum lateral force, lb/ft, applied along edge of slab in either direction.



Solid slab thickness,* in	Dead load, lb/ft^2	Load H, † lb/ft — Width of slab in direction of force, ft: 20	40	60	80	100
4	65	100	100	100	100	130
6	90	100	100	108	144	180
8	115	100	100	138	184	230
10	140	100	112	168	224	280
12	165	100	132	198	264	330
14	190	100	152	228	304	380
16	215	100	172	258	344	430
20	265	106	212	318	424	530

Note: 1 ft = 0.305 m, 1 lb/ft = 14.6 N/m, 1 lb/ft^2 (load) = 4.89 kg/m^2.

*Slab thickness given for concrete weighing 150 lb/ft^3 (2400 kg/m^3); allow 15 lb/ft^2 (73 kg/m^2) for weight of forms. For concrete of different weight or for joist slabs or beam and slab combinations, estimate dead load per ft^2 (m^2) and work from dead-load column, interpolating as needed on straight-line basis. *Note:* Do not interpolate in ranges that begin with 100 lb minimum load.

†Special conditions may require heavier bracing H.

Source: From SP4, *Formwork for Concrere,* 6th ed., American Concrete Institute. 1995,[8] by permission of the publisher.

Figure 10.34*d* and *e* shows two possible load conditions for transferring the loads on the studs to the ties, depending on the relative lateral positioning of the studs and ties.

Assume that the alternative waler loading is selected, since all tie forces for this case are less than the 2000 lb allowable, and that all spans are assumed to be simple (on the conservative side, except in spans where waler joints are located and joint locations are unknown). The bending moments in the spans are

$$M_{AB} = M_{CD} = 1327 \times 4 = 5308 \text{ in·lb}$$

$$M_{BC} = 1327 \times 24/4 = 7962 \text{ in·lb}$$

$$M_{DE} = 663 \times 12 = 7956 \text{ in·lb}$$

Check Bending. Assuming a No. 1 grade of Douglas fir-larch having an allowable F_b of 1000 lb/in^2 is used [see Appendix to this book or supplement to the 1997 edition of *National Design Specifications for Wood Construction* (NDS)[2]], the required section modulus for the waler is

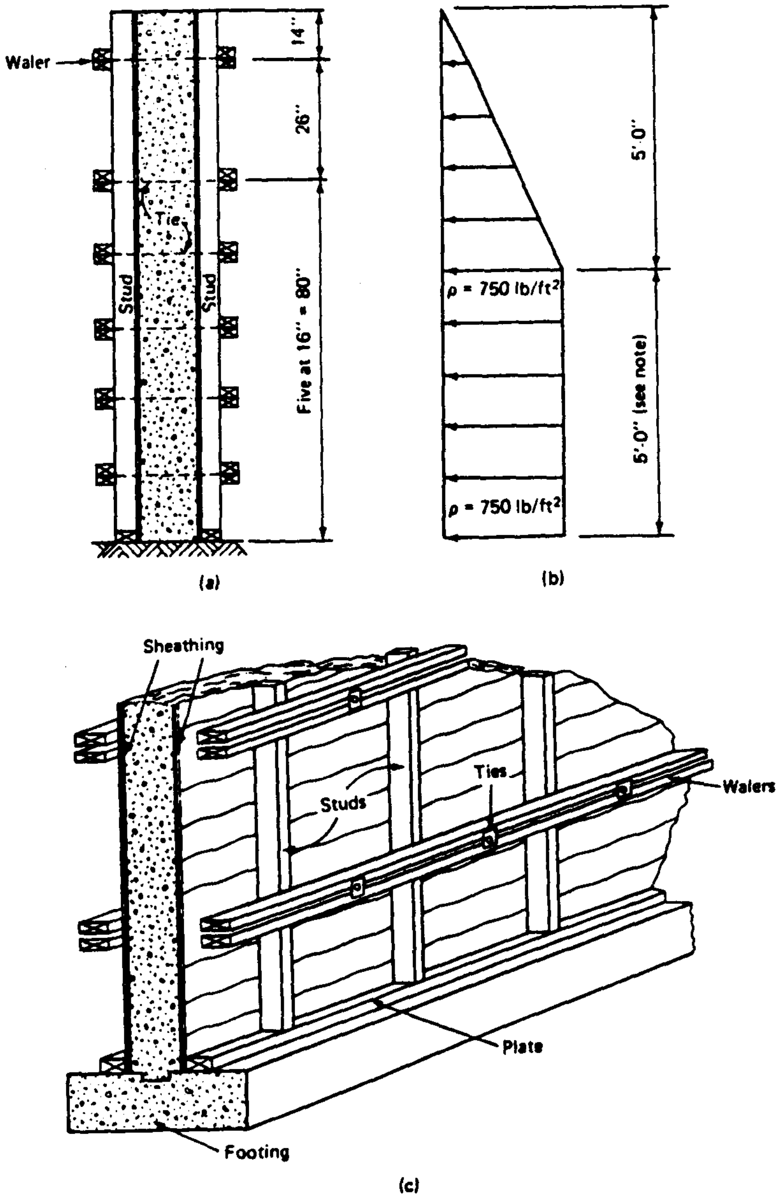
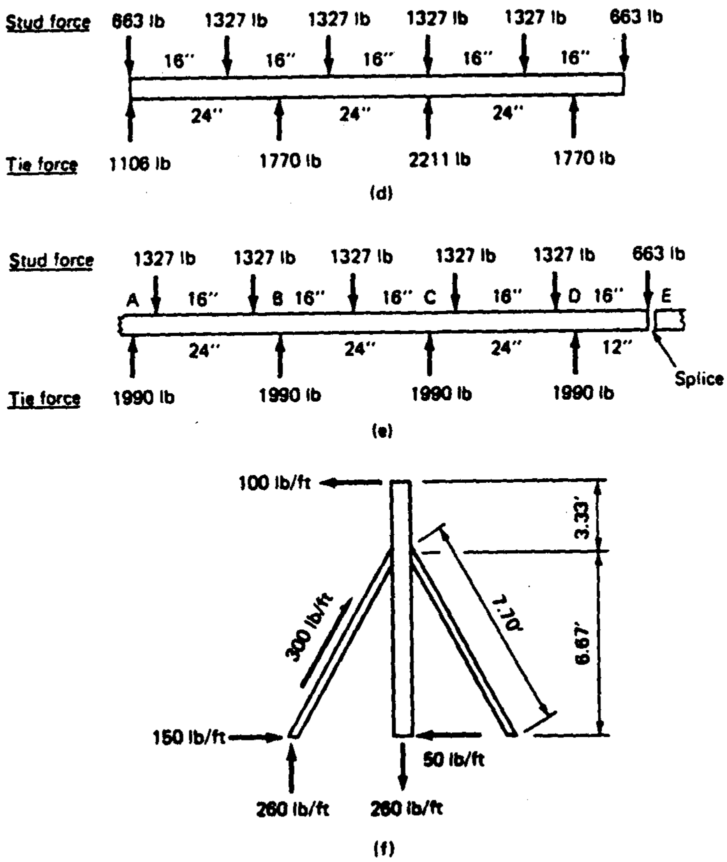


