Special Report

Recommended Practice for Precast Prestressed Concrete Composite Bridge Deck Panels

Prepared for the

PCI Bridge Producers Committee

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Based on the latest research and field experience, this PCI Committee report presents recommendations for the design, production, handling, shipping and erection of precast prestressed concrete composite bridge deck panels.

The report also covers product tolerances, cracking and repair methods, as well as plant and jobsite inspection procedures.

A design example, using the AASHTO Specifications, is included together with several design aids.

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NOTATION

A = area (in.^{2})

A_s^* = area of prestressing steel (in.^{2})

b = unit width of deck panel (in.)

b_t = effective width of top slab (in.)

c_{bottom} = distance from centroidal axis to bottom of deck panel (in.)

CR_c = loss of prestress due to creep of concrete (psi)

CR_s = loss of prestress due to relaxation of prestressing steel (psi)

$c_{top\ precast}$ = distance from centroidal axis to top of deck panel (in.)

d = distance from extreme compression fiber to centroid of prestressing force (in.)

d^2 = square of distance from composite section centroidal axis to center of gravity of element being considered (in.^{2})

D = nominal diameter of prestressing steel (in.)

E_{ci} = modulus of elasticity of concrete at time of stress transfer (psi)

$E_{precast}$ = modulus of elasticity of deck panel concrete (ksi)

E_s = modulus of elasticity of prestressing steel (psi)

$E_{top\ slab}$ = modulus of elasticity of cast-in-place top slab concrete (ksi)

ES = loss of prestress due to elastic shortening (psi)

f_{bottom} = bending stress at bottom of deck panel (psi)

f'_c = compressive strength of concrete at 28 days (psi)

f'_{ci} = compressive strength of concrete at time of initial prestress (psi)

f_{cds} = concrete stress at center of gravity of prestressing steel due to all dead loads except dead load present at time prestressing force is applied (psi)

f_{cir} = concrete stress at center of gravity of prestressing steel due to prestressing force and dead load of member immediately after stress transfer (psi)

f_r = modulus of rupture of concrete (psi)

Δf_s = total prestress loss (psi)

f'_s = ultimate strength of prestressing steel (ksi)

f_{se} = effective stress in prestressing strand after losses (ksi)

f_{su}^* = average stress in prestressing steel at ultimate load (ksi)

f_{top} = bending stress at top of cast-in-place top slab

$f_{top\ precast}$ = bending stress at top of deck panel (psi)

h_p = thickness of deck panel (in.)

I = gross section moment of inertia (in.^{4})

l_d = strand development length

l_x = distance from end of prestressing strand to center of deck panel (in.)

L = span length (ft)

M = bending moment (lb-ft)

$M_{continuous}$ = continuous span bending moment (lb-ft)

M_{cr} = moment causing flexural cracking at section due to externally applied loads

$M_{cr\,tot}$ = total bending moment at which section will crack (lb-ft)

$M_{design,\,service}$ = service load moment to be used for design (lb-ft)

M_{DL} = bending moment resulting from dead loads (lb-ft)

M_n = nominal moment strength of section

M_{ps} = bending moment necessary to overcome prestress force (lb-ft)

M_{simple} = simple span bending moment (lb-ft)

M_u = factored moment at section $\leq \phi M_n$ (lb-ft)

n = modular ratio of elasticity

p^* = ratio of prestressing steel, A_s^*/bd

$P_{all\,losses}$ = assumed effective prestress force per strand after all losses (kips)

S = effective span length as defined under "Span Lengths" Article 3.24.1 of AASHTO Specifications (ft)

SH = loss of prestress due to concrete shrinkage (psi)

S_{bottom} = section modulus of composite section with respect to bottom of deck panel (in.^{3})

$S_{bottom\,precast\,bare}$ = section modulus of untopped deck panel with respect to its bottom (in.^{3})

S_{top} = section modulus of composite section with respect to top of composite slab (in.^{3})

$S_{top\,precast}$ = section modulus of composite section with respect to top of deck panel (in.^{3})

$S_{top\,precast\,bare}$ = section modulus of untopped deck panel with respect to its top (in.^{3})

W = uniform load (psf)

Y = distance from bottom of section to center of gravity of element being considered (in.)

Y_{top} = distance from centroidal axis of composite section to extreme fiber in compression (in.)

* * *

INTRODUCTION

Precast and prestressed concrete composite bridge deck panels are used with cast-in-place concrete to provide a convenient and cost effective method of construction for concrete bridge decks. The panels are usually precast at a manufacturing plant. They are trucked to the bridge construction site and lifted by cranes onto concrete or steel girders. There, they span the opening between girders and serve as permanent forms for the cast-in-place concrete topping that completes the bridge deck. The precast concrete panels and concrete topping become composite and the panels contain all of the required positive moment reinforcement between girders.

Precast and prestressed concrete composite bridge deck panels will be referred to, in this report, as "composite deck panels" or simply as "deck panels."

Composite deck panels are similar to other prestressed concrete composite members with regard to applications and design considerations. There are, however, situations that are unique to composite deck panels due to the way the panels are produced and used.

A 1986 survey conducted by the Prestressed Concrete Institute (PCI) contacted all State Highway Departments in the United States, numerous Tollway and Transportation Authorities and PCI member plants to determine current specifications and methods of production. Some agencies have unique approaches to the design and use of composite deck panels. Similarly, a few manufacturers have developed their own methods for fabrication of these products. A review of the survey responses, however, reveals that there is uniformity between most agency specifications and prevalent methods of production.

The purpose of this report is to present recommendations for the design, manufacture and use of composite deck panels. These recommendations are supported by the referenced research, the analysis of the survey data, and the collective experience and judgment of the precast and prestressed concrete industry. The practices suggested in this report, where identified in use, have been shown to be performing favorably in locations across North America, for as long as 35 years. The recommendations reported herein may be applied confidently by designers, precasters and contractors with the expectation of excellent performance for lowest possible cost.

This report was prepared by Ross Bryan Associates, Inc. of Nashville, Tennessee, with input and direction from the PCI Bridge Producers Committee, the PCI Committee on Bridges and the Technical Activities Committee. All reasonable care has been used to verify the accuracy of material contained in this report. However, the report should be used only by those experienced in structural design and should not replace sound structural engineering judgment.

This report is intended to address design, fabrication, shipping, handling, and erection requirements for precast prestressed concrete composite bridge deck panels. All designs herein conform to the 1983 AASHTO *Standard Specifications for Highway Bridges*, including the 1984 through 1987 Interim Specifications.[1]

For the fabricator and the contractor, this report is intended as a guide for the production of high quality composite deck panels and their proper installation.

* * *

CHAPTER 1 — GENERAL

1.1 History

Composite deck panels are widely used in the construction of bridges in the United States. The 1986 survey conducted by PCI indicated that twenty-one State Highway Departments and three Turnpike Authorities have specified the use of composite deck panels.

The panels were first used in the early 1950s for construction of bridges on the Illinois Tollway. A number of states began to use the panels in the late 1960s and early 1970s. Also, several research projects were undertaken and completed during that time.[2-6] The results of that research along with earlier design experience form the basis for current design practice.

The earliest installations of deck panels are nearing 35 years of age, and the panels are performing well.[7] In more recent years, isolated problems have been reported, especially cracking in the composite top slab and excessive bowing of deck panels prior to installation. It was these concerns that prompted the producer and user survey and has resulted in this recommended practice report. These problems and the recommended solutions are discussed in this report.

Additional historical information, references and photographs may be found in the special state-of-the-art report, "Precast Prestressed Concrete Bridge Deck Panels," prepared by the PCI Committee on Bridges.[8]

1.2 Materials

Composite bridge deck panels are typically cast using low slump concrete mixes. The use of water reducers and plasticizers are sometimes specified in order to attain the required stress transfer and 28-day strengths. Some agencies specify the use of rust or corrosion inhibitors in the mixes; however, most rely on the cast-in-place top slab to provide sufficient cover to prevent chloride penetration into the composite deck panels. Requirements for air entrainment in the composite deck panels vary.

