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SECTION 9

DECKS AND DECK SYSTEMS

Commentary is opposite the text it annotates.

9.1—SCOPE

This Section contains provisions for the analysis and design of bridge decks and deck systems of concrete, metal, wood, or combinations thereof subjected to gravity loads.

For monolithic concrete bridge decks satisfying specific conditions, an empirical design, requiring no analysis, is permitted.

Continuity in the deck and its supporting components is encouraged.

Composite action between the deck and its supporting components is required where technically feasible.

C9.1

Implicit in this Section is a design philosophy that prefers jointless, continuous bridge decks and deck systems to improve the weather- and corrosion-resisting effects of the whole bridge, reduce inspection efforts and maintenance costs, and increase structural effectiveness and redundancy.

9.2—DEFINITIONS

Appurtenances—Curbs, parapets, railings, barriers, dividers, and sign and lighting posts attached to the deck.

Arching Action—A structural phenomenon in which wheel loads are transmitted primarily by compressive struts formed in the slab.

Band—A strip of laminated wood deck within which the pattern of butt joints is not repeated.

Bolster—A spacer between a metal deck and a beam.

Bulkhead—A steel element attached to the side of stress-laminated timber decks to distribute the prestressing force and reduce the tendency to crush the wood.

Cellular Deck—A concrete deck with void-ratio in excess of 40 percent.

Clear Span—The face-to-face distance between supporting components.

Closed Rib—A rib in an orthotropic deck consisting of a plate forming a trough, welded to the deck plate along both sides of the rib.

Closure Joint—A cast-in-place concrete fill between precast components to provide continuity.

Compatibility—The equality of deformation at the interface of elements, components, or both joined together.

Component—A structural element or combination of elements requiring individual design consideration.

Composite Action—A condition in which two or more elements or components are made to act together by preventing relative movement at their interface.

Continuity—In decks, both structural continuity and the ability to prevent water penetration without the assistance of nonstructural elements.

Core Depth—The distance between the top of top reinforcement and the bottom of bottom reinforcement in a concrete slab.

Deck—A component, with or without wearing surface, that supports wheel loads directly and is supported by other components.

Deck Joint—A complete or partial interruption of the deck to accommodate relative movement between portions of a structure.

Deck System—A superstructure, in which the deck is integral with its supporting components, or in which the effects or deformation of supporting components on the behavior of the deck is significant.

Design Span—For decks, the center-to-center distance between the adjacent supporting components, taken in the primary direction.

Effective Length—The span length used in the empirical design of concrete slabs defined in Article 9.7.2.3.

Elastic—A structural response in which stress is directly proportional to strain and no deformation remains upon removal of loading.

Equilibrium—A state where the sum of forces parallel to any axis and the sum of moments about any axis in space are 0.0.

Equivalent Strip—An artificial linear element, isolated from a deck for the purpose of analysis, in which extreme force effects calculated for a line of wheel loads, transverse or longitudinal, will approximate those actually taking place in the deck.

Extreme—Maximum or minimum.

Flexural Continuity—The ability to transmit moment and rotation between components or within a component.

Floorbeam—The traditional name for a cross-beam.

Footprint—The specified contact area between wheel and roadway surface.

Frame Action—Transverse continuity between the deck and the webs of cellular cross-section or between the deck and primary components in large bridges.

Glued Laminated Deck Panel—A deck panel made from wood laminations connected by adhesives.

Governing Position—The location and orientation of a transient load to cause extreme force effects.

Inelastic—The structural response in which stress is not directly proportional to strain and deformation may remain upon removal of loading.

Interface—The location where two elements and/or components are in contact.

Internal Composite Action—The interaction between a deck and a structural overlay.

Isotropic Plate—A plate having essentially identical structural properties in the two principal directions.

Isotropic Reinforcement—Two identical layers of reinforcement, perpendicular to and in touch with each other.

Lateral—Any horizontal or close to horizontal direction.

Laminated Deck—A deck consisting of a series of laminated wood elements that are tightly abutted along their edges to form a continuous surface.

Local Analysis—An in-depth study of strains and stresses in or among components using force effects obtained from global analysis.

Net Depth—The depth of concrete, excluding the concrete placed in the corrugations of a metal formwork.

Open Grid Floor—A metal grid floor not filled or covered with concrete.

Open Rib—A rib in an orthotropic deck consisting of a single plate or rolled section welded to the deck plate.

Orthotropic—A plate having significantly different structural properties in the two principal directions.

Overfill—The concrete above the top of the steel grid of filled or partially filled steel grid deck systems.

Partial Composite Action—A condition in which two or more elements or components are made to act together by decreasing, but not eliminating, relative movement at their interface, or where the connecting elements are too flexible to fully develop the deck in composite action.

Primary Direction—In isotropic decks: direction of the shorter span; in orthotropic decks: direction of the main load-carrying elements.

Secondary Direction—The direction normal to the primary direction.

Segmental Construction—A method of building a bridge utilizing match-cast, prefabricated, or cast-in-place concrete segments joined together by longitudinal post-tensioning.

Shear Connector—A mechanical device that prevents relative movements both normal and parallel to an interface.

Shear Continuity—A condition where shear and displacement are transmitted between components or within a component.

Shear Key—A preformed hollow in the side of a precast component filled with grout or a system of match-cast depressions and protrusions in the face of segments that is intended to provide shear continuity between components.

Skew Angle—The angle between the axis of support relative to a line normal to the longitudinal axis of the bridge; e.g., a zero-degree skew denotes a rectangular bridge.

Spacing—Center-to-center distance of elements or components, such as reinforcing bars, girders, bearings, etc.

Stay-in-Place Formwork—Permanent metal or precast concrete forms that remain in place after construction is finished.

Stiffener Beam—An unsupported beam attached to the underside of a wood deck to enhance lateral continuity.

Stress Range—The algebraic difference between extreme stresses.

Structural Overlay—An overlay bonded to the deck that consists of concretes other than asphaltic concretes.

Tandem—Two closely spaced and mechanically interconnected axles of equal weight.

Tie-Down—A mechanical device that prevents relative movement normal to an interface.

Void—An internal discontinuity of the deck by which its self-weight is reduced.

Voided Deck—Concrete deck in which the area of the voids does not constitute more than 40 percent of the gross area.

Wearing Surface—An overlay or sacrificial layer of the structural deck to protect the structural deck against wear, road salts, and environmental effects. The overlay may include waterproofing.

Wheel—One tire or a pair of tires at one end of an axle.

Wheel Load—One-half of a specified design axle load.

Yield Line—A plastic hinge line.

Yield Line Analysis—A method of determining the load-carrying capacity of a component on the basis of the formation of a mechanism.

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Yield Line Method—A method of analysis in which a number of possible yield line patterns of concrete slabs are examined in order to determine minimum load-carrying capacity.

9.3—NOTATION

A_B = effective bearing area of anchorage bulkhead (in.^{2}) (9.9.5.6.3)
A_s = area of steel bar or strand (in.^{2}) (9.9.5.6.3)
c = depth of the bottom cutout to accommodate a rib in an orthotropic deck (in.) (9.8.3.6.4)
d = effective depth: distance between the outside compressive fiber and the center of gravity of the tensile reinforcement (in.) (C9.7.2.5)
F = nominal bearing resistance of wood across the grain (ksi) (9.9.5.6.3)
$f_{0.5}$ = the surface stress at a distance of 0.5 t from the weld toe (ksi) (9.8.3.4.4)
$f_{1.5}$ = the surface stress at a distance of 1.5 t from the weld toe (ksi) (9.8.3.4.4)
h = depth of deck (in.) (9.9.5.6.3)
L = span length from center-to-center of supports (9.5.2)
P_{BU} = factored compressive resistance of the wood under the bulkhead (kip) (9.9.5.6.3)
P_{pt} = prestressing force per prestressing element (kip) (9.9.5.6.3)
R_{sw} = steel–wood ratio (9.9.5.6.3)
S = effective span length (ft) (9.7.3.2)
s = spacing of prestressing elements (in.) (9.9.5.6.3)
ϕ = resistance factor (9.9.5.6.3)

9.4—GENERAL DESIGN REQUIREMENTS

9.4.1—Interface Action

Decks other than wood and open grid floors shall be made composite with their supporting components, unless there are compelling reasons to the contrary. Noncomposite decks shall be connected to their supporting components to prevent vertical separation.

Shear connectors and other connections between decks, other than open grid floors and wood decks, and their supporting members shall be designed for force effects calculated on the basis of full composite action, whether or not that composite action is considered in proportioning the primary members. The details for transmitting shear across the interface to metal supporting components shall satisfy the applicable provisions of Article 6.6 or Article 7.6.

Force effects between the deck and appurtenances or other components shall be accommodated.

C9.4.1

Composite action is recommended to enhance the stiffness and economy of structures.

Some decks without shear connectors have historically demonstrated a degree of composite action due to chemical bond, friction, or a combination of both that cannot be accounted for in structural design.

It is difficult to design and detail a tie-down device that does not attract shear forces due to transient loads, temperature changes, and fluctuation in moisture content. These forces may loosen or break such devices, and cause fatigue damage in other parts of the floor system and its connections to main members, and to floorbeams in particular.

9.4.2—Deck Drainage

With the exception of unfilled steel grid decks, cross and longitudinal slopes of the deck surface shall be provided as specified in Article 2.6.6.

Structural effects of drainage openings shall be considered in the design of decks.

9.4.3—Concrete Appurtenances

Unless otherwise specified by the Owner, concrete curbs, parapets, barriers, and dividers should be made structurally continuous. Consideration of their structural contribution to the deck should be limited in accordance with the provisions of Article 9.5.1.

C9.4.3

Experience indicates that the interruption of concrete appurtenances at locations other than deck joints does not serve the intended purpose of stress relief. Large cracks, approximately a foot or so away from open joints, have been observed in concrete parapets. The structural participation of these components is usually, but not always, beneficial. One possible negative aspect of continuity is increased cracking in the appurtenance.

9.4.4—Edge Supports

Unless the deck is designed to support wheel loads in extreme positions with respect to its edges, edge supports shall be provided. Nonintegral edge beams shall conform to the provisions of Article 9.7.1.4.

C9.4.4

If the deck joint hardware is integrated with the deck, it may be utilized as a structural element of the edge beam.

9.4.5—Stay-in-Place Formwork for Overhangs

Stay-in-place formwork, other than that in filled steel decks, shall not be used in the overhang of concrete decks.

9.5—LIMIT STATES

9.5.1—General

The structural contribution of a concrete appurtenance to the deck may be considered for service and fatigue but not for strength or extreme event limit states.

For other than the deck overhang, where the conditions specified in Article 9.7.2 are met, a concrete deck may be assumed to satisfy service, fatigue, and fracture and strength limit state requirements and need not meet the other provisions of Article 9.5.

C9.5.1

Exclusion of contribution of an appurtenance at strength limit state is a safety measure in that advantage is not taken of a component that may be damaged, disconnected, or destroyed by a collision.

Article 9.7.2.2 states that the empirical design method does not apply to overhangs.

9.5.2—Service Limit States

At service limit states, decks and deck systems shall be analyzed as fully elastic structures and shall be designed and detailed to satisfy the provisions of Sections 5, 6, 7, and 8.

The effects of excessive deck deformation, including deflection, shall be considered for metal grid decks and other lightweight metal and concrete bridge decks. For these deck systems, the deflection caused by live load plus dynamic load allowance shall not exceed the following criteria:

- L/800 for decks with no pedestrian traffic,
- L/1,000 for decks with limited pedestrian traffic, and
- L/1,200 for decks with significant pedestrian traffic

where:

L = span length from center-to-center of supports.

C9.5.2

Deck deformation refers to local dishing at wheel loads, not to overall superstructure deformation.

The primary objective of curtailing excessive deck deformation is to prevent breakup and loss of the wearing surface. No overall limit can be specified because such limit is a function of the composition of the wearing surface and the adhesion between the deck and the wearing surface. The limits should be established by testing.

Substantial work has been done relating accelerations to user comfort. Acceleration is a function of the fundamental frequency of vibration of the deck on a particular span, and the magnitude of dynamic deflection due to live load. Dynamic deflections are typically 15 percent to 20 percent of static deflections. Analysis shows that static deflections serve well as a proxy for acceleration levels for deck systems.

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9.5.3—Fatigue and Fracture Limit State

Fatigue need not be investigated for:

- concrete decks and
- wood decks as listed in Article 9.9.

Open grid, filled grid, partially filled grid, and unfilled grid decks composite with reinforced concrete slabs shall comply with the provisions of Articles 4.6.2.1, 6.5.3, and 9.8.2.

Steel orthotropic decks shall comply with the provisions of Article 6.5.3. Aluminum decks shall comply with the provisions of Article 7.6.

Concrete decks, other than those in multigirder applications, shall be investigated for the fatigue limit states as specified in Article 5.5.3.

C9.5.3

The provisions that do not require fatigue investigation of certain types of decks are based exclusively on observed performance and laboratory testing.

A series of 35 pulsating load fatigue tests of model slabs indicate that the fatigue limit for the slabs designed by the conventional AASHTO moment methods was approximately three times the service level. Decks based on the isotropic reinforcement method specified in Article 9.7.2 had fatigue limits of approximately twice the service level (deV Batchelor et al., 1978).