FIG. 10.34 Formwork design. (*a*) Wall form cross section; (*b*) distribution of concrete pressure on wall form; (*c*) wall forms; (*d*) possible waler load pattern; (*e*) alternative waler load pattern; (*f*) forces per foot of wall length. *Note:* Partially consolidated fluid pressure for concrete; see first footnote in Table 10.8.



FIG, 10.34 ***(Continued)***

$$S = \frac{M}{F'_b} = \frac{7962}{1000 \times 1.25^* \times 1.15^\dagger \times 1.5^\ddagger} = 3.69 \text{ in}^3$$

Note: If forms are used under wet conditions, F_b also should be multiplied by 0.85. S = 3.06 in^3 for a single 2×4. Try two 2×4s at 16 in on center:

$$S = 3.06 \times 2 > 3.82 \text{ in}^3 \qquad \text{OK}$$

*A 7-day load-duration factor was assumed.

†A 15 percent increase in allowable stress was assumed due to the load sharing provided when three or more parallel members are placed 24 in or less on center.

‡Size-factor increase for a 4 in wide member.

Check Shear

$$V(\max) = 1327 \text{ lb}$$

$$f_v = \frac{1.5V}{2A} = \frac{1.5 \times 1327}{2 \times 1.5 \times 3.5} = 190 \text{ lb/in}^2$$

$$F_V = 95 \times 1.25 = 119 \text{ lb/in}^2 < 190 \text{ lb/in}^2 \qquad \text{NG}$$

Thus try two 2 × 6s:

$$f_v = \frac{1.5 \times 1327}{2 \times 1.5 \times 5.5} = 121 \text{ lb/in}^2 = 121 \approx 119 \text{ lb/in}^2 \qquad \text{OK}$$

Check Bearing at Tie Washers (Wet Use)

$$f_{c\perp} = \frac{1990}{1.5 \times 3.0} = 442 \text{ lb/in}^2$$

$$F'_{c\perp} = F_{c\perp} C_M C_b = 625 \times 0.67 \times 1.13 = 473 \text{ lb/in}^2 > 442 \text{ lb/in}^2 \qquad \text{OK}$$

C_b is the bearing-area factor which applies when the bearing length is less than 6 in. C_M is the wet-service factor.

Stud Design. Assume that studs act as simple span beams between walers (conservative assumption) with a uniformly distributed load. The required size for a Douglas fir-larch construction grade stud having an allowable bending stress of 1000 lb/in^2 is

$$M = \frac{w\ell^2}{8} = \frac{750 \times 1.33 \times 16^2}{12 \times 8} = 2660 \text{ in·lb}$$

$$S = \frac{M}{F'_b} = \frac{2660}{1000 \times 1.25 \times 1.15 \times 1.5} = 1.23 \text{ in}^3$$

Try one 2 × 4 at 16 in on center:

$$S = 3.05 \text{ in}^3 > 1.23 \text{ in}^3 \qquad \text{OK}$$

Check Shear, Simple Span

$$w = 1000 \text{ lb/ft or } 83.3 \text{ lb/in}$$

$$R = 1000\left(\frac{16}{12}\right)\left(\frac{1}{2}\right) = 667 \text{ lb}$$

$$V = 667 - (3.5 \times 83.3) = 376 \text{ lb}$$

$$f_v = \frac{3V}{2A} = \frac{3 \times 376}{2 \times 1.5 \times 3.5} = 107 \text{ lb/in}^2 < 95 \times 1.25 = 119 \text{ lb/in}^2 \qquad \text{OK}$$

This is the required spacing for the bottom 5 ft of the form. The spacing for the top 5 ft can be increased as long as the load on the waler from the stud remains at 1327 lb or less.