The types of aggregates used vary widely across the country. Aggregate size should be no larger than ¾ in. to ensure that concrete is thoroughly consolidated below the strand in the shallow pan forms and to achieve concrete strengths specified.

Strand used in deck panels includes nearly every size and type of strand produced. Trends now are toward the use of ⅜ in. diameter strand. The practice of strand use varies depending on considerations of strand development lengths.

Typically, mild steel reinforcement is Grade 60 steel or welded wire fabric.

1.3 Advantages

Common bridge deck construction consists of prestressed concrete girders or steel girders supporting a roadway deck. This deck can be formed as a full depth cast-in-place slab or it can be constructed using precast prestressed composite bridge deck panels as shown in Fig. 1.

If the bridge deck is cast in place for its full depth, forms must be installed and later removed. Forming costs are high and the installation and removal of forms is time consuming. In some situations this creates safety hazards to traffic under the construction or to the workers themselves. Stay-in-place metal forms are sometimes used. These also eliminate the need for form removal, but they are subject to long term corrosion and do not replace or reduce the bottom reinforcement as they do not act compositely with the concrete slab.

Precast prestressed concrete composite bridge deck panels eliminate the need for most of this expensive forming

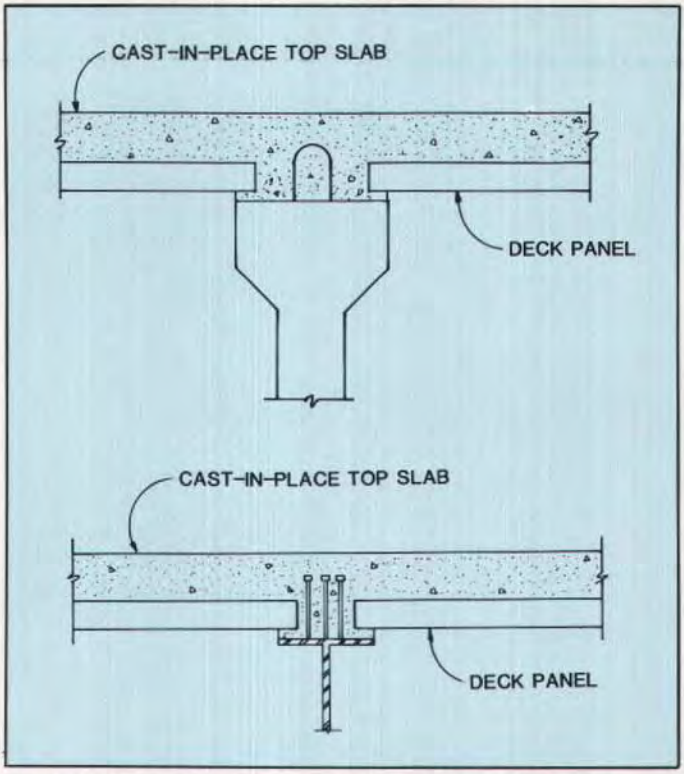

Fig. 1. Typical details of deck panels on prestressed girder (top) and steel girder.

operation. The deck panel and cast-in-place top slab act compositely and result in a significant material and cost savings. The prestressing strands in the deck panels provide the necessary bottom reinforcement in the composite slab system. This eliminates the need for one layer of mild steel reinforcement in the deck. The deck panels are manufactured with dense, high strength concrete that is more resistant to chloride penetration. Because the deck panels are prestressed, the designer has the ability to control stresses in the important tensile stress zone.

Composite bridge deck panels are manufactured in established plants where proven production and quality control methods result in a product which conforms to specifications. Millions of square feet of precast prestressed concrete composite bridge deck panels have been manufactured and installed. They have proven to be a sound economical alternative to conventional forms and are usually chosen for construction when specifications permit alternate solutions.

1.4 Design Considerations

The design of precast prestressed concrete composite bridge deck panels must include an analysis of the panels for stresses due to handling and during construction as well as ultimate strength of the composite section.

Design drawings should show every aspect of production and installation of the composite deck panels, including storage instructions, bearing details, and all other special considerations.

Shop drawings prepared by the fabricator must include all information contained on the design drawings as well as any special information necessary for production. Composite deck panel lengths should be carefully selected with consideration given to tolerances for girder horizontal sweep in order to achieve proper bearing.

When composite deck panels are used on steel or prestressed concrete girders, the designer should recognize that shear studs on steel girders and projecting stirrups on concrete girders must be located to minimize interference with composite deck panel placement.

Torsional stresses are typically not induced in composite deck panels in bridges with straight girders. Torsion in deck panels due to girder movement should be considered in the design when composite deck panels are used on bridges with horizontally curved steel girders or box sections. These are special cases which should be carefully studied.

Stresses from differential movement of long, relatively flexible steel girders should also be evaluated by the bridge designer.

CHAPTER 2 — DESIGN

2.1 General Information

The design of precast prestressed concrete composite bridge deck panels is based on the requirements in the *Standard Specifications for Highway Bridges* published by the American Association of State Highway and Transportation Officials (AASHTO).[1]

As with prestressed concrete members in general, composite deck panels are checked for transfer stresses, handling stresses, service load stresses, and factored strength in shear and flexure.

Research has shown that composite deck panels as presently designed perform well under fatigue test conditions, and therefore no special provisions are needed for fatigue.[2-6] Strand slippage is not a common problem and tolerances for stressing forces as established by specifying agencies are enforced.

Although precast prestressed concrete composite deck panels have been used with success for many years, concerns relating to their production and use, as well as differences between agencies specifying the products, have raised some questions. This chapter addresses those questions and evaluates their effect on design of the composite deck panels.

2.2 Panel Thickness and Size of Prestressing Strand

Panel thicknesses commonly used for composite deck panels have ranged from 2½ to 4½ in. The panels should be made as light as practicable for handling purposes, but this must be evaluated considering the potential for damage if the sections are too thin.

The recommended minimum composite deck panel thickness for general use is 3 in. The panel thickness is an important factor in choosing the size of strands to be used. The relationship of panel thickness to strand diameter should be approximately 8:1 as shown below. This has been reported to give very satisfactory results.

Panel thickness	Maximum strand size
3 in.	⅜ in.
3½ in.	7/16 in.
4 in. or larger	½ in.

The absolute minimum composite

deck panel thickness should be 2½ in. Panels with this thickness, however, require special care during manufacturing, handling, and erection. With ⅜ in. strands, the resulting clear concrete cover is 1¹⁄₁₆ in. Article 9.25.1.2.2 of the AASHTO specifications requires 1 in. minimum cover on prestressing strands in the bottom of slabs. Full scale tests of bridges with composite deck panels indicated that 1 in. of concrete cover on the strands in the panels was adequate.[9]

Deviations from this recommendation are possible, but consideration must be given to the possibility of splitting of the panel ends due to prestress transfer forces. Use of thin panels or large strands may require special end zone reinforcement. Deviations should be submitted to the specifying agency for approval.

2.3 Strand Location

The majority of composite deck panels produced over the years have had strands located at the centroid of the section although eccentric prestressing can result in significant cost savings over concentric prestressing.

Uniform spacing of the prestressing strands is recommended to prevent splitting cracks in the panels.[1] Approximately equal strand patterns may be used where equally spaced strand patterns do not coincide with the composite deck panel width or jacking header spacing.[10]

2.4 Factored Moment Strength and Strand Development

Article 9.27 of the AASHTO Specifications provides a relationship for determining the development length of prestressing strand:

Development length:

$$l_d = (f^*_{su} - \tfrac{2}{3} f_{se})D \qquad (9\text{-}32)$$

where f^*_{su} is the average stress in the prestressing steel at ultimate load.