9.5.4—Strength Limit States

At strength limit states, decks and deck systems may be analyzed as either elastic or inelastic structures and shall be designed and detailed to satisfy the provisions of Sections 5, 6, 7, and 8.

C9.5.4

These Specifications do not permit an unlimited application of inelastic methods of analysis due to the lack of adequate background research. There are, however, well-established inelastic plate analyses whose use is allowed.

9.5.5—Extreme Event Limit States

Decks shall be designed for force effects transmitted by traffic and combination railings using loads, analysis procedures, and limit states specified in Section 13. Acceptance testing complying with Section 13 may be used to satisfy this requirement.

9.6—ANALYSIS

9.6.1—Methods of Analysis

Approximate elastic methods of analysis specified in Article 4.6.2.1, refined methods specified in Article 4.6.3.2, or the empirical design of concrete slabs specified in Article 9.7 may be used for various limit states as permitted in Article 9.5.

C9.6.1

Analytical methods presented herein should not be construed as excluding other analytical approaches, provided that they are approved by the Owner.

9.6.2—Loading

Loads, load positions, tire contact area, and load combinations shall be in accordance with the provisions of Section 3.

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9.7—CONCRETE DECK SLABS

9.7.1—General

9.7.1.1—Minimum Depth and Cover

Unless approved by the Owner, the depth of a concrete deck, excluding any provision for grinding, grooving, and sacrificial surface, should not be less than 7.0 in.

Minimum cover shall be in accordance with the provisions of Article 5.10.1.

C9.7.1.1

For slabs of depth less than $^{1}/_{20}$ of the design span, consideration should be given to prestressing in the direction of that span in order to control cracking.

Construction tolerances become a concern for thin decks.

Minimum cover requirements are based on traditional concrete mixes and on the absence of protective coating on either the concrete or the steel inside. A combination of special mix design, protective coatings, dry or moderate climate, and the absence of corrosion chemicals may justify a reduction of these requirements provided that the Owner approves.

9.7.1.2—Composite Action

Shear connectors shall be designed in accordance with the provisions of Section 5 for concrete beams and Sections 6 and 7 for metal beams.

C9.7.1.2

Some research efforts have dealt with wood beams composite with concrete decks and steel beams with stressed wood decks, but progress is not advanced to a point which permits codification.

9.7.1.3—Skewed Decks

If the skew angle of the deck does not exceed 25 degrees, the primary reinforcement may be placed in the direction of the skew; otherwise, it shall be placed perpendicular to the main supporting components.

C9.7.1.3

The intent of this provision is to prevent extensive cracking of the deck, which may result from the absence of appreciable reinforcement acting in the direction of principal flexural stresses due to a heavily skewed reinforcement, as shown in Figure C9.7.1.3-1. The somewhat arbitrary 25-degree limit could affect the area of steel as much as ten percent. This was not taken into account because the analysis procedure and the use of bending moment as a basis of design were not believed to be sufficiently accurate to warrant such an adjustment. Owners interested in making this refinement should also consider one of the refined methods of analysis identified in Article 4.6.3.2.

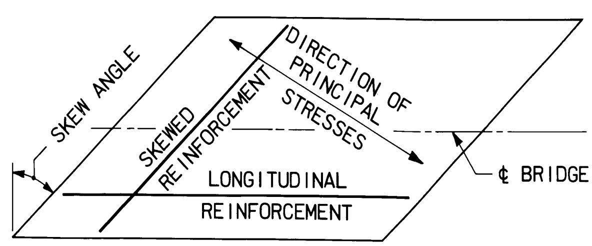

Figure C9.7.1.3-1—Reinforcement Layout

9.7.1.4—Edge Support

Unless otherwise specified, at lines of discontinuity, the edge of the deck shall either be strengthened or be supported by a beam or other line component. The beam or component shall be integrated in or made composite with the deck. The edge beams may be designed as beams whose width may be taken as the effective width of the deck specified in Article 4.6.2.1.4.

Where the primary direction of the deck is transverse, and/or the deck is composite with a structurally continuous concrete barrier, no additional edge beam need be provided.

9.7.1.5—Design of Cantilever Slabs

The overhanging portion of the deck shall be designed for railing impact loads and in accordance with the provisions of Article 3.6.1.3.4.

Punching shear effects at the outside toe of a railing post or barrier due to vehicle collision loads shall be investigated.

C9.7.1.5

An acceptable method of analyzing deck overhangs for railing impact loads is presented in the appendix to Section 13.

Any combination of increasing the depth of the slab, employing special reinforcement extending the slab width beyond the railing, and enlarging base plates under railing posts may be utilized to prevent failure due to punching shear.

9.7.2—Empirical Design

9.7.2.1—General

The provisions of Article 9.7.2 relate exclusively to the empirical design process for concrete deck slabs supported by longitudinal components and shall not be applied to any other Article in this Section, unless specifically permitted.

C9.7.2.1

Extensive research into the behavior of concrete deck slabs discovered that the primary structural action by which these slabs resist concentrated wheel loads is not flexure, as traditionally believed, but a complex internal membrane stress state referred to as internal arching. This action is made possible by the cracking of the concrete in the positive moment region of the design slab and the resulting upward shift of the neutral axis in that portion of the slab. The action is sustained by in-plane membrane forces that develop as a result of lateral confinement provided by the surrounding concrete slab, rigid appurtenances, and supporting components acting compositely with the slab.

The arching creates what can best be described as an internal compressive dome, the failure of which usually occurs as a result of overstraining around the perimeter of the wheel footprint. The resulting failure mode is that of punching shear, although the inclination of the fracture surface is much less than 45 degrees due to the presence of large in-plane compressive forces associated with arching. The arching action, however, cannot resist the full wheel load. There remains a small flexural component for which the specified minimum amount of isotropic reinforcement is more than adequate. The steel has a dual purpose: it provides for both local flexural resistance and global confinement required to develop arching effects (Fang, 1985; Holowka et al., 1980).

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All available test data indicate that the factor of safety of a deck designed by the flexural method specified in the 17th edition of the AASHTO *Standard Specifications for Highway Bridges*, working stress design, is at least 10.0. Tests indicate a comparable factor of safety of about 8.0 for an empirical design. Therefore, even the empirical design possesses an extraordinary reserve strength.

The design of reinforced concrete decks using the concept of internal arching action within the limits specified herein has been verified by extensive nonlinear finite element analysis (Hewitt and deV Batchelor, 1975; Fang et al. 1990). These analyses are accepted in lieu of project-specific design calculations as a preapproved basis of design.

Slabs with the minimum specified reinforcement have demonstrated nearly complete insensitivity to differential displacement among their supports.

The longitudinal bars of the isotropic reinforcement may participate in resisting negative moments at an internal support in continuous structures.

The additional longitudinal reinforcement provided for the slab in the negative moment region of continuous beams and girder-type bridges beyond that required for isotropic reinforcement according to the provisions of Article 9.7.2.5 need not be matched in the perpendicular direction. Theoretically, this portion of the deck will be orthotropically reinforced, but this does not weaken the deck.

9.7.2.2—Application

Empirical design of reinforced concrete decks may be used if the conditions set forth in Article 9.7.2.4 are satisfied.

The provisions of this Article shall not be applied to overhangs.

The overhang should be designed for:

- wheel loads for decks with discontinuous railings and barriers using the equivalent strip method,
- equivalent line load for decks with continuous barriers specified in Article 3.6.1.3.4, and
- collision loads using a failure mechanism as specified in Article A13.2.

C9.7.2.2

Although current tests indicated that arching action may exist in the cantilevered overhang of the slab, the available evidence is not sufficient to formulate code provisions for it (Hays et al., 1989).

As indicated in Article 9.5.5, acceptance testing complying with Section 13 may be used to satisfy design requirements for deck overhangs.

9.7.2.3—Effective Length

For the purpose of the empirical design method, the effective length of slab shall be taken as:

- **For slabs monolithic with walls or beams:** the face-to-face distance, and
- **For slabs supported on steel or concrete girders:** the distance between flange tips, plus the flange overhang, taken as the distance from the extreme flange tip to the face of the web, disregarding any fillets.

C9.7.2.3

Physical tests and analytical investigations indicate that the most important parameter concerning the resistance of concrete slabs to wheel loads is the ratio between the effective length and the depth of the slab.

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In case of nonuniform spacing of supporting components, the effective length, $S_{effective}$, shall be taken as the larger of the deck lengths at the two locations shown in Figure 9.7.2.3-1.

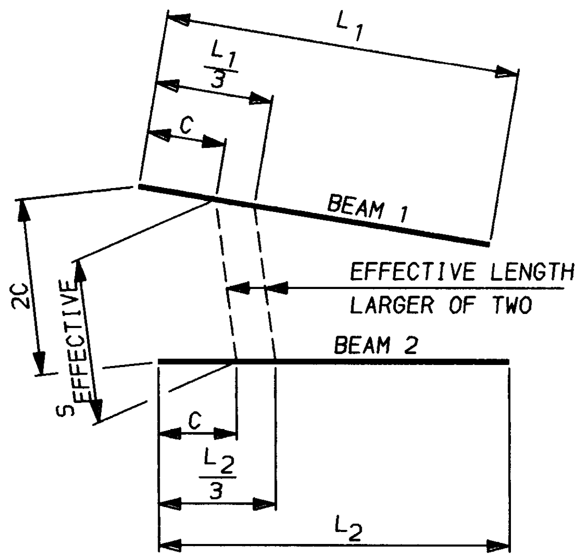

Figure 9.7.2.3-1—Effective Length for Nonuniform Spacing of Beams

9.7.2.4—Design Conditions

For the purpose of this Article, the design depth of the slab shall exclude the loss that is expected to occur as a result of grinding, grooving, or wear. The empirical design may be used only if the following conditions are satisfied:

- Cross-frames or diaphragms are used throughout the cross-section at lines of support;
- For cross-sections involving torsionally stiff units, such as individual separated box beams, either intermediate diaphragms between the boxes are provided at a spacing not to exceed 25.0 ft, or the need for supplemental reinforcement over the webs to accommodate transverse bending between the box units is investigated and reinforcement is provided if necessary;
- The supporting components are made of steel and/or concrete;
- The deck is fully cast-in-place and water cured;
- The deck is of uniform depth, except for haunches at girder flanges and other local thickening;
- The ratio of effective length to design depth does not exceed 18.0 and is not less than 6.0;
- Core depth of the slab is not less than 4.0 in.;
- The effective length, as specified in Article 9.7.2.3, does not exceed 13.5 ft;
- The minimum depth of the slab is not less than 7.0 in., excluding a sacrificial wearing surface where applicable;

C9.7.2.4

Intermediate cross-frames are not needed in order to use the empirical deck design method for cross-sections involving torsionally weak open shapes, such as T- or I-shaped girders.

Use of separated, torsionally stiff beams without intermediate diaphragms can give rise to the situation, shown in Figure C9.7.2.4-1, in which there is a relative displacement between beams and in which the beams do not rotate sufficiently to relieve the moment over the webs. This moment may or may not require more reinforcing than is provided by the empirical deck design.

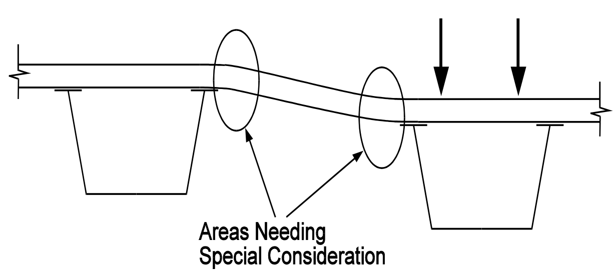

Figure C9.7.2.4-1—Schematic of Effect of Relative Displacements in Torsionally Stiff Cross-Section

All the tests carried out so far were restricted to specimens of uniform depth. Slabs supported by wood beams are not qualified for the empirical design due to the lack of experimental evidence regarding adequate lateral shear transfer between the slab and the relatively soft timber beams.

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No experience exists for effective lengths exceeding 13.5 ft. The 7.0-in. depth is considered an absolute minimum with 2.0-in. cover on top and 1.0-in. cover on the bottom, providing for a reinforced core of 4.0 in., as indicated in Figure C9.7.2.4-2.

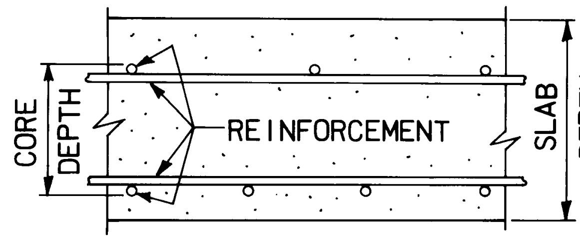

Figure C9.7.2.4-2—Core of a Concrete Slab

The provisions of the *Ontario Highway Bridge Design Code* (1991), based on model test results, do not permit length-to-depth ratios in excess of 15.0. The larger value of 18.0 is based on recent experiments (Hays et al., 1989).

- There is an overhang beyond the centerline of the outside girder of at least 5.0 times the depth of the slab; this condition is satisfied if the overhang is at least 3.0 times the depth of the slab and a structurally continuous concrete barrier is made composite with the overhang;
- The specified 28-day strength of the deck concrete is not less than 4.0 ksi; and
- The deck is made composite with the supporting structural components.