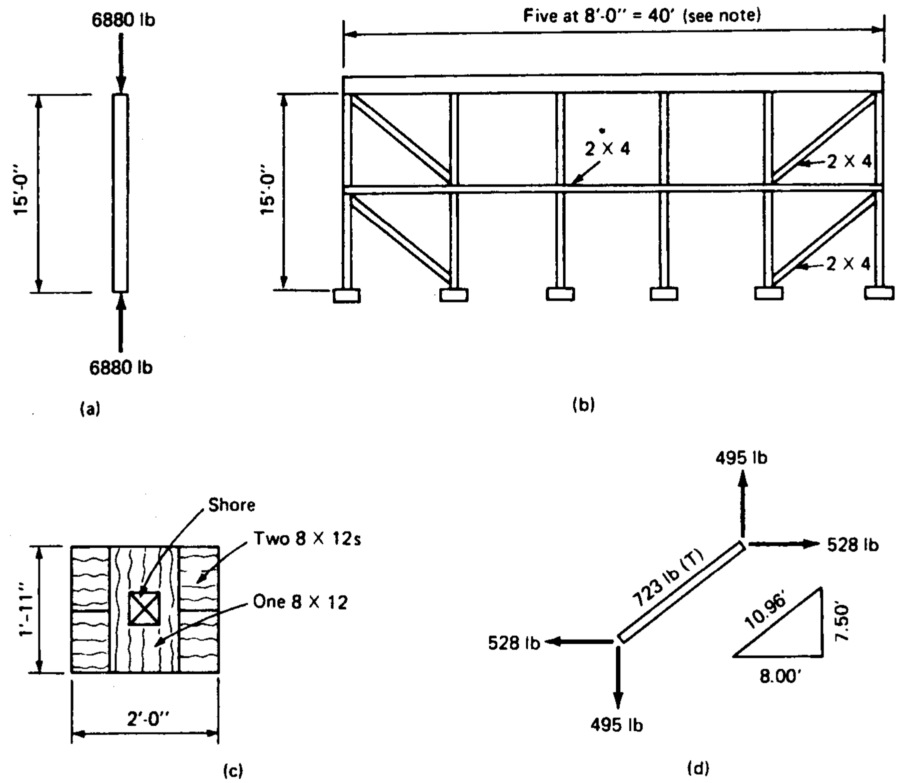


FIG. 10.35 Shore design details. (*a*) Typical shore; (*b*) elevation. (*Note:* Braced bents at 4'-0" on center.) (*c*) Base for shore; (*d*) diagonal brace-forces and dimensions.

Bracing. The form must be braced to carry the minimum 100-lb/ft lateral load at the top as determined from Table 10.9, assuming a wind force of 20 lb/ft^2 or less. Place braces at 4-ft centers, using a double 2 × 4 (38.1 × 88.9 mm*) nailed together with 16d nails at 12-in (305-mm) centers braced to deadmen. Braces will be placed on both sides so that they only need to act in compression. See Fig. 10.34*f* for forces:

$$f_c = \frac{300 \times 4}{2 \times 1.5 \times 3.5} = 114 \text{ lb/in}^2 \qquad \text{OK BY INSPECTION}$$

EXAMPLE 10.6 Design the shores and bracing for the forms shown in Fig. 10.35. The shores are 15 ft (4.57 m) long and are spaced at 4-ft (1.2-m) and 8-ft (2.44 m) centers under a 40- by 40-ft (12.2- by 12.2-m) slab which is 12 in (305 mm) thick. The concrete is to be pumped. Therefore no lateral loads will result from concrete-placing equipment.

*All SI units shown for sawn lumber are for the actual net dimensions corresponding to the nominal dimensions indicated.

SOLUTION: Assume a live load of 50 lb/ft^2 due to worker and equipment and a dead load of 150 lb/ft^2 for the weight of concrete plus 15 lb/ft^2 for the weight of forms. Therefore, the total load per shore is

Shore Design. Try a 4 × 4 No. 2 Douglas fir (dry-use conditions) and assume the shore has no rotational restraint at its ends (pinned ends) and that it is laterally supported at its midlength. Assume E = 1,600,000 lb/in^2 (11.72 GPa) and F_c = 1350 lb/in^2 (10.0 MPa).

$$\frac{\ell_e}{d} = \frac{7.5 \times 12}{3.5} = 25.7 < 50 \qquad \text{OK}$$

$$F_{cE} = \frac{K_{cE}E'}{(\ell_e/d)^2} = \frac{0.3 \times 1{,}600{,}000}{(25.7)^2} = 727 \text{ lb/in}^2$$

$$F_c^* = F_c C_D C_F = 1350 \times 1.25 \times 1.15 = 1941 \text{ lb/in}^2$$

$$C_P = \frac{1 + (F_{cE}/F_c^*)}{2c} - \sqrt{\left[\frac{1 + (F_{cE}/F_c^*)}{2c}\right]^2 - \frac{(F_{cE}/F_c^*)}{c}}$$

$$= \frac{1 + (727/1941)}{2 \times 0.8} - \sqrt{\left[\frac{1 + (727/1941)}{2 \times 0.8}\right]^2 - \frac{(727/1941)}{0.8}}$$

$$= 0.339$$

$$F_c' = F_c^* C_p = 2084 \times 0.339 = 658 \text{ lb/in}^2$$

$$P_{\text{allowable}} = F_c'A = 658 \times 3.5 \times 3.5$$

$$= 8059 \text{ lb} > 6880 \text{ lb} \qquad \text{OK}$$

Assuming an allowable soil bearing pressure of 2000 lb/ft^2, the required base area per shore is

$$A \text{ (required)} = \frac{6880}{2000} = 3.44 \text{ ft}^2$$

Use a 2-ft 0-in by 1-ft 11-in base, as shown in Fig. 10.2*c*.

$$A = 3.83 \text{ ft}^2 \qquad \text{OK}$$

Bracing. It is generally recommended that slab forms be braced for a minimum lateral load of 100 lb/ft of slab edge or for 2 percent of total dead load on the form (distributed as uniform load per lineal foot of slab edge), whichever is greater. Only the area of slab placed in a single pour needs to be considered,

$$F \text{ (2 percent of dead load)} = 0.02 \times 165 \times 40 = 132 \text{ lb/ft}$$

Table 10.10 is based on 2 percent dead load, with a minimum of 100 lb/ft, and will give identical results. Based on a value of 132 lb/ft, the force in a brace for the 8-ft-wide shore spacing is (Fig. 10.35*d*)

$$F = \frac{132 \times 4 \times 10.96}{8.0} = 723 \text{ lb}$$

If it is assumed that the bracing acts in tension only and that a No. 3 Douglas fir 2 × 4 will be used with F_t = 325 lb/in^2 (dry condition of use), then the allowable force for the 2 × 4 is

$$P = F_t C_D C_F A = 325 \times 1.25 \times 1.5 \times 5.25 = 3200 \text{ lb} > 723 \text{ lb} \qquad \text{OK}$$

Each end of the brace must be capable of transferring a force of 723 lb. (For the design of joints using fasteners and connectors, see Chap. 5). Bracing using 2 × 4s adequately fastened to the shores would be satisfactory and must be provided as shown in Fig. 10.35*b*. It is also necessary to provide bracing in the 4-ft shore spacing direction.