The term f^*_{su} also occurs in the equation for factored moment strength, M_u, Article 9.17.2, Eq. (9-13). However, the value of f^*_{su} may not always be the same as applied to these two equations. The reason for this is that when short length deck panels are used, there may not be enough total length available to develop the full strength of the prestressing strands.

Article 9.17.4.2 states that the maximum value for f^*_{su} at ultimate load be limited to:

$$f^*_{su} = \frac{l_x}{D} + \tfrac{2}{3} f_{se} \qquad (9\text{-}19)$$

where the term l_x is the distance from the end of the prestressing strand to the center of the panel. In other words, a strand can develop a maximum value for f^*_{su} [Eq. (9-17)] if the full development length, l_d, is available. When l_x is less than l_d, the maximum stress in the strand will be correspondingly less. This will consequently reduce the value of f^*_{su} and the design flexural strength, M_u.

This is demonstrated graphically in Fig. 2. The prestressing reinforcement shown for this panel is insufficient for a span of less than about 5 ft. This is because the allowable f^*_{su} has been significantly reduced due to the small value of l_x. At a span length of just over 5 ft the reduction of f^*_{su} is at its most critical point, but as the span increases, the amount of the reduction becomes less critical because M_u is increasing much faster than the factored moment. For this reason, in deck panels with longer spans, the formation of some splitting cracks or damage resulting from handling at the ends should not necessarily be a cause for rejection. See the recommendations concerning cracks in Section 4.2.1.

This subject of reduced values for f^*_{su} is illustrated in the design example in Appendix A:

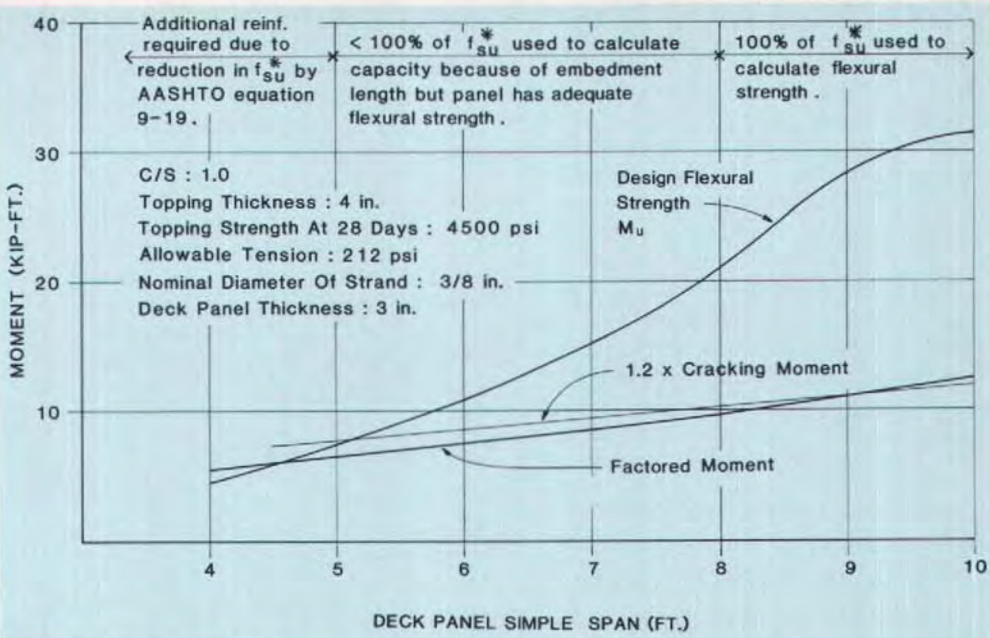

Fig. 2. Moment capacity versus requirement with respect to span.

f^*_{su} = 257 ksi [Eq. (9-17) unreduced value]

f_{se} = 180 ksi

D = 0.375 in.

Substituting these values into Eq. (9-32) gives:

Development length, l_d:

$= [257 - (2/3)(180)](0.375)$

$= 50.1$ in.

In the design example, a reduction in f^*_{su} was made from 257 to 243 ksi in compliance with Article 9.17.4.2. This reduction was a function of the distance from the end of the prestressing strand to the center of the panel (l_x). In order to use the full value of f^*_{su} to calculate design flexural strength, the development length in this example must be 50.1 in. As can be seen, even when f^*_{su} was reduced, the design flexural strength was greater than the actual required. This reveals that development length is not critical for the majority of spans.

Results of research on development length vary substantially. Tests conducted on different size strands have shown that average development lengths of 22 in. for ⅜ in. strands, and 34 in. for ½ in. strands are required.[4,11] Kluge recommends a 62 in. development length for ⅜ in. strands.[6]

Although many values are below, and some are above, the AASHTO Specification requirements, the results are not conclusive enough to justify any recommended changes to these requirements.

2.5 Panel Width and Span

Composite deck panels have been fabricated with widths ranging from 2 ft to as much as 10 ft, with 4 and 8 ft being the most common.

Decisions about panel width should involve discussions with the fabricator and contractor concerning the advantages and disadvantages of wide versus narrow composite deck panels. Wider panels afford more effective use of forms. They also permit stripping and handling to proceed more rapidly because there are fewer pieces to handle.

Preferred composite deck panel widths are 4 and 8 ft. Two foot wide panels may be used at edges or where necessary due to bridge geometry. Trapezoidal or triangular pieces may be used at ends on skewed bridges.

The composite deck panel span length is selected by the designer based on girder spacing. Deck panel spans can vary and are influenced by deck panel thickness, cast-in-place slab thickness, superimposed loads, and required prestress force.

2.6 Stress Transfer and 28-Day Strengths

Article 9.22 of the AASHTO Specifications requires a minimum concrete strength at the time of stress transfer of 4000 psi. Article 9.15 indicates designs are based on a 28-day minimum compressive strength of 5000 psi. These strengths are routinely reached in panel production.

2.7 Corrosion Protection

Cracks which form in the cast-in-place top slab have not been observed to extend to the deck panels (see Section 2.11). Research conducted by the Texas Transportation Institute[6] indicated these cracks did not occur in the test bridges until several months after construction. The cracks most likely formed due to shrinkage and thermal effects. Cores taken after ultimate load tests of the structure indicated the average depth of the cracks was approximately one-half the depth of the cast-in-place top slab. Another study[25] also pointed out the importance of ensuring proper curing methods for the cast-in-place slab.

The use of corrosion inhibiting admixtures or epoxy coated reinforcement in the deck panels is generally not necessary.

2.8 Fatigue

Research indicates cyclic loading up to 11 million cycles had no detrimental effect on the long term performance of precast prestressed concrete composite bridge deck panels.[2,3,4,12,13]

Special attention is therefore not required in the design of composite deck panels for fatigue. However, cyclic loading should be considered in reviewing details for permanent bearings. Proven systems such as those described in Section 2.10 should be used.

2.9 Strand Projections

Research indicates that the overall behavior of bridges with and without deck panel strand projections is very nearly the same.[13,14] The most important factor is adequate bearing for the panels on the bridge girders. The use of strand projections is not an acceptable substitute for good bearings. In many cases, strand projections contribute a substantial cost component by inhibiting the production of deck panels by long line casting and saw cutting. Therefore, it is recommended they not be used.

2.10 Bearing Systems

2.10.1 General

One of the most important components of a bridge constructed using prestressed composite deck panels is bearing of the deck panel on the bridge girder.

Composite bridge deck panels must be supported on the bridge girders by a permanent bearing material providing continuous and solid support and consisting of mortar, grout, concrete or steel.[6] Research conducted on bridges in Florida, [15,16] which used only a soft fibrous material as the final bearing under the ends of the deck panels, indicated:

(a) The overall bridge deck behaved more like simple spans than a continuous slab over the stringers.