The intention of the overhang provision is to ensure confinement of the slab between the first and the second beam.

The 4.0-ksi limit is based on the fact that none of the tests included concrete with less than 4.0-ksi strength at 28 days. Many jurisdictions specify 4.5-ksi concrete for ensuring reduced permeability of the deck. On the other hand, tests indicate that resistance is not sensitive to the compressive strength, and 3.5 ksi may be accepted with the approval of the Owner.

For the purpose of this Article, a minimum of two shear connectors at 24.0-in. centers shall be provided in the negative moment region of continuous steel superstructures. The provisions of Article 6.10.1.1 shall also be satisfied. For concrete girders, the use of stirrups extending into the deck shall be taken as sufficient to satisfy this requirement.

9.7.2.5—Reinforcement Requirements

Four layers of isotropic reinforcement shall be provided in empirically designed slabs. Reinforcement shall be located as close to the outside surfaces as permitted by cover requirements. Reinforcement shall be provided in each face of the slab with the outermost layers placed in the direction of the effective length. The minimum amount of reinforcement shall be 0.27 in.^{2}/ft of steel for each bottom layer and 0.18 in.^{2}/ft of steel for each top layer. Spacing of steel shall not exceed 18.0 in. Reinforcing steel shall be Grade 60 or better. All reinforcement shall be straight bars, except that hooks may be provided where required.

Both lap splices and mechanical splices shall be allowed. Mechanical splices shall be tested and approved to conform to the limits for slip in Article 5.10.8.4.2b and

C9.7.2.5

Prototype tests indicated that 0.2 percent reinforcement in each of four layers based on the effective depth, *d*, satisfies strength requirements. However, the conservative value of 0.3 percent of the gross area, which corresponds to about 0.27 in.^{2}/ft in a 7.5-in. slab, is specified for better crack control in the positive moment area. Field measurements show very low stresses in negative moment steel; this is reflected by the 0.18-in.^{2}/ft requirement, which is about 0.2 percent reinforcement steel. The additional intent of this low amount of steel is to prevent spalling of the deck due to corrosion of the bars or wires.

Welded splices are not permitted due to fatigue considerations. Tested and preapproved mechanical splices may be permitted when lapping of reinforcing is

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for fatigue in Article 5.5.3.4. Sleeve wedge-type couplers shall not be permitted on coated reinforcing.

not possible or desirable, as often occurs in staged construction and widenings. Sleeve wedge-type couplers will not be permitted on coated reinforcing due to stripping of the coating.

If the skew exceeds 25 degrees, the specified reinforcement in both directions shall be doubled in the end zones of the deck. Each end zone shall be taken as a longitudinal distance equal to the effective length of the slab specified in Article 9.7.2.3.

The intent of this provision is crack control. Beam slab bridges with a skew exceeding 25 degrees have shown a tendency to develop torsional cracks due to differential deflections in the end zone (OHBDC, 1991). The extent of cracking is usually limited to a width that approximates the effective length.

9.7.2.6—Deck with Stay-in-Place Formwork

For decks made with corrugated metal formwork, the design depth of the slab shall be assumed to be the minimum concrete depth.

Stay-in-place concrete formwork shall not be permitted in conjunction with empirical design of concrete slabs.

C9.7.2.6

Concrete in the troughs of the corrugated metal deck is ignored due to lack of evidence that it consistently contributes to the strength of the deck. Reinforcement should not be placed directly on corrugated metal formwork.

The empirical design is based on a radial confinement around the wheel load, which may be weakened by the inherent discontinuity of the bottom reinforcement at the boundaries between formwork panels. Limited tests carried out on flexurally designed slabs with stay-in-place concrete formwork indicate a punching shear failure mode, but somewhat less resistance than that provided by fully cast-in-place slabs. The reason for this decrease is that the discontinuity between the panels intercepts, and thus prevents, the undisturbed formation of the frustum of a cone where punching shear occurs (Buth et al., 1992).

9.7.3—Traditional Design

9.7.3.1—General

The provisions of this Article shall apply to concrete slabs that have four layers of reinforcement, two in each direction, and that comply with Article 9.7.1.1.

C9.7.3.1

The traditional design is based on flexure. The live load force effect in the slab may be determined using the approximate methods of Article 4.6.2.1 or the refined methods of Article 4.6.3.2.

9.7.3.2—Distribution Reinforcement

Reinforcement shall be placed in the secondary direction in the bottom of slabs as a percentage of the primary reinforcement for positive moment as follows:

- For primary reinforcement parallel to traffic:

$$100/\sqrt{S} \leq 50 \text{ percent}$$

- For primary reinforcement perpendicular to traffic:

$$220/\sqrt{S} \leq 67 \text{ percent}$$

where:

S = the effective span length taken as equal to the effective length specified in Article 9.7.2.3 (ft)

9.7.4—Stay-in-Place Formwork

9.7.4.1—General

Stay-in-place formwork shall be designed to be elastic under construction loads. The construction load shall not be taken to be less than the weight of the form and the concrete slab plus 0.050 ksf.

Flexural stresses due to unfactored construction loads shall not exceed:

- 75 percent of the yield strength of steel, or
- 65 percent of the 28-day compressive strength for concrete in compression or the modulus of rupture in tension for prestressed concrete form panels.

The elastic deformation caused by the dead load of the forms, plastic concrete, and reinforcement shall not exceed:

- **For form span lengths of 10.0 ft or less:** the form span length divided by 180 but not exceeding 0.50 in.; or
- **For form span lengths greater than 10.0 ft:** the form span length divided by 240 but not exceeding 0.75 in.

C9.7.4.1

The intent of this Article is to prevent excessive sagging of the formwork during construction, which would result in an unanticipated increase in the weight of the concrete slab.

Deflection limits are specified to ensure adequate cover for reinforcing steel and to account for all dead load in the design.

9.7.4.2—Steel Formwork

Panels shall be specified to be tied together mechanically at their common edges and fastened to their support. No welding of the steel formwork to the supporting components shall be permitted, unless otherwise shown in the contract documents.

Steel formwork shall not be considered to be composite with a concrete slab.

C9.7.4.2

For steel stay-in-place formwork, it has been common to provide an allowance for the weight of the form and additional concrete, with the provision added to the contract documents that if the allowance is exceeded by the Contractor's choice, the Contractor is responsible for showing that the effects on the rest of the bridge are acceptable or providing additional strengthening as needed at no cost to the Owner. The customary allowance has been 0.015 ksf, but this should be reviewed if form spans exceed about 10.0 ft.

9.7.4.3—Concrete Formwork

9.7.4.3.1—Depth

The depth of stay-in-place concrete should neither exceed 55 percent of the depth of the finished deck slab nor be less than 3.5 in.

C9.7.4.3.1

Thousands of bridges have successfully been built with a depth ratio of 43 percent or somewhat higher; 55 percent is believed to be a practical limit, beyond which cracking of the cast-in-place concrete at the panel interface may be expected.

9.7.4.3.2—Reinforcement

Concrete formwork panels may be prestressed in the direction of the design span.

If the precast formwork is prestressed, the strands may be considered as primary reinforcement in the deck slab.

Transfer and development lengths of the strands shall

C9.7.4.3.2

The transfer and development lengths for epoxy-coated strands with alkali-resistant hard particles embedded in the coating may be less than that for uncoated strands. Where epoxy-coated strands are used, this value should be determined by testing.

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be investigated for conditions during construction and in service.

Prestressing strands and/or reinforcing bars in the precast panel need not be extended into the cast-in-place concrete above the beams.

Tests indicate no difference between constructions with and without reinforcement extended into the cast-in-place concrete over the beams (Bieschke and Klingner, 1982). The absence of extended reinforcement, however, may affect transverse load distribution due to a lack of positive moment continuity over the beams or may result in reflective cracking at the ends of the panel. In addition to transverse cracking, which usually occurs at the panel joints due to creep and shrinkage, the latter may appear unseemly and/or make the construction of this type of deck questionable where deicing salts are used.

If used, bottom distribution reinforcement may be placed directly on the top of the panels. Splices in the top primary reinforcement in deck slab shall not be located over the panel joints.

The concrete cover below the strands should not be less than 0.75 in.

9.7.4.3.3—Creep and Shrinkage Control

The age of the panel concrete at the time of placing the cast-in-place concrete shall be such that the difference between the combined shrinkage and creep of the precast panel and the shrinkage of the cast-in-place concrete is minimized.

The upper surface of the panels shall be specified to be roughened in such a manner as to ensure composite action with the cast-in-place concrete.

C9.7.4.3.3

The objective of this Article is to minimize interface shear stresses between the precast panel and the cast-in-place concrete and to promote good bond. Normally, no bonding agents and/or mechanical connectors are needed for composite action.

9.7.4.3.4—Bedding of Panels

The ends of the formwork panels shall be supported on a continuous mortar bed or shall be supported during construction in such a manner that the cast-in-place concrete flows into the space between the panel and the supporting component to form a concrete bedding.

C9.7.4.3.4

Setting screws, bituminous fiber boards, neoprene glands, etc., may be appropriate as temporary supports. In the past, some jurisdictions have had bad experience where prestressed concrete panels were supported only by flexible materials. Creep due to prestress had apparently pulled the panel ends away from cast-in-place concrete. Load was transferred to the flexible panel supports, which compressed, resulting in excessive reflective cracking in the cast-in-place concrete.

9.7.5—Precast Deck Slabs on Girders

9.7.5.1—General

Both reinforced and prestressed precast concrete slab panels may be used. The depth of the slab, excluding any provision for grinding, grooving, and sacrificial surface, shall not be less than 7.0 in.

9.7.5.2—Transversely Joined Precast Decks

Flexurally discontinuous decks made from precast panels and joined together by shear keys may be used. The design of the shear key and the grout used in the key shall be approved by the Owner. The provisions of Article 9.7.4.3.4 may be applicable for the design of bedding.

C9.7.5.2

The shear keys tend to crack due to wheel loads, warping, and environmental effects, leading to leaking of the keys and decreased shear transfer. The relative movement between adjacent panels tends to crack the overlay, if present. Therefore, this construction is not recommended for the regions where the deck may be exposed to salts.

9.7.5.3—Longitudinally Post-Tensioned Precast Decks

The precast components may be placed on beams and joined together by longitudinal post-tensioning. The minimum average effective prestress shall not be less than 0.25 ksi.

The transverse joint between the components and the block-outs at the coupling of post-tensioning ducts shall be specified to be filled with a nonshrink grout having a minimum compressive strength of 5.0 ksi at 24 hours.

Block-outs shall be provided in the slab around the shear connectors and shall be filled with the same grout upon completion of post-tensioning.

C9.7.5.3

Decks made flexurally continuous by longitudinal post-tensioning are the more preferred solution because they behave monolithically and are expected to require less maintenance on the long-term basis.

The post-tensioning ducts should be located at the center of the slab cross-section. Block-outs should be provided in the joints to permit the splicing of post-tensioning ducts.

Panels should be placed on the girders without mortar or adhesives to permit their movement relative to the girders during prestressing. Panels can be placed directly on the girders or located with the help of shims of inorganic material or other leveling devices. If the panels are not laid directly on the beams, the space therein should be grouted at the same time as the shear connector block-outs.

A variety of shear key formations has been used in the past. Recent prototype tests indicate that a "V" joint may be the easiest to form and to fill.

9.7.6—Deck Slabs in Segmental Construction

9.7.6.1—General

The provisions of this Article shall apply to the top slabs of post-tensioned girders whose cross-sections consist of single or multicell boxes. The slab shall be analyzed in accordance with the provisions of Article 4.6.2.1.6.

9.7.6.2—Joints in Decks

Joints in the decks of precast segmental bridges may be dry joints, epoxied match-cast surfaces, or cast-in-place concrete.

Dry joints should be used only in regions where deicing salts are not applied.

The strength of cast-in-place concrete joints shall not be less than that of the precast concrete. The width of the concrete joint shall permit the development of reinforcement in the joint or coupling of ducts if used, but in no case shall it be less than 12.0 in.

C9.7.6.2

Dry joints in the deck, with or without a nonstructural sealant, have been observed to permit percolation of water due to shrinkage as well as creep and temperature-induced warping of segments. Both epoxied match-cast and cast-in-place concrete joints permitted herein should produce water-tight joints. The 12.0-in. cast-in-place closure joint is believed to provide a better riding profile if the deck is not overlaid.

A combination joint in which only the deck part of a match-cast joint is epoxied should be avoided.

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9.8—METAL DECKS

9.8.1—General

Metal decks shall be designed to satisfy the requirements of Sections 6 and 7. The tire contact area shall be determined as specified in Article 3.6.1.2.5.

9.8.2—Metal Grid Decks

9.8.2.1—General

Grid deck shall be composed of main elements that span between beams, stringers, or cross-beams and secondary members that interconnect and span between the main elements. The main and secondary elements may form a rectangular or diagonal pattern and shall be securely joined together. All intersections of elements in open grid floors, partially filled grid decks, and unfilled grid decks composite with reinforced concrete slabs shall be welded.

Force effects may be determined using one of the following methods:

- The approximate methods specified in Article 4.6.2.1, as applicable;
- Orthotropic plate theory;
- Equivalent grid; or
- Design aids provided by the manufacturers, if the performance of the deck is documented and supported by sufficient technical evidence.