10.3 WOOD-METAL COMPOSITES

Wood-metal structures are typically of two types. The first is basically the combination of two beams of different materials which share in carrying the load, while the second uses the metal as a tensile reinforcement for the wood. The simplest of the first type is illustrated by a header composed of two wood 3 × 12s (63 × 286 mm) with a $^3/_8$- by 11-in (9.5- by 279-mm) steel plate between them (Fig. 10.36). All pieces are bolted together at close intervals, and all members bear on the supports at their ends. These are typically used in situations where a deeper wood beam would not leave adequate headroom or in rehabilitation work where it is necessary to reinforce existing wood members (Fig. 10.37). In this latter case the steel plates can be positioned on the outside of the existing wood member.

When sheet steel plates are placed between lumber laminations, as shown in Fig. 10.37, and used as flexural members, they are known as *flitch beams.* They have become more economically feasible as a result of the availability of helically threaded and hardened framing nails. These nails can be hammer- and machine-driven effectively through the wood and the sheet steel plates. Other forms of wood such as glulam and laminated veneer lumber, however, have largely replaced the use of flitch beams. It is important in composite construction that the members be adequately fastened together so that both the wood and the steel deflect equally and each component material carries its proportionate share of the design load. Therefore, the fasteners or connectors must be designed to transfer the shear forces

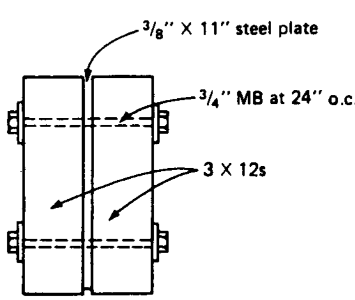


FIG. 10.36 Composite beam.

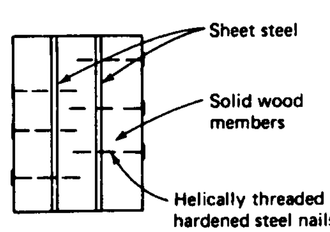


FIG. 10.37 Flitch beam.

between members unless adequate bearing is provided so that loads are applied directly to both wood and steel.

EXAMPLE 10.7: COMPOSITE BEAM ANALYSIS Determine the maximum design dead load plus snow load for the composite wood-steel beam shown in Fig. 10.38*a* and *b*. Assume that the wood members are No. 1 & Btr Douglas fir and the plate is A36 steel. Also assume that the beam will remain dry in service.

SOLUTION: Allowable stresses and moduli of elasticity are as follows:

Steel	Wood
$E_s = 29 \times 10^6$ lb/in²	$E_w = 1.8 \times 10^6$ lb/in²
$F_{bs} = 0.66f_y = 23{,}800$ lb/in²	$F_{bw} = 1200$ lb/in²
$F_{vs} = 0.4\ F_y = 14{,}400$ lb/in²	$F_{vw} = 95$ lb/in²

The load-duration factor for the combination of dead and snow loads is 1.15. Using the concept of transformed section and converting all steel to an equivalent wood area, the modified section is

$$n = \frac{E_s}{E_w} = \frac{29 \times 10^6}{1.8 \times 10^6} = 16.1$$

The width of the transformed steel (Fig. 10.38*c*) is

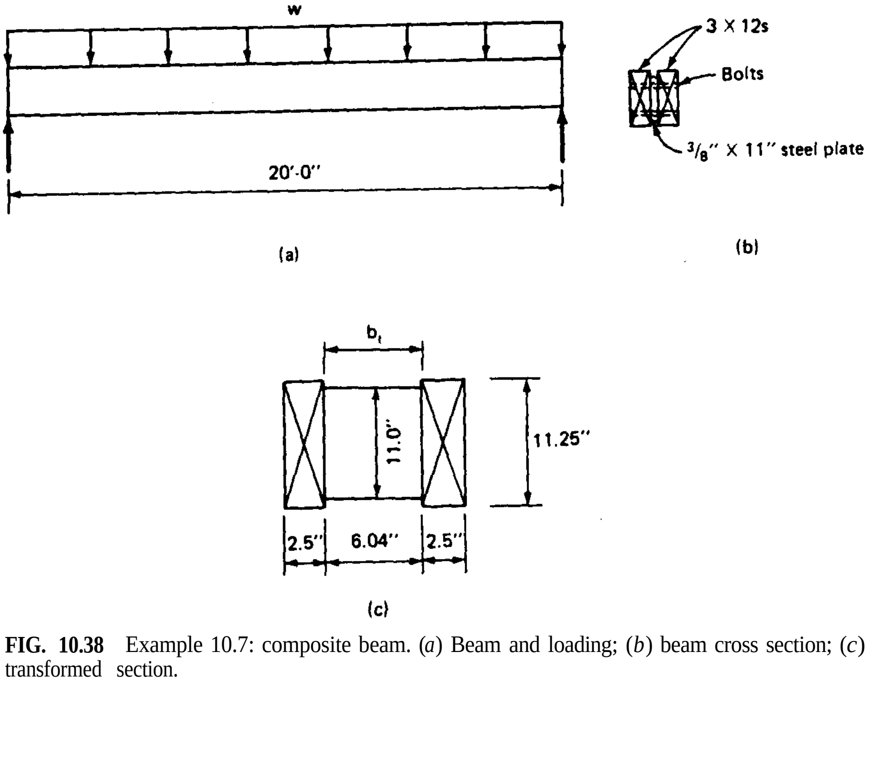


FIG. 10.38 Example 10.7: composite beam. (*a*) Beam and loading; (*b*) beam cross section; (*c*) transformed section.