(b) The ends of the composite deck

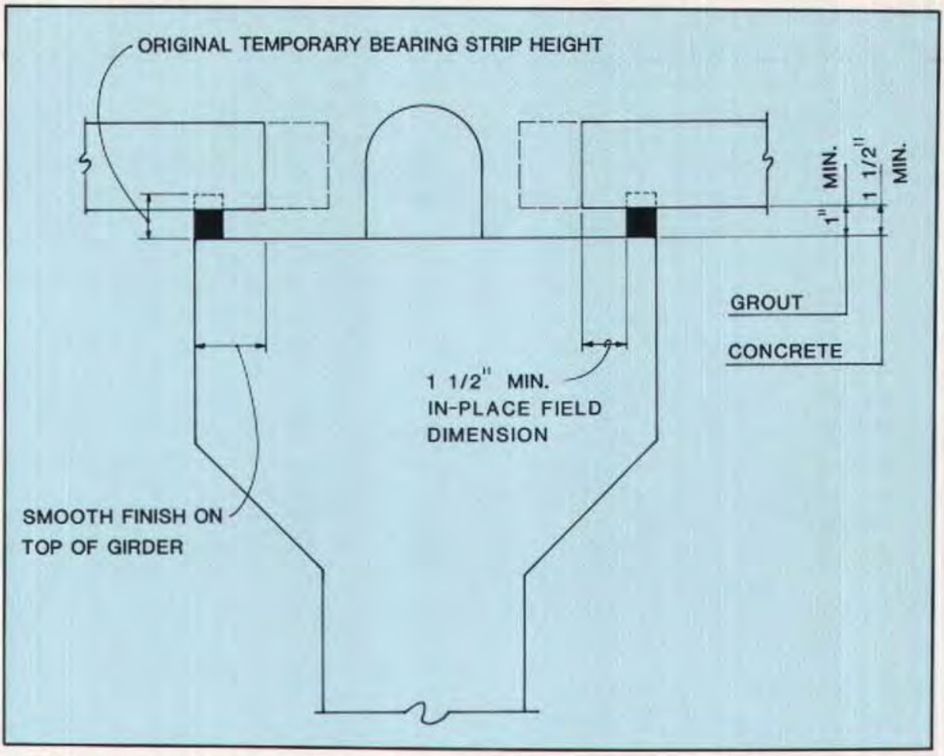

Fig. 3. Dimensions of temporary bearings for supporting deck panels.

panels delaminated from the cast-in-place topping, forcing the topping alone to carry the live load shear.

(c) The majority of these bridges developed extensive cracking. Bridges with this bearing detail will have a reduced service life.[7]

2.10.2 Temporary Bearing

If grout or concrete is used as permanent bearing material, the composite deck panels must be supported above the bridge girders by a temporary bearing system. The temporary bearing system provides support for the panels during erection and until the grout or concrete reaches design strength. The temporary bearing system may be designed to be removed or may remain in place.

Removable systems include wood planks and steel angles or channels attached to the sides of the bridge girders. These supports are removed after the cast-in-place top slab reaches design strength.

Temporary systems designed to remain in place include continuous strips of compressible material such as expanded polystyrene, fiberboard, or bituminous fiberboard. Unyielding materials such as steel or hard plastic shims, which are left in place after grouting, will continue to provide the primary support for deck panels should the permanent grout or concrete bearing material shrink. This will result in undesirable cracking in the bridge deck slab over these rigid bearing points. Therefore, temporary bearing materials which are designed to remain in place must be compressible.

Proper performance of the finished deck requires that the height of the temporary bearing material be adequate to allow grout or concrete to flow easily under the deck panel overhang. This should be a minimum of 1 in. when grout is used. The vertical clearance should be increased to a minimum of 1½ in. if the top slab concrete is to flow under the deck panels to provide permanent support (see Fig. 3). The tops of supporting girders must be smooth enough to allow the grout or concrete to flow readily. The composite deck panels must extend a minimum of 1½ in. beyond the temporary bearing material.

Compressible bearing materials will indeed be compressed due to the weight of the deck panel. This decrease in vertical clearance must be accounted for in sizing the temporary bearings.

2.10.3 Permanent Bearing

When composite deck panels were first used, they were supported on continuous hand placed mortar beds. This method of support works well and is still widely used. The mortar bed should be a minimum of 1 in. thick and 3 in. wide. The thickness can be controlled by using small wooden or plastic pegs cut to the proper length acting as spacers. The spacers must be retrieved before setting deck panels so as not to provide permanent support when the mortar shrinks. Deck panels must be placed on the mortar beds before the mortar hardens to ensure they have even support.

The use of grout, concrete, and mortar beds for permanent bearings have been tested and found to perform satisfactorily through as many as two million applications of simulated design axle load with impact.[2,5,9,17]

When flowable grout or concrete is used, venting is required to ensure that final placement is not inhibited by air lock. This should be accomplished by leaving small gaps in the strips of compressible bearing material at a spacing of approximately 36 in.

If grout is used as the permanent bearing material, it should be a high strength, low slump cementitious grout or a nonshrink grout with a maximum compressive strength of 5000 psi. The strength of the grout should be verified by tests prior to placement of the cast-in-place top slab.

If the top slab concrete is designed to flow under the composite deck panel for permanent support, proper placement procedures should be followed. Concrete should be deposited in continuous strips over the girders and allowed to flow under the ends of the composite deck panels. Concrete should then be placed on the remainder of the panels. This procedure improves the flow of concrete under the panel ends, helps eliminate air pockets, and places concrete under the panels before the compressible temporary bearings are loaded with the weight of the fresh concrete.

2.10.4 Installation Profile

The contour of the roadway surface usually does not follow the top surfaces of the girder. This is due to girder camber, vertical curvature of the structure, superelevation of the roadway, or a combination of these. To account for elevation variations, there are two options to consider for support of the composite deck panels: (1) The composite deck panels can have a constant, uniform thickness permanent bearing (constant fillet) and therefore a variable thickness top slab, or (2) The composite deck panels can have variable thickness permanent bearing (variable fillet) with a relatively constant top slab thickness.

The advantage of the constant fillet option is the relative ease of installation of the temporary or permanent bearing system. Disadvantages of this option include the varying thickness of the topping, having the top slab reinforcing steel not parallel with the roadway surface, or providing variable height chairs to support the reinforcing steel parallel to the roadway surface. The variable thickness top slab uses a greater volume of concrete. The cost associated with this extra weight may be offset by savings in the erection of the deck panels on constant fillet bearings. The added dead load of the thickened portions of the slab should be taken into account by the bridge designer in the design of the girders and other elements of the bridge structure. No research has been found that would indicate that performance and service life of a bridge are adversely affected due to a variable thickness top slab.

The variable fillet system results in a constant thickness bridge deck where the composite deck panels are parallel to the roadway surface and the reinforcing steel. The installation of variable thickness bearing systems requires careful dimensional control and is somewhat more difficult to accomplish than constant fillet systems.

2.11 Load Distribution

A subject of past and current research has been the ability of this deck system to distribute wheel loads in the direction transverse to the strands in the composite deck panels. One question has been the effect of the joints between adjacent panels. Continuity at this joint is provided by the cast-in-place portion of the deck. Another question has been the quantity and location of reinforcement placed within the panel transverse to the strands.