One of the accepted approximate methods is based on transformed cross-section area. Mechanical shear transfer devices, including indentations, embossment, sand coating of surface, and other appropriate means may be used to enhance the composite action between elements of the grid and the concrete fill.

If a filled or partially filled grid deck, or an unfilled grid deck composite with reinforced concrete slab is considered to be composite with its supporting members for the purpose of designing those members, the effective width of slab in composite section shall be as specified in Article 4.6.2.6.1.

C9.8.2.1

Research has shown that welds between elements in partially filled grids “may be very important to the survival of the cross bar” (Gangarao et al., 1992).

Laboratory tests have shown that section properties of filled and partially filled grids, computed by the transformed area method, are conservative (Gangarao et al., 1992). Tests have also demonstrated that a monolithic concrete overpour may be considered fully effective in determining section properties.

Filled and partially filled grid decks and unfilled grid decks composite with reinforced concrete slabs have better potential for composite action with the supporting components due to their considerable in-plane rigidity.

In computing section properties, omit any effect of concrete in tension (e.g., below the neutral axis in positive bending, and above the neutral axis in negative bending).

The modular ratios may be applied to the composite action of concrete fill with grid deck in flexure and to the composite action between the deck and its supporting beams.

Field tests of systems consisting of unfilled grid decks composite with reinforced concrete slabs and stringers or floorbeams demonstrate significant levels of composite action, with the effective width being at least 12.0 times the overall thickness of the deck, including the grid portion and the structural reinforced concrete slab.

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9.8.2.2—Open Grid Floors

Open grid floors shall be connected to the supporting components by welding or by mechanically fastening at each main element. Where welding is used to make this connection, a single-sided 3.0-in. long weld or a 1.5-in. weld on each side of the main element may be used.

Unless evidence is provided to the contrary, welding within open grid floors should be considered as a Category E detail, and the provisions of Article 6.6 shall apply.

Ends and edges of open grid floors that may be exposed to vehicular traffic shall be supported by closure bars or other effective means.

C9.8.2.2

Long-term experience indicates that even where there is an apparently insignificant degree of composite action between the deck and its supporting components, high stresses may develop at their interface, resulting in local failures and separation of the deck. Therefore, the requirement to connect at each intersection of a main bar, as indicated, applies even to open grid floors.

9.8.2.3—Filled and Partially Filled Grid Decks

9.8.2.3.1—General

These decks shall consist of a metal grid or other metal structural system filled either completely or partially with concrete.

The provisions of Article 9.8.2.1 shall apply to filled and partially filled grid decks.

Where possible, a 1.75-in. thick structural overfill should be provided.

Filled and partially filled grids shall be attached to supporting components by welding or shear studs to transfer shear between the two surfaces.

C9.8.2.3.1

Full-scale tests on systems consisting of partially filled grid decks and stringers demonstrated significant levels of composite action, with the effective width being at least 12.0 times the depth of the deck. Under load, the deck strain readings across the width of the deck were nearly uniform, with extremely small slip recorded at the deck-stringer interface.

In order to activate the deck in composite action, large shear forces need be resisted at the interface. A preferred method of shear transfer is by welded studs encased in a concrete haunch, similar to that illustrated in Figure C9.8.2.3.1-1.

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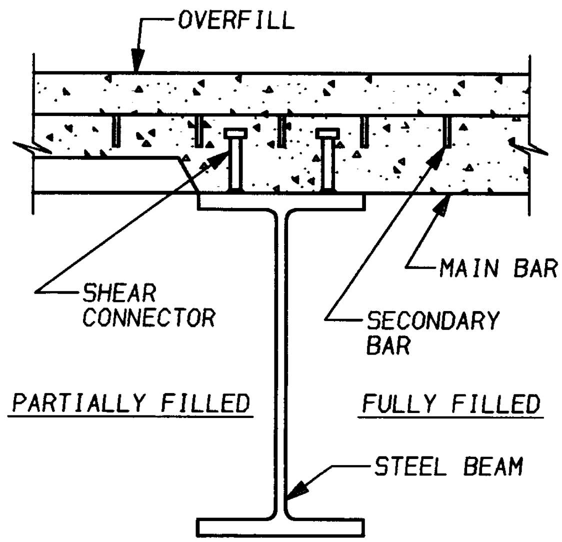

Figure C9.8.2.3.1-1—An Acceptable Shear Connection of Partially and Fully Filled Grid Decks to Beams

9.8.2.3.2—Design Requirements

Design of filled and partially filled grid decks shall be in accordance with the provisions of Article 9.8.2.1 and Article 4.6.2.1.8.

The concrete portion of filled and partially filled grid decks shall be in accordance with the general provisions of Section 5 relating to long-term durability and integrity.

For cast-in-place applications, weight of concrete fill shall be assumed to be carried solely by the metal portion of the deck. The transient loads and superimposed permanent loads may be assumed to be supported by the grid bars and concrete fill acting compositely. A structural overfill may be considered as part of the composite structural deck. Where a structural overfill is provided, the design depth of the deck shall be reduced by a provision for loss that is expected as a result of grinding, grooving, or wear of the concrete.

C9.8.2.3.2

The presence of a composite structural overlay improves both the structural performance and riding quality of the deck.

9.8.2.3.3—Fatigue and Fracture Limit State

All connections among the elements of the steel grid in a fully filled grid deck or the connections within the concrete fill of a partially filled grid deck need not be investigated for fatigue in the local negative moment region of the deck (e.g., negative moment in the deck over a longitudinal stringer or floorbeam) when the deck is designed with a continuity factor of 1.0.

C9.8.2.3.3

Fully filled and partially filled steel grid decks must be checked for fatigue only in the positive moment region (mid span of the deck). However, the deck fatigue moment should be calculated for a simple span configuration (C = 1.0) regardless of the actual span configuration.

The fatigue category to be used for fatigue investigation should be determined by appropriate laboratory testing in positive and negative bending. The fatigue category for welds and punchouts shall not be taken as better than Category C, which has been shown by testing to be appropriate for most details of grid decks constructed with concrete.

The small fillet welds used in the fabrication of grid decks are generally less than 1.5 in. long, but are not considered “tack welds.” In grid decks, “tack welds” refers

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only to small welds used to attach sheet metal pans that serve only as forms for concrete poured onto or into the grid.

Where possible, form pans should be attached by means other than tack welding.

9.8.2.4—Unfilled Grid Decks Composite with Reinforced Concrete Slabs

9.8.2.4.1—General

An unfilled grid deck composite with reinforced concrete slab consists of a reinforced concrete slab that is cast on top of and is composite with an unfilled steel grid. Composite action between the concrete slab and the grid deck shall be ensured by providing shear connectors or other means capable of resisting horizontal and vertical components of interface shears.

Composite action between the grid deck and the supporting components should be ensured by mechanical shear connectors.

Unless otherwise specified, provisions of Article 9.8.2.1 shall apply.

Discontinuities and cold joints in such decks should be kept to a minimum.

C9.8.2.4.1

This bridge deck combines the attributes of a concrete deck and a steel grid deck.

An acceptable way of providing composite action between the deck and the supporting components is shown in Figure C9.8.2.4.1-1.

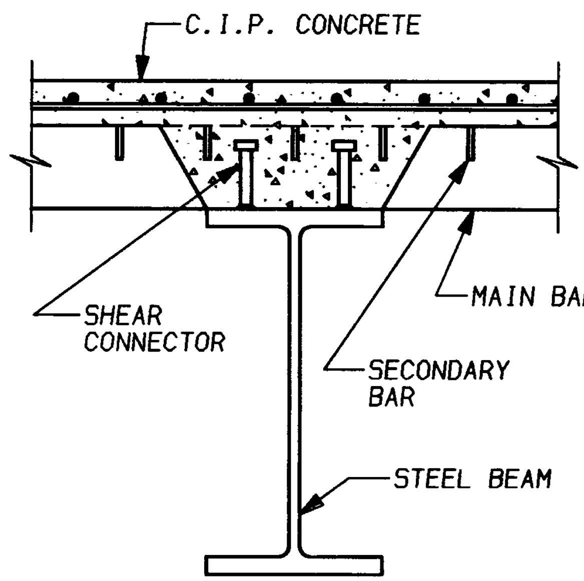

Figure C9.8.2.4.1-1—An Acceptable Shear Connection of Unfilled Grid Decks Composite with Reinforced Concrete Slabs to Beams

9.8.2.4.2—Design

Design of unfilled grid decks composite with reinforced concrete slabs shall be in accordance with the provisions of Article 9.8.2.1 and Article 4.6.2.1.8. The design depth of the deck shall be reduced by a provision for loss that is expected as a result of grinding, grooving, or wear of the concrete.

The reinforced concrete portion of unfilled grid decks composite with reinforced concrete slabs shall be in accordance with the general provisions of Section 5 relating to long-term durability and integrity.

In the concrete slab, one layer of reinforcement in each principal direction may be used.

For cast-in-place applications, weight of concrete slab shall be assumed to be carried solely by the grid portion of the deck. The transient loads and superimposed permanent loads may be assumed to be supported by the composite section.

C9.8.2.4.2

For the purpose of design, the deck can be subdivided into intersecting sets of composite concrete/steel beams.

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The interface between the concrete slab and the metal system shall satisfy the provisions of Article 6.10.10. Acceptable methods of shear connection shall include tertiary bars to which 0.5-in. diameter rebar or round studs have been welded, or the punching of holes at least 0.75 in. in size in the top portion of the main bars of the grid which are embedded in the reinforced concrete slab by a minimum of 1.0 in.

9.8.2.4.3—Fatigue Limit State

The internal connection between the elements of the steel grid in unfilled grid decks composite with reinforced concrete slabs shall be investigated for fatigue.

Unless evidence is provided to the contrary, tack welds attaching horizontal form pans to metal grids shall be considered Category E′ details.

The composite reinforced concrete slab shall be included in the calculation of stress range.

C9.8.2.4.3

The fatigue category to be used for fatigue investigation should be determined by appropriate laboratory testing in positive and negative bending. The fatigue category for welds and punchouts shall not be better than Category C, which has been shown by testing to be appropriate for most details of grid decks constructed with concrete.

The small fillet welds used in the fabrication of grid decks are generally less than 1.5 in. long, but are not considered “tack welds.” In grid decks, “tack welds” refers only to small welds used to attach sheet metal pans that serve only as forms for concrete poured onto or into the grid.

Where possible, form pans should be attached by means other than tack welding.

9.8.3—Orthotropic Steel Decks

9.8.3.1—General

Orthotropic steel decks shall consist of a deck plate stiffened and supported by longitudinal ribs and transverse floorbeams. The deck plate shall act as a common flange of the ribs, the floorbeams, and the main longitudinal components of the bridge.

In rehabilitation, if the orthotropic deck is supported by existing floorbeams, the connection between the deck and the floorbeam should be designed for full composite action, even if the effect of composite action is neglected in the design of floorbeams. Where practical, connections suitable to develop composite action between the deck and the main longitudinal components should be provided.

C9.8.3.1

The intent of this Article is to ensure the structural integrity of the deck and its structural participation with the cross-beams and the primary longitudinal components, as appropriate. Any structural arrangement in which the orthotropic deck is made to act independently from the main components is undesirable.

9.8.3.2—Wheel Load Distribution

A 45-degree distribution of the tire pressure may be assumed to occur in all directions from the surface contact area to the middle of the deck plate. The tire footprint shall be as specified in Article 3.6.1.2.5.

C9.8.3.2

The 45-degree distribution is the traditional, conservative assumption.

9.8.3.3—Wearing Surface

The wearing surface should be regarded as an integral part of the total orthotropic deck system and shall be specified to be bonded to the top of the deck plate.

C9.8.3.3

Wearing surfaces acting compositely with the deck plate may reduce deformations and stresses in orthotropic decks.

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The contribution of a wearing surface to the stiffness of the members of an orthotropic deck may be considered if structural and bonding properties are satisfactorily demonstrated over the temperature range of −20°F to +120°F. If the contribution of the wearing surface to stiffness is considered in the design, the required engineering properties of the wearing surface shall be indicated in the contract documents.

Force effects in the wearing surface and at the interface with the deck plate shall be investigated with consideration of engineering properties of the wearing surface at anticipated extreme service temperatures.

The long-term composite action between deck plate and wearing surface shall be documented by both static and cyclic load tests.

For the purpose of designing the wearing surface and its adhesion to the deck plate, the wearing surface shall be assumed to be composite with the deck plate, regardless of whether the deck plate is designed on that basis.

The deck stiffening effect of the wearing surface is dependent upon its thickness, the elastic modulus which is dependent on temperature and the load application, e.g., static or dynamic, and bond characteristics.

The combination of temperature and live load effects has resulted in debonding of some wearing surfaces in the field, which should be regarded as failure of the wearing surface. The Designer should consider past experience in selection of a wearing surface and in determination of its long-term contribution to the structural system.

Wearing surface cracking is related to stresses exceeding tensile strength of surfacing material. Flexural stresses in surfacing may be reduced by limiting local deck flexibility, as indicated in Article 2.5.2.6.2. Safety against surfacing cracking may be best assured by using surfacing materials with semiplastic properties or with low elastic modulus not subject to much variation with temperature.