$$b_t = 3/8 \times 16.1 = 6.04 \text{ in}$$

The transformed section modulus is

$$I_t = 2 \times 296.6 + \frac{6.04 \times 11^3}{12} = 1263 \text{ in}^4$$

Assuming that the wood controls the design based on its flexural capacity, the allowable uniformly distributed load is

$$F'_b = \frac{Mc}{I_t}$$

$$M = \frac{F'_b I_t}{c} = \frac{1200 \times 1.15 \times 1.0 \times 1263}{11.25/2} = 309{,}900 \text{ lb·in}$$

$$M = \frac{w\ell^2}{8}$$

$$w = \frac{8M}{\ell^2} = \frac{8 \times 309{,}900}{(20 \times 12)^2} \times 12 = 516 \text{ lb/ft}$$

Check Stress in Steel

$$f_{bs} = \frac{Mc}{I_t} n = \frac{309{,}900 \times 5.5}{1263} \times 16.1 = 21{,}730 \text{ lb/in}^2 < 23{,}800 \text{ lb/in}^2$$

Therefore, the flexural stress in the wood component controls the maximum load for the moment as assumed.

Check Shearing Stress in Wood

$$V_w = 516\left(10 - \frac{11.25}{12}\right) = 4676 \text{ lb}$$

$$Q = \frac{56.25}{2}\left(\frac{11.25}{4}\right)^2 + \frac{11.0}{2} \times 6.04\left(\frac{11.0}{4}\right) = 170.5 \text{ in}^3$$

$$f_v \text{ (wood)} = \frac{VQ}{I_t t} = \frac{4676 \times 170.5}{1263 \times 11.04} = 57.2 \text{ lb/in}^2$$

$$F'_{vw} = 95 \times 1.15 = 109 \text{ lb/in}^2 > 57.2 \text{ lb/in}^2 \qquad \text{OK}$$

Check Shearing Stress in Steel

$$V_s = 516 \times 10 = 5160 \text{ lb}$$

$$f_v \text{ (steel)} = \frac{VQ}{I_t t} n = \frac{5160 \times 170.5}{1263 \times 11.04} \times 16.1$$

$$= 1016 \text{ lb/in}^2 \ll 14{,}400 \text{ lb/in}^2 \qquad \text{OK}$$

Note: Steel shear stress rarely controls the design of the steel component.

Check Total Deflection

$$\Delta = \frac{5w\ell^4}{384EI_t} = \frac{5 \times 516 \times 20^4 \times 1728}{384 \times 1{,}800{,}000 \times 1263} = 0.817 \text{ in (20.8 mm)}$$

If the limit on deflection is $\ell/240$ for dead load plus applied load,

$$\Delta \text{ (allowable)} = \frac{20 \times 12}{240} = 1.0 \text{ in (25.4 mm)} > 0.817 \text{ in (20.8 mm)} \qquad \text{OK}$$

Fasteners. It is also necessary that an adequate number and size of fasteners be provided in the design to ensure that both wood and steel deflect equally. For fastener design, see Chap. 5.

Summary. The maximum uniform load capacity is 516 lb/ft (7.53 kN/m) and is controlled by the flexural stress in the wood.

The preceding example is a simple combination of two materials and is given to illustrate some of the basic calculations needed in this type of composite member. Many other arrangements for combining these two materials are possible.

Examples of the use of steel to carry tensile forces in combination with a wood compression member are shown in Fig. 10.39. Figure 10.39*a* and *b* illustrates the king and queen post trusses using steel tension rods. They can serve to increase the span length or load capacity of a wood member and have been used to reinforce a two-span beam whose center support is to be removed, or to reinforce a simple-span beam of insufficient capacity for changed conditions. The kind and queen post trusses are statically indeterminate and are discussed in some textbooks. The solution is based on the elastic strain properties of the materials and the equation usually given obtains the force in the post.

The equation for the force in the post of a king post truss and for the forces in the posts of a queen post truss in which the posts are symmetrically placed about the centerline of the flexural member is

$$P = \frac{\int Mm\, dx/EI}{\Sigma u^2L/AE + \int m^2\, dx/EI} \tag{10.8}$$

If the flexural member is divided into a finite number of elements, the integral signs may be replaced by summation signs and the equation becomes

$$P = \frac{\Sigma Mm\, \Delta x/EI}{\Sigma u^2L/AE + \Sigma m^2\, \Delta x/EI} \tag{10.9}$$

where P = force in post of king or queen post truss
A = area of truss member
E = modulus of elasticity of truss member
L = length of truss member
M = moment at center of beam segment due to external loads without redundant member (post or posts)
m = moment at center of beam segment due to unit force in redundant member (post or posts)