Research results indicate that the presence of the joint is not detrimental to the load distribution performance of bridge deck systems using composite deck panels.[2,5,6] Tests have been performed with longitudinal (with respect to the bridge) distribution reinforcement placed directly on top of the composite deck panels.[6] Tests have also been performed using supplemental reinforcement directly on the composite deck panels across the joint in addition to the normal longitudinal reinforcement.[2]

Results of these tests demonstrate that the concrete top slab successfully transfers wheel loads across the joints. The supplemental joint reinforcement does not improve performance.[2] In all tests performed with concentrated loads placed immediately adjacent to the butt joint, the mode of failure was punching shear.

Research performed in Texas[2] and Florida[6] indicates that even at failure loads there was no tensile cracking observed on the bottom of the cast-in-place top slab directly over the butt joint. This demonstrates the noncritical behavior of flexural strength across the joint.[6]

Research performed in Texas on inservice bridges[5] and in laboratory full scale mockups[2] indicated a tendency for shrinkage and thermal cracks to form in the cast-in-place top slab directly above the joint between adjacent composite deck panels. The cracks extended down approximately one-half the way through the top slab. The presence of these cracks did not adversely affect the ability of the slab to transfer wheel loads across the joints.[2] It therefore can be concluded that the distribution reinforcement in the cast-in-place top slab performs better when placed toward the top to control shrinkage and thermal cracking than when placed at the bottom of the topping in an attempt to control flexural cracking. Reinforcement placed at the top of the slab also serves as distribution reinforcement by providing steel for cantilever action across the composite deck panel joints.

The distribution reinforcement in the cast-in-place top slab should be placed directly under the main transverse top flexural reinforcement.[6] The main flexural reinforcement should be placed with attention to minimum cover requirements in the AASHTO Specifications.

In addition, research in Florida[6] was

conducted to determine reinforcing steel requirements transverse to composite deck panel spans. Panels were tested under simulated concentrated wheel loads to failure. These failures were attributed to punching shear at load levels many times design load. The basic conclusion of this research was that #3 bars at 12 in. centers (0.11 in.^{2}/ft) should be the minimum transverse steel requirement. The primary purpose of this reinforcement is to compensate for the potential effect on strand development due to transverse tensile strains resulting from two-way plate action.

A study was performed in Texas on in-service bridges.[5] The 3 in. thick composite deck panels had #2 bars at 6 in. centers (0.10 in.^{2}/ft) placed across the top of the strands. There was no evidence that this level of reinforcement resulted in any detrimental effects on the performance of the deck. However, the study was limited to widths of 4 ft and spans of 7 ft 3 in.

AASHTO has adopted the use of 0.11 in.^{2}/ft as the minimum reinforcement to be placed in the composite deck panel transverse to the strands. This level of reinforcement has been shown to be satisfactory.

The placement of the transverse reinforcement above the strands has been satisfactory although the likelihood of cracking during handling is increased due to this steel being located above the neutral axis. The placement of the transverse reinforcement, whether above or below the strands, should be left to the discretion of the designer with due regard to the minimum cover requirements of AASHTO.

2.12 Composite Behavior

Mechanical shear connectors are not necessary to achieve composite action between the deck panel and the cast-in-place slab.[3,17] Full composite action is achieved if the deck panel surface is roughened and is free of contaminants. Panels should be raked in the direction parallel to the strands in order to minimize the reduction in section modulus.[2,13,14,18]

CHAPTER 3 — MANUFACTURE

3.1 Forms

The condition of forms used for casting composite deck panels is of the utmost importance to the quality of the product. Forms should be constructed of steel and properly designed to manufacture this product. Side forms should be constructed to eliminate product damage at the time of stripping. This may be accomplished with side forms which have a slight draft or which can be loosened or removed.

Forms should be cleaned after each pour to prevent a build-up of concrete. Forms should be scheduled for regular cleaning. The flatness of the form soffit should be checked regularly to ensure that the finished products meet required tolerances (see Section 4.1). The form may slope uniformly as necessary for drainage.

Uneven forms and poor form condition are two of the items that have been reported as causes of problems such as cracking, incorrect strand position, and damaged panels.

Excessive form temperatures at the time of concrete placement should be avoided. The form should be shaded and/or cooled by spraying with water as required. Additional information may be found in PCI MNL-116.[19]

3.2 Strand Positioning

Deck panels may be manufactured in a form using headers between individual units or in a continuous length and sawn to the proper length.

In a process containing headers, the most effective means of maintaining strand position is the use of accurately manufactured steel headers securely fastened to the forms. Chairs should be used in forms containing headers where the strand position is not controlled adequately by the header. Strand position in continuous panel forms without headers is most effectively maintained by the use of chairs located at sufficient intervals along the length of the bed.

It is important to note that in each of these situations the position of the strand is a function of the contour of the forms. Further, whatever means is used to position strands, it should be unaffected by the concrete placing operation.

3.3 Strand Surface Condition

The length required to develop the tension in the strand is of importance due to the relatively short length of the panels. Conditions detrimental to bonding of the strand must be avoided. The condition of the strand surfaces is of primary concern. Care must be taken to prevent contamination of the strands from the form release agent or other substances. The presence of a light coating of rust on the strands is not detrimental.[20-23]

In response to requests from prestressed concrete producers, several manufacturers of strand currently market "cleaned strand." This is normal low relaxation or stress relieved strand which has been cleaned of the lubricant remaining from the manufacturing process.

3.4 Concrete Work

No special concreting procedures are necessary to produce high quality composite deck panels. The procedures given in MNL-116[19] will achieve the desired results.

3.5 Curing

Prestressed concrete composite deck panels can be cured using either accelerated heat curing methods or natural curing. When using accelerated heat curing, attention should be given to the behavior of the forms as they expand and contract. Proper form design and construction and the uniform application of heat are essential to prevent product damage. These procedures are outlined in PCI MNL-116,[19] Sections 3.4 and C3.4.

If curing compounds are used, they must be of a type that will not impair the bond of the cast-in-place top slab to the composite deck panels.

3.6 Detensioning Procedures

Another factor affecting strand development length is the method used for detensioning. A sudden release of the prestressing force has been shown to increase strand development length. It is therefore desirable to release the prestress force gradually. In beds with headers, the strands should be cut slowly with acetylene torches.

When composite deck panels are fabricated in a continuous length, strands between individual panels are detensioned more rapidly by the saw cutting process. Therefore, provisions to monitor strand slippage must be made. Prior to initial release of prestress at the ends of the bend, reference lines should be drawn across the end panels as near as possible to their free ends. All strands should be marked at a measured distance from this reference line so that any strand slippage may be measured after initial detensioning.

Strand slippage on the ends of saw cut panels can be visually observed. In short products such as deck panels, any slippage beyond a slight dimpling may be excessive. The effect of any slippage must be therefore evaluated.

For further information on detensioning, see PCI MNL-116,[19] Section 2.3.

3.7 Stripping

Several methods are used to lift and handle precast prestressed concrete composite deck panels. Inserts or lifting loops near the four corners of the panels are most often used. Stripping or lifting the panels by attachment of lifting devices to strand projections must be avoided. A design review of any handling system is recommended.

3.8 Storage

Storage areas must be smooth and well compacted to resist settlement of dunnage and resultant product damage. Stacks of deck panels should be supported by continuous strips of dunnage directly on the ground perpendicular to the strands near the ends. Intermediate dunnage between panels may be full length or provide support near all four corners. A template should be used to locate the dunnage to ensure that support throughout the stack is uniform and the dead load of upper panels does not induce any unwanted stresses into panels stored below. Stacks of panels should be limited to a height that will not cause settlement of the ground dunnage or crushing of intermediate dunnage.

Composite deck panels should be stored for the absolute minimum period of time possible. Storage over the winter months requires special considerations as freeze/thaw cycles may result in soil heaving and loss of stack levelness, which can result in product damage. Stacks may be supported on more rigid foundations or shifted as needed when dunnage heaves. Panels with strand projections should be oriented in a manner so projections are not subject to damage by vehicular traffic moving in aisleways.