The wearing surface plays an important role in improving skid resistance, distributing wheel loads, and protecting the deck against corrosion and abuse.

Selection or design of a wearing surface should include evaluation of the following functional requirements:

- Sufficient ductility and strength to accommodate expansion, contraction, and imposed deformation without cracking or debonding;
- Sufficient fatigue strength to withstand flexural stresses due to composite action of the wearing surface with the deck plate resulting from local flexure;
- Sufficient durability to resist rutting, shoving, and wearing;
- Imperviousness to water and motor vehicle fuels and oils;
- Resistance to deterioration from deicing salts; and
- Resistance to aging and deterioration due to solar radiation.

9.8.3.4—Analysis of Orthotropic Decks

9.8.3.4.1—General

Design of orthotropic decks shall be based on appropriate use of the three levels of analysis specified herein. The fatigue limit state shall be investigated using at least one of the three levels of design specified in Articles 9.8.3.4.2 through 9.8.3.4.4. Strength, service, and extreme event limit states, as appropriate, and constructability criteria shall be investigated using Level 2 design.

C9.8.3.4.1

The updated design approach for orthotropic deck bridges is based on the following considerations:

- Simplified methods do not currently exist which can evaluate the fatigue limit state at all fatigue-sensitive details;
- Design cannot be accomplished by detailing requirements alone due to a lack of tested and established standard deck panel details;
- Refined analysis for new designs will add engineering cost and potentially limit use for routine span arrangements; and

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- Verification testing of every design adds unnecessary cost and has the potential to delay construction.

Hence, design verification of orthotropic steel bridge decks requires a new approach. Since many of the controlling aspects of orthotropic deck panel design are local rather than global demands, a well-designed and detailed panel has the potential to be reused in future applications and become a standardized modular component. Therefore, the required effort for design can vary depending on the application and available test data. These different levels of required effort for design or design levels are summarized as follows:

- Level 1 design is based on little or no structural analysis but is done by selection of details that are verified to have adequate resistance by experimental testing (new or previous). When appropriate laboratory tests have been conducted for previous projects or on specimens similar in design and details to those proposed for a new project, the previous tests may be used as the basis for the design on the new project. All details must provide a level of safety consistent with the AASHTO *LRFD Bridge Design Specifications*.
- Level 2 design is based on simplified one-dimensional or two-dimensional analysis of certain panel details where such analysis is sufficiently accurate or for certain details that are similar to previously tested details as described in Level 1. Calculations consider only nominal stresses and not local stress concentrations. This is primarily intended to allow incremental improvement of previously tested details as verified by Level 1. Approximate analysis of both open rib and closed rib decks may be based on the Pelikan–Esslinger method presented by Wolchuck (1963) and Troitsky (1987). This method gives conservative values of global force effects in the orthotropic deck supported on longitudinal edge girders. Load distribution of adjacent transversely located wheel loads on decks with closed ribs is discussed in Wolchuck (1963).
- Level 3 design is based on refined three-dimensional analysis of the panel to quantify the local stresses to the most accurate extent reasonably expected from a qualified design engineer experienced in refined analysis. Level 3 designs will be dictated by the requirements to provide safety against fatigue failure. If no test data is available for a panel, design Level 3 is required unless it can be proven that the local distortional mechanisms (floorbeam distortion and rib distortion) will not lead to fatigue cracking. For design of panels for bridge redecking applications, design Level 3 should always be used unless an exception is approved by the Owner.

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Level 3 design is an extension of current AASHTO methodology for fatigue evaluation by nominal stresses. The proposed Level 3 design method is also similar to methodology applied by the American Petroleum Institute (API) and American Welding Society (AWS, 2004) and is well documented by the International Institute of Welding (IIW, 2007). It is used extensively for the fatigue evaluation of tubular structures and plate-type structures with complex geometries by various industries, where there is no clearly defined nominal stress due to complicated geometric effects, conditions very similar to orthotropic deck details. This approach recognizes that fatigue damage is caused by stress raisers that exist at details and attempts to quantify them by refined analysis rather than accounting for the stresses risers using classification into general categories.

Research has demonstrated that evaluating the local structural stress and evaluating the stress range with AASHTO Category C provides a reliably conservative assessment of the weld toe cracks at orthotropic deck panel welded joints subjected to distortional stresses. The AASHTO Category C curve is similar to the curves provided in the Eurocode (ECS, 1992) and IIW (2007) for evaluation of welded details. Furthermore, research by Dexter et al. (1994) found that the AASHTO Category C curve provides the 97.5 percent survival lower bound for welded details on flexible plates subjected to combined in-plane and out-of-plane stresses in all cases where local stress measured 5 mm from weld toe was used for the fatigue life stress range. The work by Connor and Fisher (2006) also found similar results. This approach is predicated on the modeling and stress analysis being conducted by the method prescribed by Level 3 design.

The procedures for calculating local structural stress for welded connections are representative of a mesh sizing where the length and width of each individual shell or solid element is equivalent to the thickness, *t,* of the connected component. For modeling with other mesh spacing, different procedures are required for extrapolation of the local structural stress and are presented in more detail in Recommendations *for Fatigue Design of Welded Joints and Components* (IIW, 2007) and *Manual for Design, Construction, and Maintenance of Orthotropic Steel Deck Bridges* (FHWA, 2012).

9.8.3.4.2—Level 1 Design

Orthotropic deck panels and details verified by appropriate full-scale laboratory testing may be used without consideration of design Levels 2 and 3 provided the testing protocol envelopes the structural design loads and stresses for the new application. Test loading should be equivalent to the maximum truck load; stress ranges at details should accurately simulate expected in-service demands and should have accurate boundary conditions.

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For finite fatigue life design, the resistance shall provide 97.5 percent confidence of survival. For infinite fatigue life design, the constant amplitude fatigue limit (CAFL) should be exceeded no more than one in 10,000 cycles (0.01 percent). Full-scale tests should include a minimum of two rib spans with three floorbeams.

Previously verified Level 1 designs that have been verified by laboratory testing may be used as the basis for design on new projects without additional testing, subject to approval by the Owner.

9.8.3.4.3—Level 2 Design

9.8.3.4.3a—General

Details not subjected to local distortional mechanisms similar to those previously proven by appropriate laboratory testing or those that have been proven effective by Level 3 designs and long term observation while subjected to the appropriate loads may be verified considering only nominal stresses determined from simplified analysis.

9.8.3.4.3b—Decks with Open Ribs

The rib may be analyzed as a continuous beam supported by the floorbeams.

For rib spans not exceeding 15.0 ft, the load on the one rib due to wheel loads may be determined as the reaction of transversely continuous deck plate supported by rigid ribs. For rib spans greater than 15.0 ft, the effect of rib flexibility on the lateral distribution of wheel loads may be determined by elastic analysis.

For rib spans less than 10.0 ft or for decks with shallow floorbeams, the flexibility of the floorbeams shall be considered in calculating force effects in the ribs.

9.8.3.4.3c—Decks with Closed Ribs

For the global analysis of decks with closed ribs, the semi-empirical Pelikan–Esslinger method may be used. The load effects on a closed rib with a span not greater than 20.0 ft may be calculated from wheel loads placed over one rib only, without regard for the effects of adjacent transversely located wheel loads.

For longer rib spans, appropriate corrections of load effects on ribs shall be calculated.

9.8.3.4.4—Level 3 Design

New orthotropic details may be designed using refined three-dimensional analysis as defined in Article 4.6.3.2.3 and as specified below. For fatigue analysis, the structural modeling techniques shall include:

- use of shell or solid elements with acceptable formulation to accommodate steep stress gradients,

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- mesh density of $t \times t$, where t is the thickness of the plate component, and
- local structural stresses shall be determined as specified below.

For fatigue design, the local structural stress shall be used for comparison to the nominal fatigue resistance. Local structural stress at welded connections shall be measured perpendicular to the weld toe and is determined using reference points in the finite element model and extrapolation as shown in Figure 9.8.3.4.4-1. The reference points shall be located at the surface of elements at a distance of $0.5t$ and $1.5t$ measured perpendicular from the weld toe, respectively, with the local structural stress, f_{lss}, determined as:

$$f_{lss} = 1.5 f_{0.5} - 0.50 f_{1.5} \quad (9.8.3.4.4\text{-}1)$$

where:

$f_{0.5}$ = the surface stress at a distance of $0.5t$ from the weld toe (ksi)
$f_{1.5}$ = the surface stress at a distance of $1.5t$ from the weld toe (ksi)

Design Level 3 shall be required for all bridge redecking applications unless the redecking procedure can be demonstrated as meeting the requirements of Article 9.8.3.4.1 and is approved by the Owner.

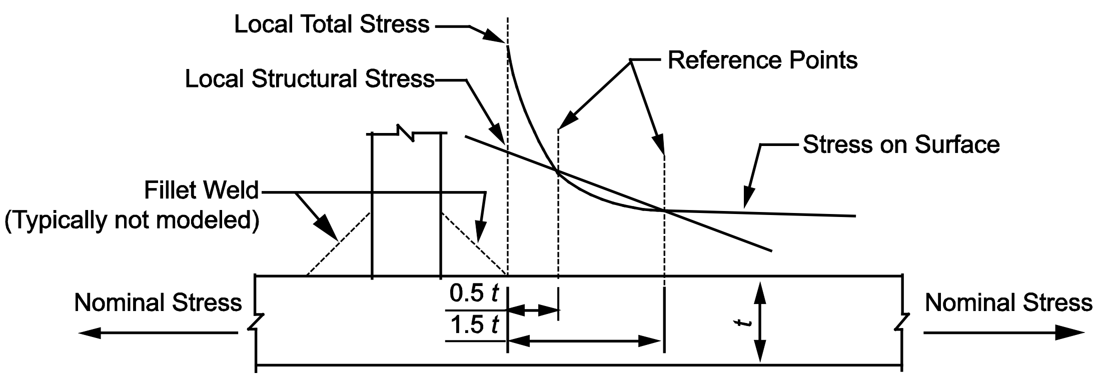

Figure 9.8.3.4.4-1—Local Structural Stress for Level 3 Design of Orthotropic Decks

9.8.3.5—Design

9.8.3.5.1—Superposition of Local and Global Effects

In calculating extreme force effects in the deck, the combination of local and global effects should be determined as specified in Article 6.14.3.

C9.8.3.5.1

The orthotropic deck is part of the global structural system, and, therefore, participates in distributing global stresses. These stresses may be additive to those generated in the deck locally. The axles of the design truck or the design tandem are used for the design of decks, whereas the rest of the bridge is proportioned for combinations of the design truck, the design tandem, and the design lane load. The governing positions of the same load for local and global effects could be quite different. Therefore, the

Designer should analyze the bridge for both load regimes separately, apply the appropriate dynamic load allowance factor, and use the one that governs.

9.8.3.5.2—Limit States

9.8.3.5.2a—General

Orthotropic decks shall be designed to meet the requirements of Section 6 at all applicable limit states unless otherwise specified herein.

C9.8.3.5.2a

Tests indicate a large degree of redundancy and load redistribution between first yield and failure of the deck. The large reduction in combined force effects is a reflection of this performance.

9.8.3.5.2b—Service Limit State

At the service limit state, the deck shall satisfy the requirements as specified in Article 2.5.2.6.

C9.8.3.5.2b

Service Limit State I must be satisfied for overall deflection limits and is intended to prevent premature deterioration of the wearing surface.

Service Limit State II is for the design of bolted connections against slip for overload and should be considered for the design of the rib and floorbeam splices. The remaining limit states are for tensile stresses in concrete structures and can be ignored.

9.8.3.5.2c—Strength Limit State

At the strength limit state for the combination of local and global force effects, the provisions of Article 6.14.3 shall apply.

The effects of compressive instability shall be investigated at the strength limit state. If instability does not control, the resistance of orthotropic plate deck shall be based on the attainment of yield strength at any point in the cross-section.

C9.8.3.5.2c

The deck, because it acts as part of the global structural system, is exposed to in-plane axial tension or compression. Consequently, buckling should be investigated.

Strength design must consider the following demands: rib flexure and shear, floorbeam flexure and shear, and panel buckling. The rib, including the effective portion of the deck plate, must be evaluated for flexural and shear strength for its span between the floorbeams. The floorbeam, including the effective portion of the deck plate, must be evaluated for flexural and shear strength for its span between primary girders or webs. The reduction in floorbeam cross-section due to rib cutouts must be considered. When the panel is part of a primary girder flange, the panel must be evaluated for axial strength based on stability considerations.

Strength Limit IV condition is only expected to control where the orthotropic deck is integral with a long-span bridge superstructure.

9.8.3.5.2d—Fatigue Limit State

Structural components shall be checked for fatigue in accordance with the appropriate design level as specified in Article 9.8.3.4. The provisions of Article 6.6.1.2 shall apply for load-induced fatigue.

With the Owner's approval, application of less stringent fatigue design rules for interior traffic lanes of multilane decks subjected to infrequent traffic may be considered.

C9.8.3.5.2d

Experience has shown that fatigue damage on orthotropic decks occurs mainly at the ribs under the truck wheel paths in the exterior lanes.

For Level 1 design, test loads should be representative of the fatigue truck factored for the Fatigue I load combination and the critical details of the test specimen(s) should simulate both the expected service conditions and the appropriate boundary conditions; verification of these details is sufficient in lieu of a detailed refined fatigue analysis.