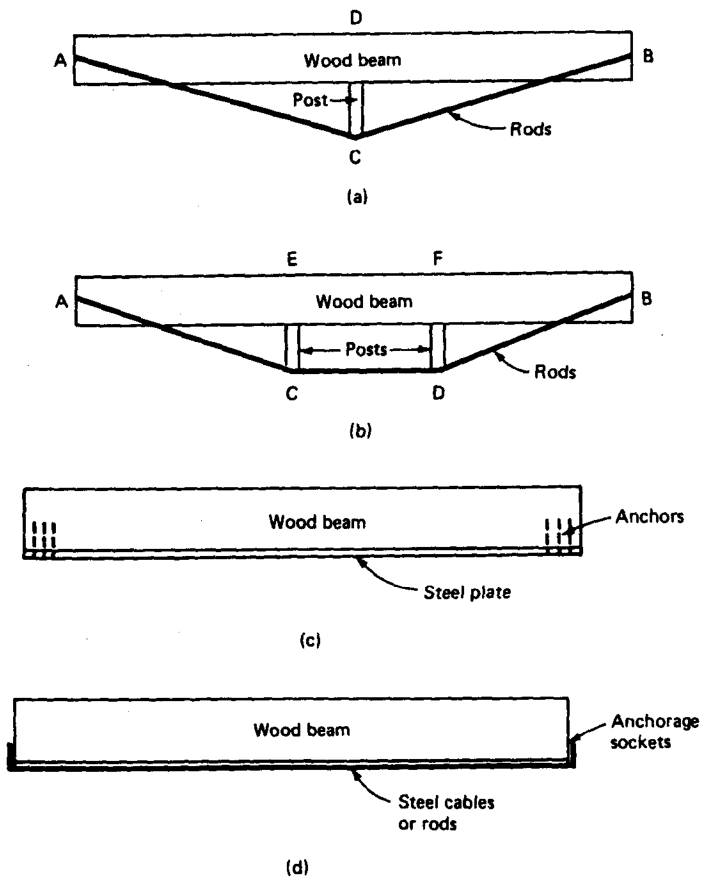


FIG. 10.39 Types of composite beams. (*a*) King post truss; (*b*) queen post truss; (*c*) composite wood-steel beam; (*d*) composite wood-cables/rods beam.

u = force in truss member due to unit force applied to redundant member (post or posts)
Δx = length of beam segment
I = moment of inertia of beam segment

Usually it is not necessary to use a large number of segments for the flexural member (*AB*) of the king or queen truss in order to obtain accurate results, even if a variable-depth member were used, which is unlikely. Four or five segments for each half, as shown in Example 10.8, usually give satisfactory results. The term for axial strains of Eq. (10.9), $\Sigma u^2L/AE$, would involve only four terms for the king post truss, one each for members *AC, CB, CD,* and *ADB* (Fig. 10.39*a*), and due to symmetry, AC would be equal to *CB.* For the queen post truss, six terms would be involved.

After the force in the post or posts has been determined, the forces and moments in other members of the indeterminate structure can be calculated by using the equations of equilibrium. In general, trussed beams should have as much depth as conditions will permit in order to minimize their stresses. To accomplish this may require too much encroachment on available headroom if the king post truss shown in Fig. 10.39*a* is used. Therefore, the queen post truss of Fig. 10.39*b* or a reinforced beam as shown in Fig. 10.39*c* or *d* may be a better solution.

EXAMPLE 10.8 A loading dock 40 ft wide with a 15-ft overhang is to be converted to a simple span by removing an interior existing support and installing a new support at the free end of the overhang (Fig. 10.40*a* and *b*). A queen post truss supporting the uniformly distributed normal load of 1000 lb/ft (14.59 kN/m) is selected.

SOLUTION: To obtain a solution for this problem, it will be necessary to solve Eq. (10.9). To do this, the following assumptions are made:

1. The cross-sectional area of the tension member is assumed to be 2.00 in^2.
2. The beam is divided into 10 equal segments.
3. The moment is assumed constant throughout the segment and the value used is that at the center of the segment.
4. The cross-sectional area of the post is assumed to be 30 in^2.
5. E(wood) = 1600 kips/in^2 and E(stee1) = 29,000 kips/in^2.

Figure 10.40*c* and *d* gives moments M and m, and Fig. 10.40*e* gives the axial forces u, assuming a 1-kip force exists in the posts.

Segment	M, kips·ft	m, kips·ft	Δx, ft	Mm Δx_3 kips^2·ft^3	m^2 Δx_3, kips^2·ft^3	Final moments, kips·ft
1	38	1.6	4	243	10.2	12.8
2	102	4.8	4	1,958	92.2	26.4
3	150	8.0	4	4,800	256.0	24.0
4	182	11.2	4	8,154	501.8	5.6
5	198	12.0	4	9,504	576.0	16.1
Totals				24,659	1436.2	

Since the beam is symmetrical about its centerline, the values for one-half can be multiplied by 2 to obtain the values for the entire beam:

$$\frac{\Sigma Mm\ \Delta x}{EI} = \frac{24{,}659 \times 2}{EI} = \frac{24{,}659 \times 2 \times 1728}{1600 \times 3244} = 16.42 \text{ kips·in}$$

$$\frac{\Sigma m^2\ \Delta x}{EI} = \frac{1436.2 \times 2}{EI} = \frac{1436.2 \times 2 \times 1728}{1600 \times 3244} = 0.956 \text{ kips·in}$$

For axial strain energy, the queen post truss is divided into sections, as shown in Fig. 40*e*.