CHAPTER 4 — PRODUCT TOLERANCES, CRACKING AND DAMAGE REPAIR

4.1 Tolerances

The dimensional tolerances shown in Fig. 4 are recommended for use by specifying agencies and producers. Through careful production practice, these tolerances can be met and will result in high quality products.

4.2 Cracking

4.2.1 Types of Cracking

Due to the constitution of concrete, any concrete product can experience random cracking. Precast prestressed concrete deck panels are no exception.

As with other types of products, the primary question arising over cracks which occur is to determine which ones are structurally significant. Most cracks in deck panels have no structural importance while some cracks indicate loss of structural integrity. For the latter, methods of evaluation and repair are necessary.

Due to the relatively short length of composite deck panels, the most objectionable cracks are typically those that would increase the strand development length with a resulting decrease in panel capacity. In general, cracks that would indicate an increase in development length are:

1. Two cracks, each occurring within 1 in. of two adjacent strands.
2. Corner cracks or breaks involving two or more strands.
3. Cracks parallel to and along more than 25 percent of the strands.

Cracks of these types can be a cause for rejection of composite deck panels because the effective prestress force in the panel may have been significantly reduced.

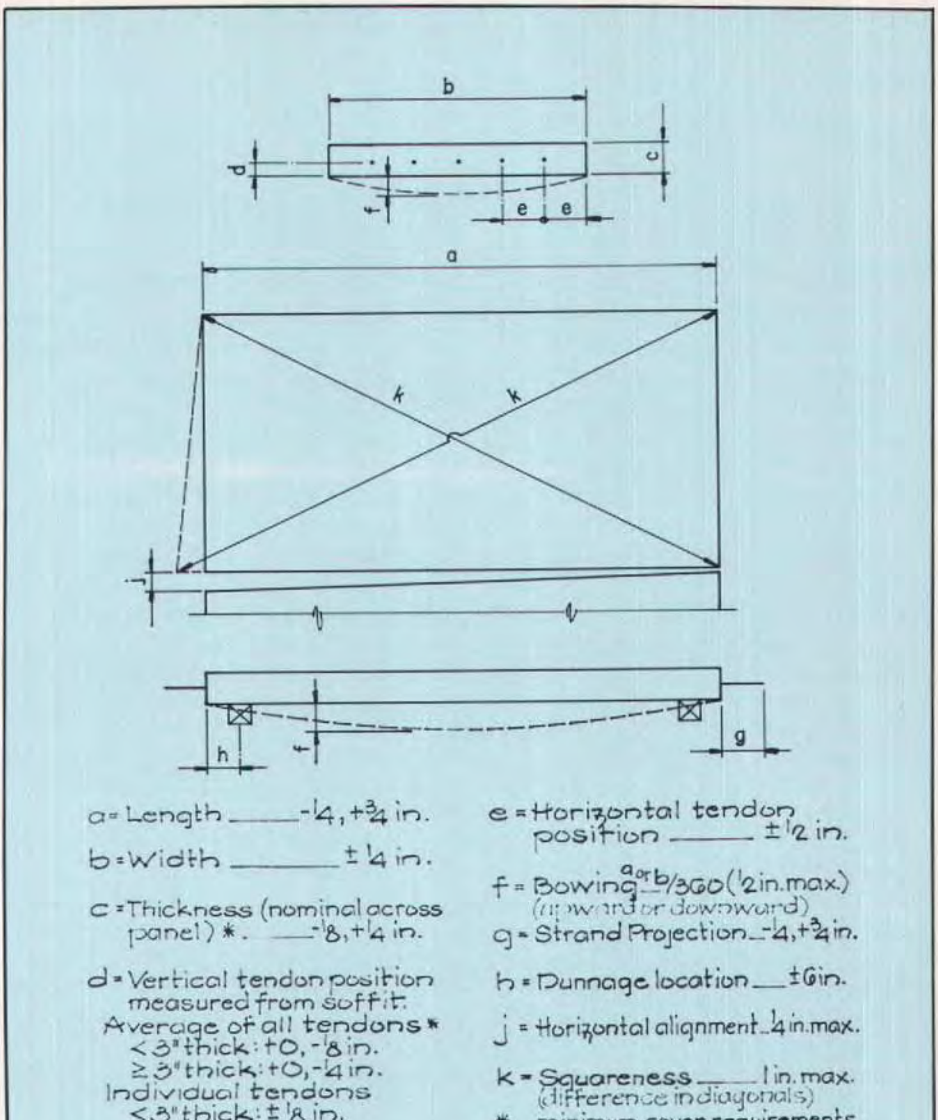

Fig. 4. Deck panel dimensional tolerances.

Cracks of less significance include shrinkage and other cracks that are not in proximity to the strands. There are no specific types of cracks which are defined as acceptable. Common practice is for cracks to be evaluated on a panel by panel basis by qualified individuals. Because of the difficulty of evaluation, it is extremely important to use methods of design and production which eliminate as many potential sources of cracking as possible.

4.2.2 Repairs

At the present time, the only universally accepted method of correcting repairable cracks in prestressed concrete deck panels is epoxy injection. Cracks which cannot be injected because of their very small size may be painted or covered with epoxy.

CHAPTER 5 — SHIPPING AND HANDLING

5.1 Panel Length and Width

Composite deck panel lengths and widths vary due to bridge geometry. In general, the wider and longer deck panels are more susceptible to damage during shipping and handling. Handling stresses must be considered in the design of the panels.

As with all precast elements, composite deck panels should be handled in a manner consistent with their design. Panels should be handled only with approved devices at designated locations. Long or wide panels may be handled using normal equipment and techniques, but extra care must be taken due to their increased size.

5.2 Support on Trailers

Composite deck panels should be loaded on trailers with dunnage located as described in Section 3.8. Care should be used in securing the stacks to the trailer to ensure that no excessive loads will be induced in the members. Tie-down straps must fall over lines of dunnage and protective blocking must be used to prevent damage to the panels. Banding of several panels together in both directions has been used to eliminate shifting of panels and supports.

Damage to panels in transit has been reported for stacks toward the rear end of trailers. For this reason, panel stacks may be reduced to half the normal shipping height for the last 20 ft of the trailer length.

Drivers and loading personnel should be given instructions on proper methods of support and correct tie-down procedures.

5.3 Multiple Panel Handling

Great care should be exercised when multiple panel handling is used. It is difficult to support a stack of panels by a method that does not induce undesirable stresses into the lower panels. Special support slings or strong backs are required to prevent overloading the lower panels. Methods for multiple panel handling should be closely reviewed by a qualified engineer to ensure that no damage will result from the multiple panel handling method.

5.4 Jobsite Storage

A common problem which results in damage to composite deck panels is the lack of adequate storage areas at the jobsite. An area should be prepared that is capable of supporting stacked deck panels without settlement of the ground contact dunnage. Composite deck panels are very easily cracked when warped due to uneven settlement of dunnage. Section 3.8 describes recommended storage methods. The contractor must be notified of the potential for damage to panels if he fails to provide an adequate storage area.

Composite deck panels must not be stacked on previously erected deck panels until the topping is cast and these loads are evaluated by the designer. If temporary dunnage is placed on previously erected panels, it must be placed over the supported ends of the panel only after the permanent bearing system is in place. Placing dunnage inboard of the end supports will induce high flexural and shear stresses which composite deck panels were not designed to resist.

CHAPTER 6 — ERECTION

6.1 Panel Handling

The same care used in loading deck panels at the plant must be exercised at the jobsite for unloading. Equipment designed for lifting these units should be used. Front end loaders and other grading equipment must not be substituted for proper cranes.