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9.8.3.6—Detailing Requirements

9.8.3.6.1—Minimum Plate Thickness

Minimum plate thickness shall be determined as specified in Article 6.7.3.

9.8.3.6.2—Closed Ribs

The one-sided weld between the web of a closed rib and the deck plate shall have a minimum penetration of 60 percent and no blow-through, and shall be placed with a tight fit providing less than or equal to a 0.02 in. gap prior to welding. The weld throat shall be greater than or equal to the rib wall thickness.

C9.8.3.6.2

Historically, the rib-to-deck plate weld has been specified as a one-sided partial penetration weld with minimum 80 percent penetration. Achieving a minimum of 80 percent penetration without blow-through is very difficult and fabricators have often failed to consistently satisfy this requirement. Fatigue tests have demonstrated superior behavior of the weld when the weld throat size is equal to or exceeds the thickness of the rib, with 60 percent or more of the rib-plate thickness fused (Fisher and Barsom, 2015).

The root gap is also a parameter that may influence performance. Research has shown that fatigue resistance of the weld is clearly improved when the root gap is closed in the final condition. When there is full contact, it appears that the root is protected and cracking is prevented. Shop experience indicates that using a tight fit prior to welding will also help prevent weld blow-through. Kolstein (2007) suggests the limit of 0.02 in. and this is adopted in these Specifications.

Additionally, melt-through of the weld is a quality issue that must be controlled. Fatigue tests on a limited number of samples (Sim and Uang, 2007) indicate that the performance of locations of melt-through is greater than or equal to those created by the notch condition of the 80 percent penetration. However, there are legitimate concerns that excessive melt-through may provide potential fatigue initiation sites and as such it should be avoided if possible. As such, the proposed detailing criteria is that the rib-to-deck weld be one-sided with a 60 percent minimum penetration and no blow-through, with the weld throat greater than or equal to the rib thickness, and with a tight fit providing less than or equal to a 0.02 in. gap prior to welding. Additional details of the weld joint should be left for the fabricator to develop.

9.8.3.6.3—Welding to Orthotropic Decks

Welding of attachments, utility supports, lifting lugs, or shear connectors to the deck plate or ribs shall require approval by the Engineer of Record for the deck system.

9.8.3.6.4—Deck and Rib Details

Deck and rib splices shall either be welded or mechanically fastened by high-strength bolts. Ribs shall be run continuously through cutouts in the webs of floorbeams, as shown in Figure 9.8.3.6.4-1. The following

C9.8.3.6.4

Closed ribs may be trapezoidal, U-shaped, or V-shaped; the latter two are most efficient.

The floorbeam web cutouts at the intersections with the ribs may be with or without an additional free cutout at fabrication details shall be required by the contract documents as identified in Figure 9.8.3.6.4-1:

a) No snipes (cutouts) in floorbeam web
b) Welds to be wrapped around
c) Grind smooth
d) Combined fillet-groove welds may have to be used 1) in cases where the required size of fillet welds to satisfy the fatigue resistance requirements would be excessive if used alone or 2) to accomplish a ground termination.

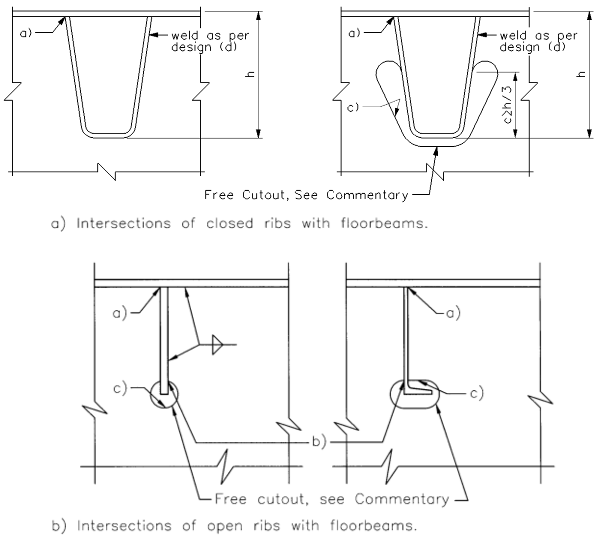

Figure 9.8.3.6.4-1—Detailing Requirements for Orthotropic Decks

the bottom of the ribs. The former detail is generally preferable since it minimizes the rib restraint against rotation in its plane and associated stresses in the welds and in the floorbeam web.

If the bottom cutout depth, *c*, is small enough, the rotation of the rib is restrained and considerable out-of-plane stresses are introduced in the floorbeam web when the floorbeam is shallow. Local secondary stresses are also introduced in the rib walls by the interaction forces between the floorbeam webs and the rib walls and by secondary effects due to the small depth of cutout, *c* (Wolchuk and Ostapenko, 1992).

If the floorbeam web is deep and flexible, or where additional depth of the cutout would unduly reduce the shear strength of the floorbeam, welding all around the rib periphery may be appropriate (*ECSC Report on Fatigue*, 1995, Wolchuk, 1999).

Fatigue tests suggested that open snipes in the floorbeam webs at the junctions of the rib walls with the deck plate may cause cracks in the rib walls. Therefore, a tight-fitting snipe and a continuous weld between the floorbeam web and the deck and rib wall plates appear to be preferable.

Open ribs may be flat bars, angles, tees, or bulb bars. Open-rib decks are less efficient and require more welding but are generally considered less risky to fabricate.

9.8.4—Orthotropic Aluminum Decks

9.8.4.1—General

Orthotropic aluminum decks shall consist of a deck plate stiffened and supported by rib extrusions. The ribs may be parallel or perpendicular to the direction of traffic.

The provisions of Article 9.8.3.2 through Article 9.8.3.3 shall apply, except that the wearing surface shall not be regarded as an integral part of the orthotropic deck for analysis and design of the deck or rib.

When an aluminum orthotropic deck is supported by components of another material, the differences in thermal expansion of the two materials and the potential for accelerated corrosion due to dissimilar metals shall be considered.

The structural interaction of an aluminum orthotropic deck with the existing structure shall be investigated.

C9.8.4.1

Only one application of ribs placed perpendicular to traffic was known as of 1997. Therefore, little or no experience of in-service fatigue behavior exists, and complete investigation of load-induced and distortion-induced fatigue should be required for this application.

9.8.4.2—Approximate Analysis

In lieu of more precise information, the effective width of deck plate acting with a rib shall not exceed the rib spacing or one-third of the span.

C9.8.4.2

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The flexibility of the supports shall be considered in determining the longitudinal moments in continuous decks.

In determining the transverse moments, the effects of the torsional rigidity of the ribs shall be included when the ribs are torsionally stiff and may be disregarded if the ribs are torsionally flexible.

For the analysis of decks with closed ribs, the provisions of Article 9.8.3.4.3c may be applied.

The transverse moments should be calculated in two stages: those due to the direct loading of the deck plate, assuming nondeflecting ribs, and those due to the transverse shear transfer resulting from the rib displacements. Stresses from these moments are then combined.

9.8.4.3—Limit States

Orthotropic decks shall be designed to meet the requirements of Section 7 at all applicable limit states.

At the service limit state, the deck shall satisfy the requirement of Article 2.5.2.6.

The longitudinal ribs, including an effective width of deck plate, shall be investigated for stability as individual beam-columns assumed as simply supported at transverse beams.

At the fatigue limit state, the deck shall satisfy the provisions of Article 7.6.

Regardless of whether the stress range is tensile, compressive, or reversal, maximum stress range shall be investigated for:

- Transverse direction at the rib-to-plate connection;
- Longitudinal direction;
- All bolted, welded end, and edge details; and
- Transverse direction at the rib-to-plate connection when the adjacent rib is loaded.

C9.8.4.3

This condition has been shown to control the design under certain geometrical conditions.

The maximum stress range is used for design because significant tensile residual stresses exist adjacent to most weldments, and gross compressive stresses may result in a net tensile stress range.

See Menzemer et al. (1987) for additional discussion.

9.8.5—Corrugated Metal Decks

9.8.5.1—General

Corrugated metal decks should be used only on secondary and rural roads.

Corrugated metal decks shall consist of corrugated metal pans filled with bituminous asphalt or another approved surfacing material. The metal pans shall be positively fastened to the supporting components.

C9.8.5.1

The intent of fastening the corrugated metal pans to the supporting components is to ensure the stability of both under transient loads.

9.8.5.2—Distribution of Wheel Loads

A 45-degree distribution of the tire load from the contact area to the neutral axis of the corrugated metal pans may be assumed.

C9.8.5.2

The 45-degree distribution is a traditional approach for most nonmetallic structural materials.

9.8.5.3—Composite Action

For contribution of the fill to composite action with the deck plate, the provisions of Article 9.8.3.3 shall apply.

Composite action of the corrugated metal deck pan with the supporting components may be considered only if the interface connections are designed for full composite action, and the deck is shown to resist the compressive forces associated with the composite action.

C9.8.5.3

Due to the sensitivity of the plate to temperature, corrosion, and structural instability, composite action should be utilized only if physical evidence is sufficient to prove that its functionality can be counted on for the specified design life.

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9.9—WOOD DECKS AND DECK SYSTEMS

9.9.1—Scope

This Article shall apply to the design of wood decks supported by beams, stringers, or floorbeams or used as a deck system.

C9.9.1

This Article applies to wood decks and deck systems that are currently being designed and built in the United States and that have demonstrated acceptable performance. The supporting components may be metal, concrete, or wood.

9.9.2—General

The provisions of Section 8 shall apply.

Materials used in wood decks and their preservative treatment shall meet the requirements of Sections 2, 5, 6, and 8.

The nominal thickness of plank decks shall not be less than 4.0 in. for roadways and 2.0 in. for sidewalks. The nominal thickness of wood decks other than plank decks shall not be less than 6.0 in.

C9.9.2

In laminated decks, large deviations in the thickness or extensive warping of the laminations may be detrimental regarding both strength and long-term performance. Although rough or full sawn material can be more economical than planed, the variations in dimensions can be quite large. If appropriate dimensional tolerances are not likely to be obtained, dressing of the components should be recommended.

9.9.3—Design Requirements

9.9.3.1—Load Distribution

Force effects may be determined by using one of the following methods:

- The approximate method specified in Article 4.6.2.1,
- Orthotropic plate theory, or
- Equivalent grid model.

If the spacing of the supporting components is less than either 36.0 in. or 6.0 times the nominal depth of the deck, the deck system, including the supporting components, shall be modeled as an orthotropic plate or an equivalent grid.

In stress-laminated decks satisfying the butt stagger requirements specified in Article 9.9.5.3, rigidity may be determined without deduction for the butt joints.

C9.9.3.1

In wood decks with closely spaced supporting components, the assumption of infinitely rigid supports upon which approximate methods of analysis are based is not valid. Two-dimensional methods of analysis are, therefore, recommended to obtain force effects with reasonable accuracy.

9.9.3.2—Shear Design

Shear effects may be neglected in the design of stress-laminated decks. In longitudinal decks, maximum shear shall be computed in accordance with the provisions of Article 8.7.

In transverse decks, maximum shear shall be computed at a distance from the support equal to the depth of the deck.

C9.9.3.2

Shear problems in laminated wood decks are rare, as the inherent load sharing benefits of the multiple-member system are believed to be quite significant. The probability of simultaneous occurrence of potentially weak shear zones in adjacent laminates is low. Therefore, a multiple-member shear failure, which would be necessary to propagate shear splits in any one lamination, would be difficult to achieve.

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For both longitudinal and transverse decks, the tire footprint shall be located adjacent to, and on the span side of, the point of the span where maximum force effect is sought.

With little test data available, no changes to the shear design for spike-laminated decks are being introduced.

9.9.3.3—Deformation

At the service limit state, wood decks shall satisfy the requirements specified in Article 2.5.2.6.

9.9.3.4—Thermal Expansion

The coefficient of thermal expansion of wood parallel to its fibers shall be taken as 0.000002 per °F.

Thermal effects may be neglected in plank decks and spike-laminated decks.

For stress-laminated and glued laminated panel decks made continuous over more than 400 ft, relative movements due to thermal expansion with respect to substructures and abutments shall be investigated.

C9.9.3.4

Generally, thermal expansion has not presented problems in wood deck systems. Except for the stress-laminated deck and tightly placed glued laminated panels, most wood decks inherently contain gaps at the butt joints that can absorb thermal movements.

9.9.3.5—Wearing Surfaces

Wood decks shall be provided with a wearing surface conforming to the provisions of Article 9.9.8.

C9.9.3.5

Experience has shown that unprotected wood deck surfaces are vulnerable to wear and abrasion, and they may become slippery when wet.

9.9.3.6—Skewed Decks

Where the skew of the deck is less than 25 degrees, transverse laminations may be placed on the skew angle. Otherwise, the transverse laminations shall be placed normal to the supporting components, and the free ends of the laminations at the ends of the deck shall be supported by a diagonal beam or other suitable means.

C9.9.3.6

With transverse decks, placement of the laminations on the skew is the easiest and most practical method for small skew angles, and cutting the ends of the laminations on the skew provides a continuous straight edge.