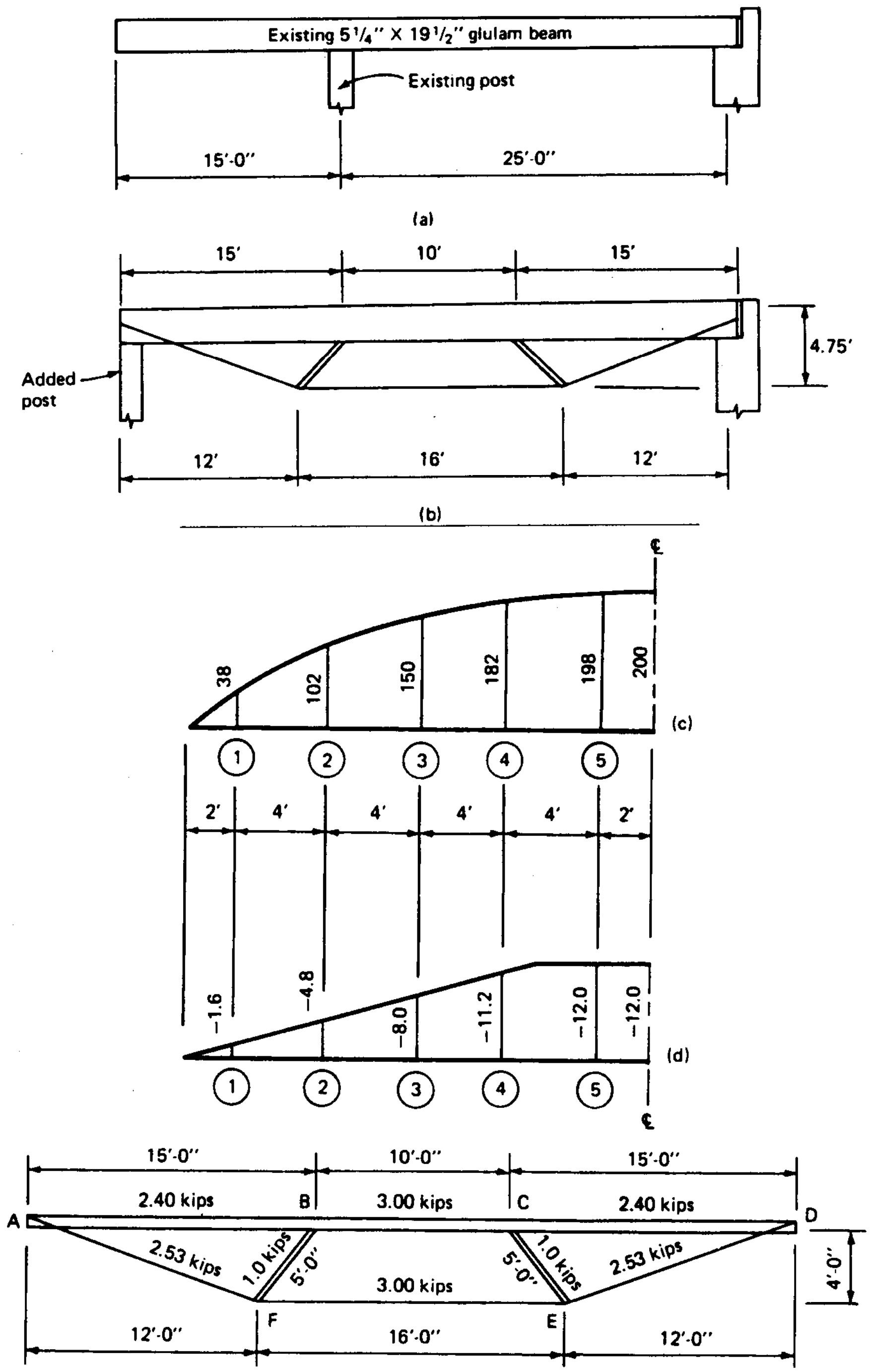


FIG. 10.40 Example 10.8. (*a*) Existing beam and supports; (*b*) modified beam, queen post truss; (*c*) moments *M* (kips·ft); (*d*) moments *m* (kips·ft); (*e*) axial forces *u* (kips) and dimensions.

Section	u, kips	L, in	A, in^2	E, kips/in^2	u^2L/AE, kips·in	force Pu, kips
AB	2.40	180	102	1,600	0.00635	37.80
BC	3.00	120	102	1,600	0.00662	47.25
CD	2.40	280	102	1,600	0.00635	37.80
AF	2.53	152	2	29,000	0.01677	40.95
FE	3.00	192	2	29,000	0.02979	47.25
ED	2.53	152	2	29,000	0.01677	40.95
BF	1.0	60	30	1,600	0.00125	15.78
CE	1.0	60	30	1,600	0.00125	15.75
Total					0.08515	

Solving Eq. (10.9),

$$P = \frac{16.42}{0.08515 + 0.956} = 15.75 \text{ kips}$$

which is the axial force in each post of the queen post truss. Other forces and moments can now be calculated as follows:

$$\text{Axial force} = Pu$$

$$\text{Axial force in rod } (FE) = 15.75 \times 3.00 = 47.25 \text{ kips}$$

$$\text{Axial force in beam } (BC) = 15.75 \times 3.00 = 47.25 \text{ kips}$$

$$\text{Maximum bending moment in beam} = M - mP$$

$$M \text{ (maximum)} = 102 - 4.8 \times 15.75 = 26.4 \text{ kips·ft}$$

which occurs at point 2 in beam.

Check the sizes of the members.

Steel Rod

$$f_t = \frac{47.25}{2.0} = 23.62 \text{ kips/in}^2$$

Try A588 threaded steel rods. F_t (allowable) = 0.33F_u, the allowable tensile stress in threaded fasteners.

$$F_t = 0.33 \times 70 = 23.1 \text{ kips/in}^2 \approx f_t$$

Select rods to provide a steel area slightly greater than 2.0 in^2. Try two 1¼-in-diameter rods:

$$\text{Area} = 2.45 \text{ in}^2 \qquad \text{OK}$$

Post. Design as a pin-ended column and assume a least dimension of 5.5 in, i.e., a nominal 6- by 6-in member,

$$\frac{\ell_e}{d} = \frac{60}{5.5} = 10.9 < 50 \qquad \text{OK}$$

$$F_{cE} = \frac{K_{cE}E}{(\ell_e/d)^2} = \frac{0.3 \times 1{,}600{,}000}{(10.9)^2} = 4040 \text{ lb/in}^2$$