Multiple panel stacks must not be placed on girders unless it can be shown that no damage to the panels or temporary bearings will result. Composite deck panels must be placed so they do not receive support from portions of the bridge structure on which they were not designed to bear. Any support other than at the designated bearing locations can result in damage to the composite deck panel and poor performance of the total bridge deck system at that location. Composite deck panels placed so as to completely cover over diaphragms can also cause difficulties in placing and vibrating concrete in the cast-in-place diaphragms.[13,14]

Construction loads imposed on individual panels must not exceed the superimposed design dead load capacity. Panels must not be used to support bundles of reinforcement, heavy equipment, or concrete buckets. Until the top slab is cast and cured, the bridge deck panels will not support heavy concentrated loads.

During erection of the bridge girders, the composite deck panel lengths should be verified. The sweep tolerance in long girders may result in short bearing for panels which are the correct design length.

All erection procedures should comply with the *Recommended Practice for Erection of Precast Concrete*[23] where applicable.

CHAPTER 7 — INSPECTION

7.1 Plant Inspection

Thorough and regular plant inspection during all phases of the manufacturing process is one key to a successful project.

7.1.1 Forms

Composite deck panel forms must be inspected daily for cleanliness, alignment, and general condition as outlined in Section 3.1.

7.1.2 Strand Position

Strand position must be verified prior to placing concrete. This can easily be accomplished with the use of a gage designed to check the space between the strands and the form. Measurements should be made to verify that the chairs or headers are maintaining the proper strand position. Chairs can deform during concrete placement or allow a strand to sag if the spacing is too great.

7.1.3 Strand Surface Condition

Inspection must verify that the strand surfaces are clean just prior to casting. If contamination exists, the strands must be thoroughly cleaned with an effective solvent. This is very difficult to do adequately, so the emphasis must be on eliminating sources of contamination. A light coating of rust on the strand is acceptable.

7.1.4 Detensioning

Inspection procedures at the time of detensioning primarily consist of verification that the method used is one which has been tested and approved. The required concrete strength necessary for stress transfer must be attained before detensioning and must be verified by the use of concrete cylinder tests.

7.1.5 Stripping

Composite deck panels should be inspected for damage at the time of stripping. The inspector should also observe the means by which the panels are stripped from the form and the methods of handling.

7.1.6 Post-Pour Inspection

A complete post-pour inspection must be made within 24 hours of the time of prestress release. This inspection must cover any deficiencies in the product's physical dimensions, and must give particular attention to any cracks or needed repairs.

7.1.7 Storage and Preshipment Inspections

After the composite deck panels have been stripped and have received a post-pour inspection, they are generally removed to a storage area. Periodic checks of panels in storage must be made to ensure that blocking is properly located and that no settlement in the stacks has occurred. Any damage observed should be evaluated for remedial action. Due to the potential for damage from settlement of dunnage or vehicular traffic, panels must be reinspected at a time just prior to shipment. This will provide documentation to show that the panels did not leave the producer's facility in a damaged condition.

7.2 Jobsite Inspection

Producer follow-up with inspection and information at the jobsite is the second key toward success in these projects.

7.2.1 Unloading Inspection

An inspection of the panels at the jobsite should be made during unloading. This inspection will determine if any damage has occurred during shipment of the panels to the jobsite. This will also provide documentation to show that damage discovered later probably resulted from mishandling of, or improper superimposed loads on, the products at the jobsite.

7.2.2 Temporary Storage

Temporary storage of the composite deck panels must be reviewed at the jobsite to ensure that an adequate area has been prepared. This storage area should meet the same requirements described in Section 3.8.

7.2.3 In-Place Inspection

Composite deck panels should be inspected after erection to document that no damage occurred as they were placed in their final position. Attention must be given to the bearing details to ensure that the composite deck panels will be properly supported on the bridge girders.

Prior to placement of the concrete top slab, all composite deck panels should be checked for any concrete laitance or other contaminants on the top surface which would adversely affect bond. Panels should be water washed and standing water removed prior to concrete placement.

* * *

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* * *

NOTE: Discussion of this report is invited. Please submit your comments to PCI Headquarters by December 1, 1988.

APPENDIX A — DESIGN EXAMPLE

In order to illustrate the AASHTO requirements for the design of composite deck panels, as found in the *Standard Specifications for Highway Bridges*,[1] an example follows. All articles referenced are from the above mentioned publication. Refer to Fig. A1 for the example configuration. Sign convention: + compression; − tension.

Step 1 — Spans

Deck panel alone (Article 3.24.1.1)
$S = 7$ ft 6 in. + 3 in. (panel thickness)
Use $S = 7$ ft 9 in.
Deck panels on concrete girders composite with top slab [Article 3.24.1.2(a)]
$S = 7$ ft 6 in. + 2 x (1½ in.) (temporary bearing strips)
= 7 ft 9 in.

Step 2 — Loads

Dead Loads (Article 3.3.6)
Deck panel:
$W = 3$ in. at 150 pcf = 37.5 psf
Top slab:
$W = 5$ in. at 150 pcf = 62.5 psf
Wearing surface:
$W = 35$ psf
Construction Live Load
$W = 50$ psf
Wheel Loads (Fig. 3.7.7A)
HS 20: $P = 16{,}000$ lb
Impact factor for wheel loads (Article 3.8.2)

$$I = \frac{50}{L + 125} \leq 30 \text{ percent [Eq. (3-1)]}$$

$I = 30$ percent

Step 3 — Material Properties

Deck Panels
$f'_{ci} = 4000$ psi (Article 9.22)
$f'_c = 5000$ psi (Article 9.15)
Top Slab
$f'_c = 4500$ psi (Div. II, Article 4.5.2 and Table 4.1)
Prestressing Steel

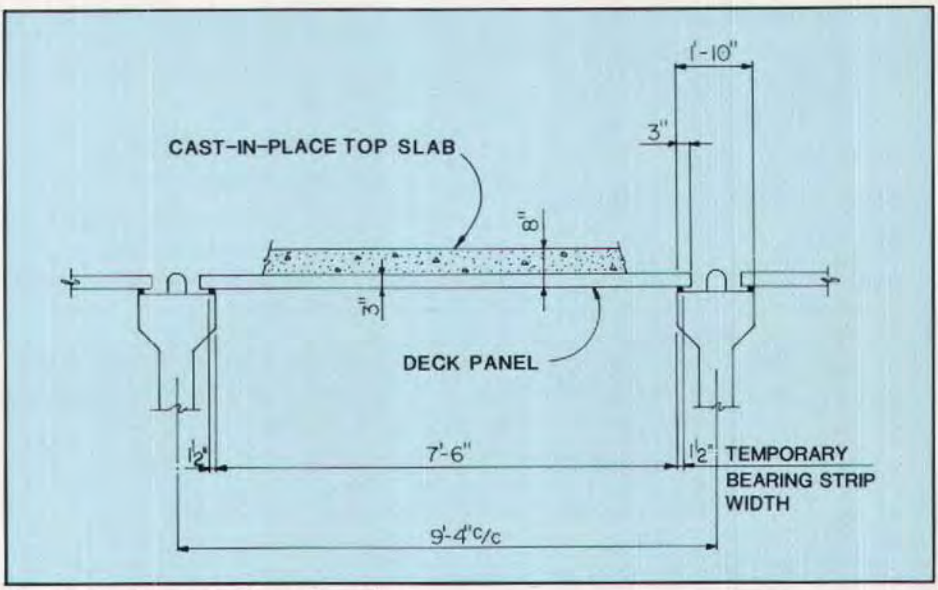

Fig. A1. Design example configuration.

$f_s' = 270{,}000$ psi (Article 9.3.1) (low relaxation)

Mild Reinforcing Steel

$f_y = 60{,}000$ psi (Article 8.3)

Step 4 — Allowable Stresses

Article 9.15 describes the allowable stresses associated with the analysis of a bridge structure.