In longitudinal decks, except for stress-laminated wood, any skew angle can generally be accommodated by offsetting each adjacent lamination on the skew.

9.9.4—Glued Laminated Decks

9.9.4.1—General

Glued laminated timber panel decks shall consist of a series of panels, prefabricated with water-resistant adhesives, that are tightly abutted along their edges.

Transverse deck panels shall be continuous across the bridge width.

If the span in the primary direction exceeds 8.0 ft, the panels shall be interconnected with stiffener beams as specified in Article 9.9.4.3.

C9.9.4.1

In glued laminated decks built to date, transverse deck panels have been 3.0 to 6.0 ft wide, and longitudinal deck panels have been 3.5 to 4.5 ft wide. The design provisions are considered applicable only to the range of panel sizes given herein.

These design provisions are based upon development work carried out at the USDA Forest Products Laboratory in the late 1970s.

This form of deck is appropriate only for roads having low to medium volumes of commercial vehicles.

9.9.4.2—Deck Tie-Downs

Where panels are attached to wood supports, the tie-downs shall consist of metal brackets that are bolted through the deck and attached to the sides of the supporting component. Lag screws or deformed shank spikes may be used to tie panels down to wood support.

Where panels are attached to steel beams, they shall be tied down with metal clips that extend over the beam flange and that are bolted through the deck.

C9.9.4.2

The methods of tie-down specified herein are based upon current practices that have proven to be adequate. Use of other methods require approval by Owner.

9.9.4.3—Interconnected Decks

9.9.4.3.1—Panels Parallel to Traffic

Interconnection of panels shall be made with transverse stiffener beams attached to the underside of the deck. The distance between stiffener beams shall not exceed 8.0 ft, and the rigidity, EI, of each stiffener beam shall not be less than 80,000 kip-in.^{2}. The beams shall be attached to each deck panel near the panel edges and at intervals not exceeding 15.0 in.

C9.9.4.3.1

Although the transverse stiffener beam ensures interpanel shear transfer of loads, some relative deflection will take place. Under frequent heavy loads, this relative deflection will cause reflective cracking of bituminous wearing surfaces.

9.9.4.3.2—Panels Perpendicular to Traffic

Interconnection of panels may be made with mechanical fasteners, splines, dowels, or stiffener beams. Where used, the stiffener beams should be continuous over the full length of the span and should be secured through the deck within 6.0 in. of the edges of each panel and as required between edges.

When panels are interconnected with stiffener beams, the beams shall be placed longitudinally along the centerspan of each deck span. Provisions of Article 9.9.4.3.1 shall apply for the design of the stiffener beams.

The live load bending moment per unit width shall be determined in accordance with the provisions of Article 4.6.2.1.3.

C9.9.4.3.2

The doweling of the deck system is intended to prevent relative displacement of the glued laminated deck panels. A design procedure for dowels can be found in Ritter (1990). With proper prefabrication and construction, this doweled system has proven to be effective in preventing relative displacement between panels. However, in practice, problems with hole alignment and the necessity for field modifications may reduce their efficiency.

Using one longitudinal stiffener beam in each space between girders has proven to be both a practical and effective method of reducing relative displacements between transverse panels.

9.9.4.4—Noninterconnected Decks

Decks not interconnected at their edges shall only be employed on secondary rural roads. No transfer of force effects at the panel edges shall be assumed in the analysis.

C9.9.4.4

The noninterconnected panel deck will likely cause reflective cracking in the wearing surface at the butt joints, even under relatively low levels of loading. It is appropriate only for roads having low volumes of commercial vehicles in order to avoid the extensive maintenance that the wearing surface may require.

9.9.5—Stress-Laminated Decks

9.9.5.1—General

Stress-laminated decks shall consist of a series of wood laminations that are placed edgewise and post-

C9.9.5.1

The majority of decks of this type include laminations which are 2.0 to 3.0 in. in thickness.

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tensioned together, normal to the direction of the lamination.

Stress-laminated decks shall not be used where the skew exceeds 45 degrees.

The contract documents shall require that the material be subjected to expansion baths to remove excess oils.

The increased load distribution and load sharing qualities of this deck, coupled with its improved durability under the effects of repeated heavy vehicles, make it the best choice among the several wood decks for high-volume road application (Csagoly and Taylor, 1979; Sexsmith et al., 1979).

The structural performance of these decks relies on friction, due to transverse prestress, between the surfaces of the laminations to transfer force effects. Unlike spiked or bolted connections in wood, the friction-based performance of stress-laminated decks does not deteriorate with time under the action of repeated heavy loads.

Experience seems to indicate that the use of waterborne preservatives can negatively affect the performance of stress-laminated decks. Wood treated with waterborne preservatives responds rapidly to the short-term changes in moisture conditions to which bridges are subjected frequently in most areas of North America. The attendant dimensional changes in the wood can result in substantial changes in the prestressing forces. Wood treated with oil-borne preservatives does not respond so readily to short-term changes in moisture conditions.

The preservative treatment for wood to be used in stress-laminated decks should be kept to the minimum specified in the standards given in Article 8.4.3. Excessive oils in the wood may be expelled after the deck is stressed and can contribute to higher prestress losses over a short period after construction.

9.9.5.2—Nailing

Each lamination shall be specified to be fastened to the preceding one by common or spiral nails at intervals not exceeding 4.0 ft. The nails shall be driven alternately near the top and bottom edges of the laminations. One nail shall be located near both the top and bottom at butt joints. The nails should be of sufficient length to pass through two laminations.

C9.9.5.2

Nailing is only a temporary construction convenience in stress-laminated decks, and it should be kept as close to minimum requirements as possible. Excessive nailing may inhibit the build-up of elastic strains during transverse stressing, which could subsequently contribute to decreasing its effectiveness.

9.9.5.3—Staggered Butt Joints

Where butt joints are used, not more than one butt joint shall occur in any four adjacent laminations within a 4.0 ft distance, as shown in Figure 9.9.5.3-1.

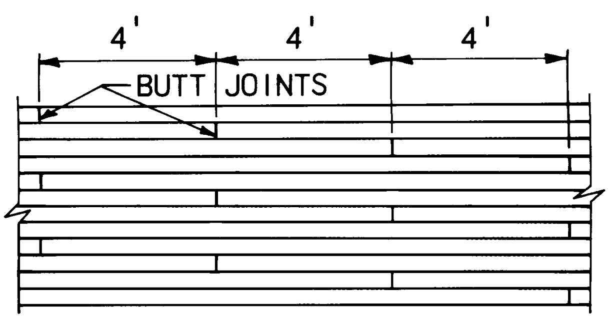

Figure 9.9.5.3-1—Minimum Spacing of Lines of Butt Joints

C9.9.5.3

Butt joint requirements are extreme values and are intended to allow for lamination lengths that are less than the deck length. Uniformly reducing or eliminating the occurrence of butt joints, distributing butt joints, or both will improve performance.

The implication of this provision is that laminations shorter than 16.0 ft cannot be used. If laminations longer than 16.0 ft are used, the spacing of butt joints is one-quarter of the length.

9.9.5.4—Holes in Laminations

The diameter of holes in laminations for the prestressing unit shall not be greater than 20 percent of the lamination depth. Spacing of the holes along the laminations shall be neither less than 15.0 times the hole diameter nor less than 2.5 times the depth of the laminate.

Only drilled holes shall be permitted.

C9.9.5.4

These empirical limitations are intended to minimize the negative effects of hole size and spacing on the performance of the deck.

Punched holes can seriously affect the performance of the laminates by breaking the wood fibers in the vicinity of the holes.

9.9.5.5—Deck Tie-Downs

Decks shall be tied down at every support, and the spacing of the tie-downs along each support shall not exceed 3.0 ft. Each tie-down shall consist of a minimum of two 0.75-in. diameter bolts for decks up to and including 12.0 in. deep and two 1.0-in. diameter bolts for decks more than 12.0 in. deep.

C9.9.5.5

The stress-laminated deck requires a more effective tie-down than toe-nailing or drift pins. It has a tendency to develop curvature perpendicular to the laminates when transversely stressed. Tie-downs using bolts or lag screws ensure proper contact of the deck with the supporting members.

9.9.5.6—Stressing

9.9.5.6.1—Prestressing System

New stressed wood decks shall be designed using internal prestressing. External prestressing may be used to rehabilitate existing nail-laminated decks and shall utilize continuous steel bulkheads.

In stress-laminated decks with skew angles less than 25 degrees, stressing bars may be parallel to the skew. For skew angles between 25 degrees and 45 degrees, the bars should be placed perpendicular to the laminations, and in the end zones, the transverse prestressing bars should be fanned in plan as shown in Figure 9.9.5.6.1-1 or arranged in a step pattern as shown in Figure 9.9.5.6.1-2.

Dimensional changes in the deck due to prestressing shall be considered in the design.

Anchorage hardware for the prestressing rods should be arranged in one of the three ways shown in Figure 9.9.5.6.1-3.

C9.9.5.6.1

External and internal prestressing systems are shown in Figure 9.9.5.6.1-3. The internal system provides better protection to the prestressing element and lessens restriction to the application of wearing surfaces.

Generally, it is not necessary to secure timber decks to the supports until all the transverse stressing has been completed. There is the potential for extensive deformation when a deck is stressed over a very long length due to unintentional eccentricity of prestressing. It is recommended that restraints during stressing be provided when the width of the deck, perpendicular to the laminations, exceeds 50.0 times the depth of the deck for longitudinal decks and 40.0 times the depth of the deck for transverse decks. These restraints should not inhibit the lateral movement of the deck over its width during the stressing procedure.

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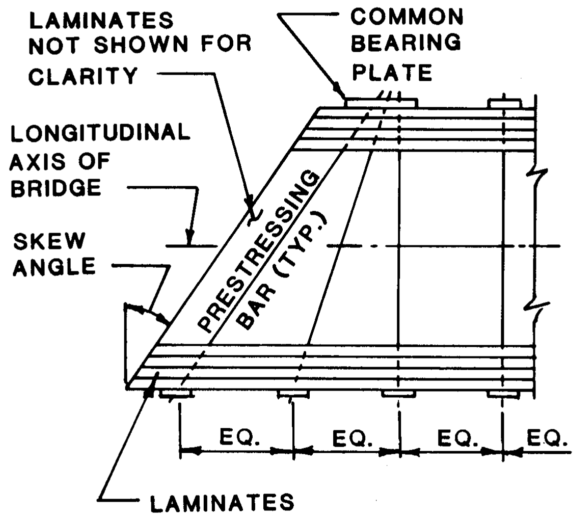

Figure 9.9.5.6.1-1—Fanned Layout of Prestressing Bars in End Zones of Skewed Decks—Illustrative Only

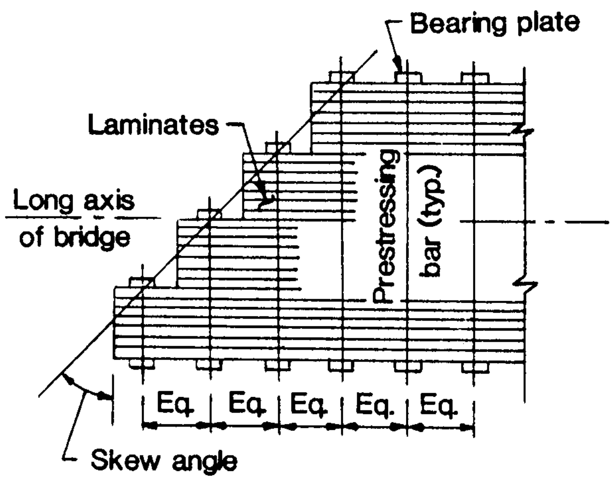

Figure 9.9.5.6.1-2—Staggered Layout of Prestressing Bars in End Zones of Skewed Decks—Illustrative Only

Potential concentration of bearing stresses and sliding of the common bearing plate should be considered in conjunction with the fanned arrangement of prestressing elements shown in Figure 9.9.5.6.1-1.

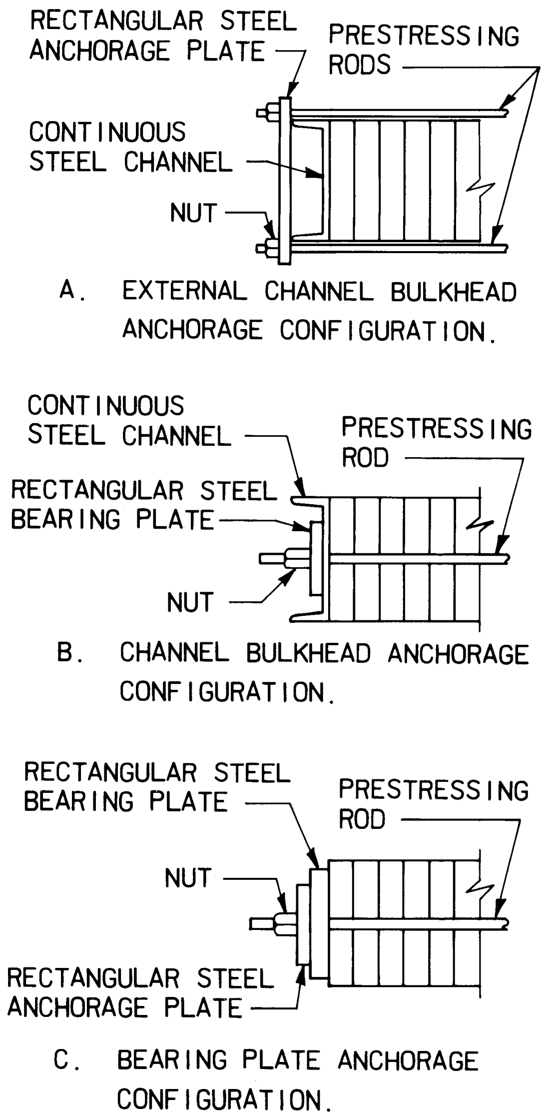

Figure 9.9.5.6.1-3—Types of Prestressing Configurations

The isolated steel bearing plates should be used only on hardwood decks, or, where a minimum of two hardwood laminations are provided, on the outside edges of the deck.