Assume $F_c = 1200$ lb/in^2, No. 1 (WCLIB) Douglas fir (posts and timbers) and a normal load duration $C_D = 1.0$.

$$F_c^* = F_c = 1200 \text{ lb/in}^2$$

$$C_p = \frac{1 + (4040/1200)}{2 \times 0.8} - \sqrt{\left[\frac{1 + (4040/1200)}{2 \times 0.8}\right]^2 - \frac{(4040/1200)}{0.8}}$$

$$= 0.929$$

$$F_c' = F_c^* C_p = 1200 \times 0.929 = 1115 \text{ lb/in}^2$$

$$P_{\text{allowable}} = F_c'A = 1115 \times 5.5 \times 5.5 = 33{,}730 \text{ lb} > 15{,}750 \text{ lb} \qquad \text{OK}$$

Beam. Design as a beam column, assuming that the top edge of the member is laterally supported by the existing roof.

Check Segment AB. The existing beam is a 5¼- by 19½-in Douglas fir glulam with allowable stresses equivalent to a 20F-V3 stress combination (see Appendix, "Reference Data Table A.3"):

$$F_b = 2000 \text{ lb/in}^2 \qquad \text{and} \qquad F_c = 1{,}000 \text{ lb/in}^2$$

$$\left(\frac{f_c}{F_c'}\right)^2 + \frac{f_{b1}}{F_{b1}'[1 - (f_c/F_{cE1})]} \leq 1.0$$

$$f_c < F_{cE1} = \frac{K_{cE}E'}{(\ell_{e1}/d_1)^2}$$

$$\frac{\ell_{e1}}{d_1} = \frac{40 \times 12}{19.5} = 24.6$$

$$F_{cE1} = \frac{0.418 \times 1{,}600{,}000}{(24.6)^2} = 1105 \text{ lb/in}^2$$

$$f_c = \frac{P}{A} = \frac{37{,}800}{99.94} = 378 \text{ lb/in}^2 < 1105 \text{ lb/in}^2 \qquad \text{OK}$$

$$F_{b1} = 2000 \text{ lb/in}^2$$

$$f_{b1} = \frac{M}{S} = \frac{27{,}400 \times 12}{324.8} = 975 \text{ lb/in}^2 < 2000 \text{ lb/in}^2 \qquad \text{OK}$$

$$\left(\frac{378}{1105}\right)^2 + \frac{975}{2000[1 - (378/1105)]} = 0.858 < 1.0 \qquad \text{OK}$$

Check Segment BC

$$f_c = \frac{47{,}250}{99.94} = 473 \text{ lb/in}^2 < 1105 \text{ lb/in}^2 \qquad \text{OK}$$

$$f_{b1} = \frac{24{,}000 \times 12}{324.8} = 887 \text{ lb/in}^2 < 1105 \text{ lb/in}^2 \qquad \text{OK}$$

$$\left(\frac{473}{1105}\right)^2 + \frac{887}{2000[1 - (473/1105)]} = 0.959 < 1.0 \qquad \text{OK}$$

In the reinforced beam shown in Fig. 10.39*c*, the metal plate can be used as a beam splice or to repair a beam damaged on its tension side. Transformed section properties are used and all wood in tension can be ignored, as is done in concrete design using the working stress method. Formulas used in the working stress method for concrete can be modified and used for a reinforced wood beam.

Another method of reinforcing the wood beam is to use steel rods or cables posttensioned to place the wood in compression (Fig. 10.39*d*). This method has been used to repair damaged beams and to reinforce beams of inadequate strength. The calculations are basically the same as for a prestressed concrete beam. The long-term creep and relaxation of the wood can only be estimated. Assume a value of 1½ to 2 times the initial elastic shortening of the wood under load. If an average of 1.75 is used,

$$\Delta = \frac{1.75fL}{E}$$

where Δ = long-term creep and deflection, in (mm)
f = average axial stress, lb/in^2 (Pa)
L = length, in (mm)
E = modulus of elasticity, lb/in^2 (Pa)

The design of the anchorages is simplified if symmetrical bearing plates are used on the ends. Manufacturing difficulties have so far prevented any common use of prestressed glulam beams in a production facility, although some experimentation has been done.[9] Such concepts as a glue joint reinforced with fiberglass or wire mesh or a steel plate glue-bonded to the wood have been and currently (1998), glulom beams are available that are partially reinforced with fiber-reinforced plastics.

If posttensioning is to be done using rods or cables, it is important that these rods or cables maintain a positive upward pressure on the beam. This is best accomplished by placing spacers between the beam and the reinforcing elements so that the reinforcing elements have a slight curvature, as shown in Fig. 10.41*a*. Failure to maintain upward pressure has resulted in the failure of the beam since the prestress can introduce an additional positive moment, as exhibited by Fig. 10.41*b*.

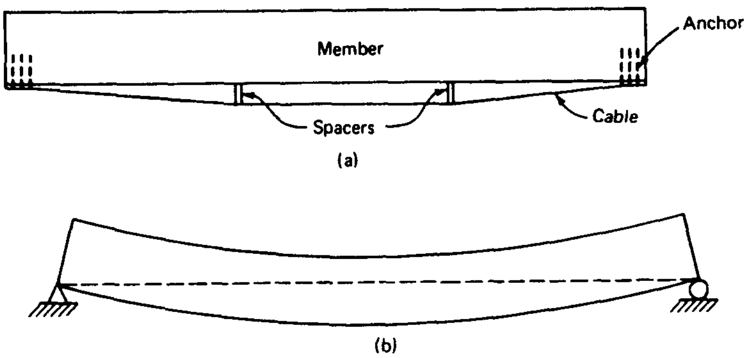


FIG. 10.41 Posttensioning tendons. (*a*) Details, posttensioning tendons; (*b*) improper placement of posttensioning tendons.

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