Continuous steel bulkheads or hardwood laminations are required because they improve field performance. Isolated steel bearing plates on softwood decks have caused crushing of the wood, substantially increased stress losses and resulted in poor aesthetics.

9.9.5.6.2—Prestressing Materials

Prestressing materials shall comply with the provisions of Article 5.4.

C9.9.5.6.2

All prestressed wood decks built to date have utilized high-strength bars as the stressing elements. Theoretically, any prestressing system that can be adequately protected against corrosion is acceptable.

9.9.5.6.3—Design Requirements

The steel–wood ratio, R_{sw}, shall satisfy:

$$R_{sw} = \frac{A_s}{sh} \leq 0.0016 \qquad (9.9.5.6.3\text{-}1)$$

where:

s = spacing of the prestressing elements (in.)
h = depth of deck (in.)
A_s = area of steel bar or strand (in.^{2})

The prestressing force per prestressing element (kip) shall be determined as:

$$P_{pt} = 0.1hs \qquad (9.9.5.6.3\text{-}2)$$

The effective bearing area, A_B, on the wood directly under the anchorage bulkhead due to prestress shall be determined by considering the relative stiffness of the wood deck and the steel bulkhead. The bulkhead shall satisfy:

$$P_{BU} = \phi F A_B \geq P_{pt} \qquad (9.9.5.6.3\text{-}3)$$

where:

P_{BU} = factored compressive resistance of the wood under the bulkhead (kip)
ϕ = resistance factor for compression perpendicular to grain as specified in Article 8.5.2.2
F = as specified in Table 9.9.5.6.3-1

Table 9.9.5.6.3-1—*F* Values for Prestressed Wood Decks

Species	*F* (ksi)
Douglas Fir–Larch	0.425
Hemlock Fir	0.275
Spruce–Pine–Fir	0.275
Eastern Softwoods	0.225
Mixed Southern Pine	0.375
Southern Pine	0.375
Spruce–Pine–Fir (South)	0.225
Northern Red Oak	0.600
Red Maple	0.400
Red Oak	0.550
Yellow Poplar	0.275

C9.9.5.6.3

The limitation on the steel–wood area ratio is intended to decrease prestress losses due to relaxation caused by wood and steel creep as well as deck dimensional changes due to variations in wood moisture content. Prestress losses are very sensitive to this ratio, and most existing structures have values less than 0.0016. A small area ratio of 0.0012 to 0.0014, coupled with an initial moisture content of less than 19 percent and proper preservative treatment, will ensure the highest long-term prestress levels in the deck.

The average compressive design stress represents the uniform pressure that is achieved away from the anchorage bulkhead. Limitation on compressive stress at maximum prestress minimizes permanent deformation in the wood. Increasing the initial compressive stress beyond these levels does not significantly increase the final compressive stress after all losses have occurred.

Eq. 9.9.5.6.3-2 is based on a uniform compressive stress of 0.1 ksi between the laminations due to prestressing. For structural analysis, a net compressive stress of 0.04 ksi, after losses, may be assumed.

Relaxation of the prestressing system is time-dependent, and the extensive research work, along with the experience obtained on the numerous field structures, have shown that it is necessary to restress the system after the initial stressing to offset long-term relaxation effects. The optimum stressing sequence is as follows:

- Stress to full design level at time of construction,
- Restress to full design level not less than one week after the initial stressing, and
- Restress to full design level not less than four weeks after the second stressing.

After the first restressing, increasing the time period to the second restressing improves long-term stress retention. Subsequent restressings will further decrease the effects of long-term creep losses and improve stress retention.

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9.9.5.6.4—Corrosion Protection

Elements of the prestressing system shall be protected by encapsulation and/or surface coatings. The protective tubing shall be capable of adjusting at least ten percent of its length during stressing without damage.

C9.9.5.6.4

Elements of a suitable protection system are shown in Figure C9.9.5.6.4-1.

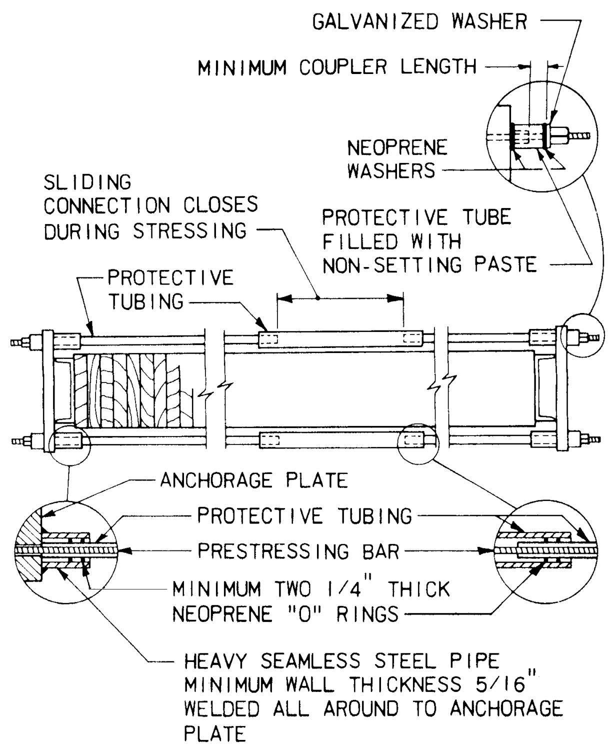

Figure C9.9.5.6.4-1—Elements of Corrosion Protection

9.9.5.6.5—Railings

Railings shall not be attached directly either to any prestressing element or to bulkhead systems. The deck shall not be penetrated within 6.0 in. of a prestressing element.

C9.9.5.6.5

Curb and railing attachment directly to any component of the stressing system increases the risk of failure in the event of vehicle impact.

9.9.6—Spike-Laminated Decks

9.9.6.1—General

Spike-laminated decks shall consist of a series of lumber laminations that are placed edgewise between supports and spiked together on their wide face with deformed spikes of sufficient length to fully penetrate four laminations. The spikes shall be placed in lead holes that are bored through pairs of laminations at each end and at intervals not greater than 12.0 in. in an alternating pattern near the top and bottom of the laminations, as shown in Figure 9.9.6.1-1.

Laminations shall not be butt spliced within their unsupported length.

C9.9.6.1

The use of spike-laminated decks should be limited to secondary roads with low truck volumes, i.e., *ADTT* significantly less than 100 trucks per day.

The majority of decks of this type have used laminations of 3.0 to 4.0 in. in thickness. The laminates are either assembled on site or are prefabricated into panels in preparation for such assembly.

The specified design details for lamination arrangement and spiking are based upon current practice. It is important that the spike lead holes provide a tight fit to ensure proper load transfer between laminations and to minimize mechanical movements.

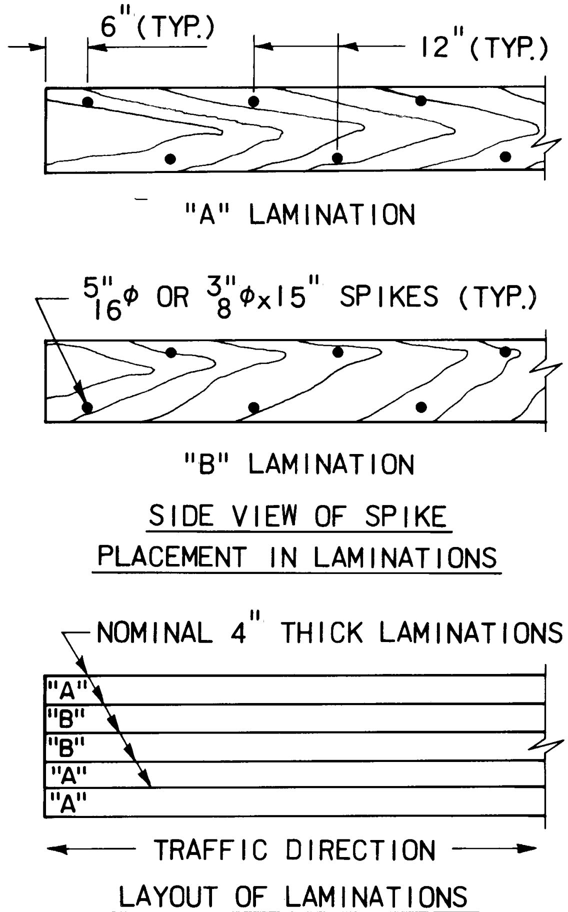

Figure 9.9.6.1-1—Spike Layout for Spike-Laminated Decks

9.9.6.2—Deck Tie-Downs

Deck tie-downs shall be as specified in Article 9.9.4.2.

9.9.6.3—Panel Decks

The distribution widths for interconnected spike-laminated panels may be assumed to be the same as those for continuous decks, as specified in Section 4.

The panels may be interconnected with mechanical fasteners, splines, dowels, or stiffener beams to transfer shear between the panels. If stiffener beams are used, the provisions of Article 9.9.4.3 shall apply.

C9.9.6.3

The use of noninterconnected decks should be limited to secondary and rural roads.

It is important to provide an effective interconnection between panels to ensure proper load transfer. Stiffener beams, comparable to those specified for glued laminated timber panels, are recommended. Use of an adequate stiffener beam enables the spike-laminated deck to approach the serviceability of glue-laminated panel construction.

With time, the deck may begin to delaminate in the vicinity of the edge-to-edge panel joints. The load distribution provisions given for the noninterconnected panels are intended for use in the evaluation of existing noninterconnected panel decks and interconnected panel decks in which the interconnection is no longer effective.

9.9.7—Plank Decks

9.9.7.1—General

Wood plank decks shall consist of a series of lumber planks placed flatwise on supports. Butt joints shall be placed over supports and shall be staggered a minimum of 3.0 ft for adjacent planks.

C9.9.7.1

This type of deck has been used on low-volume roads with little or no heavy vehicles, and it is usually economical. However, these decks provide no protection against moisture to the supporting members; they will not readily accept and/or retain a bituminous wearing surface, and usually require continuous maintenance if used by heavy vehicles.

These decks should be limited to roads that carry little or no heavy vehicles or where the running surface is constantly monitored and maintained.

9.9.7.2—Deck Tie-Downs

On wood beams, each plank shall be nailed to each support with two nails of minimum length equal to twice the plank thickness.

On steel beams, planks shall be bolted to the beams or nailed to wood nailing strips. The strips should be at least 4.0 in. thick, and their width should exceed that of the beam flange. The strips should be secured with A 307 bolts at least 0.625 in. in diameter and placed through the flanges, spaced not more than 4.0 ft apart and no more than 1.5 ft from the ends of the strips.

9.9.8—Wearing Surfaces for Wood Decks

9.9.8.1—General

Wearing surfaces shall be of continuous nature and no nails, except in wood planks, shall be used to fasten them to the deck.

C9.9.8.1

Bituminous wearing surfaces are recommended for wood decks.

The surface of the wood deck should be free of surface oils to encourage adhesion and prevent bleeding of the preservative treatment through the wearing surface. Excessive bleeding of the treatment can seriously reduce the adhesion. The plans and specifications should clearly state that the deck material be treated using the empty cell process, followed by an expansion bath or steaming.

9.9.8.2—Plant Mix Asphalt

An approved tack coat shall be applied to wood decks prior to the application of an asphalt wearing surface. The tack coat may be omitted when a geotextile fabric is used, subject to the recommendations of the manufacturer.

When possible, a positive connection between the wood deck and the wearing surface shall be provided. This connection may be provided mechanically or with a geotextile fabric.

The asphalt should have a minimum compacted depth of 2.0 in. Where cross slope is not provided by the wood deck, a minimum of one percent shall be provided by the wearing surface.

C9.9.8.2

The application of a tack coat greatly improves the adhesion of asphalt wearing surfaces.

Due to the smooth surface of individual laminations and glued laminated decks, it is beneficial to provide a positive connection in order to ensure proper performance. The use of asphalt-impregnated geotextile fabric, when installed properly, has proven to be effective.

Asphalt wearing surfaces on stress-laminated wood decks have proven to perform well with only a tack coat and no reinforcement between the deck and the asphalt.

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9.9.8.3—Chip Seal

When a chip seal wearing surface is used on wood decks, a minimum of two layers should be provided.

C9.9.8.3

Laminated decks may have offset laminations creating irregularities on the surface, and it is necessary to provide an adequate depth of wearing surface to provide proper protection to the wood deck. Chip seal wearing surfaces have a good record as applied to stress-laminated decks due to their behavior approaching that of solid slabs.

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