*Temporary compressive stress in concrete .... $0.60 f_{ci}'$

(0.60) (4000 psi) = 2400 psi

*Temporary tensile stress in concrete in tension areas with no bonded reinforcement .... 200 psi or $3\sqrt{f_{ci}'}$

$3\sqrt{4000} = 189$ psi < 200 psi

Therefore, 189 psi controls

*Temporary prestressing steel stress .... $0.75 f_s'$

(0.75) (270 ksi) = 202.5 ksi

*Temporary — At casting bed anchors after seating.

†Final compressive stress under service load conditions .... $0.40 f_c'$

(0.40) (5000 psi) = 2000 psi

†Final tensile stress under service load conditions in the precompressed tensile zone ..... $6\sqrt{f_c'}$

$6\sqrt{5000} = 424$ psi

†Final — After losses have occurred.

Step 5 — Calculate Section Properties

Section properties required for analysis of the deck will be those of the deck panel alone for analysis as a simple span supporting its self weight plus the cast-in-place top slab plus the construction load. The composite properties for the deck panel and top slab will be used to resist live loads.

Top slab transformation (see Fig. A2):

Use Article 8.7.1.

$E_{precast} = 57{,}000\sqrt{5000} = 4030$ ksi

$E_{top\,slab} = 57{,}000\sqrt{4000} = 3824$ ksi

$n = E_{top\,slab}/E_{precast}$

$n = 3824/4030 = 0.949$

Transformed area =

(12)(5)(0.949) = 56.9 in.^{2}

$b_t = (56.9)(5) = 11.4$ in.

Bare composite panel:

For a 12 in. panel width:

Area = (3)(12) = 36 in.^{2}/ft

$I = bh_p^3/12$

$= (12)(3^3)/12$

$= 27$ in.^{4}/ft

$S_{top\ precast\ bare} = I/C_{top\ precast}$

$= 27/1.5$

$= 18$ in.^{3}/ft

$S_{bottom\ bare} = I/C_{bottom}$

$= 27/1.5$

$= 18$ in.^{3}/ft

Composite section:

Area = (11.39)(5) + (3)(12)

$= 93.0$ in.^{2}/ft

$Y = \Sigma(AY)/\Sigma A$

$= (36)(1.5) + (57)(3 + 5/2)/(93.0)$

$= 3.95$ in.

$Y_{top} = 8 - 3.95 = 4.05$ in.

$I = \Sigma(I + Ad^2)$

$= 27 + 36(3.95 - 3/2)^2 +$

$(1/12)(11.4)(5)^3 + (57)(4.05 - 5/2)^2$

$= 499$ in.^{4}/ft

$S_{top} = I/Y_{top} = 498/4.05 = 123$ in.^{3}/ft

$S_{bottom} = I/Y = 499/3.95 = 126$ in.^{3}/ft

$S_{top\ precast} = I/(Y - h_p) = 499/3.95 - 3) =$ 525 in.^{3}ft

Step 6 — Calculate Moments and Stresses Neglecting the Prestress Force

The following moments and resulting stresses in the deck panel and the total composite bridge deck must be calculated:

1. In the panel under its own weight.
2. In the panel due to the weight of the concrete top slab only.
3. In the panel due to the construction load only.
4. In the composite section due to the future wearing surface only.
5. In the composite section due to vehicle live load only.

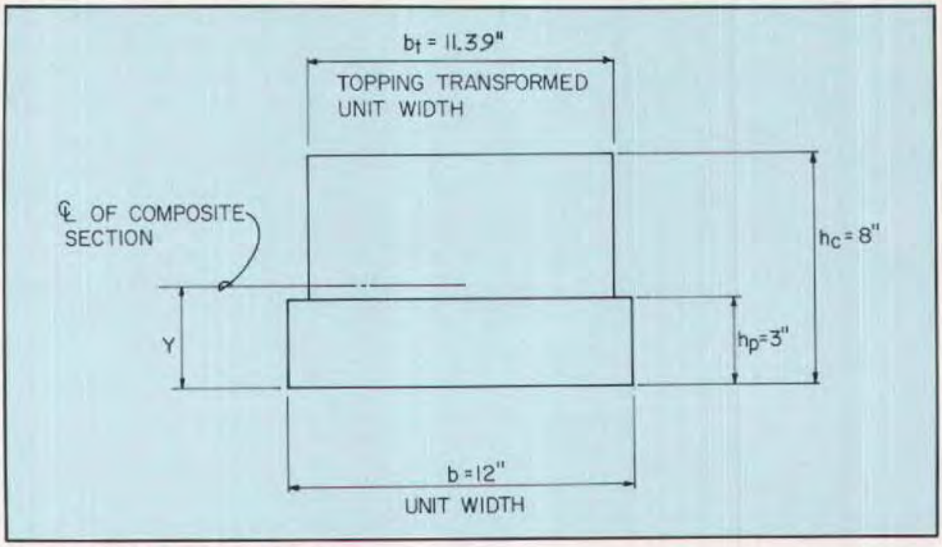

Fig. A 2. Unit transformed area.

1. Deck panel under its own weight (DL):

$M = WS^2/8$
$= (37.5)(7.75)^2/(8)$
$= 282$ lb-ft/ft

$f_{top\ precast} = M/S_{top\ precast\ bare}$
$= (282)(12)/(18)$
$= 188$ psi

$f_{bottom} = M/S_{bottom\ bare}$
$= (282)(12)/(18)$
$= -188$ psi

2. Deck panel under top slab weight only (DL):

$M = WS^2/8$
$= (62.5)(7.75)^2/(8)$
$= 469$ lb-ft/ft

$f_{top\ precast} = M/S_{top\ precast\ bare}$
$= (469)(12)/(18)$
$= 313$ psi

$f_{bottom} = M/S_{bottom\ bare}$
$= (469)(12)/(18)$
$= -313$ psi

3. Deck panel under construction load only (LL):

$M = WS^2/8$
$= (50)(7.75)^2/(8)$
$= 375$ lb-ft

$f_{top\ precast} = M/S_{top\ precast\ bare}$
$= (375)(12)/(18)$
$= 250$ psi

$f_{bottom} = M/S_{bottom\ bare}$
$= (375)(12)/(18)$
$= -250$ psi

4. Composite section under future wearing surface load (DL):

Positive moment for interior spans =
$WS^2/10 = (35)(7.75)^2/(10)$
$= 210$ lb-ft/ft

$f_{top} = M/S_{top}$
$= (210)(12)/(123)$
$= 20$ psi

$f_{top\ precast} = M/S_{top\ precast}$
$= (210)(12)/(525)$
$= -5$ psi

$f_{bottom} = M/S_{bottom}$
$= (210)(12)/(126)$
$= -20$ psi

5. Composite section under vehicle load (LL):

Unless more exact methods are used, apply Eq. (3.24.3) to find the moment in lb-ft per ft of width.

$$M = \frac{S + 2}{32} P$$

where P = 16,000 lb and,
and S = 7.75 ft (Step 1).

$$M_{simple} = \frac{7.75 + 2}{32}(16{,}000)$$
$$= 4875 \text{ lb-ft/ft}$$

Using Eq. (3.24.3.1):

$$M_{continuous} = (0.80)(4875)$$
$$= 3900 \text{ lb-ft/ft}$$

Using Eq. (3.8.2):

$$M_{design,\ service} = (1.3)(3900)$$
$$= 5070 \text{ lb-ft/ft}$$

$$f_{top} = M/S_{top}$$
$$= (5070)(12)/(123)$$
$$= 495 \text{ psi}$$

$$f_{top\ precast} = M/S_{top\ precast}$$
$$= (5070)(12)/(525)$$
$$= -116 \text{ psi}$$

$$f_{bottom} = M/S_{bottom}$$
$$= (5070)(12)/(126)$$
$$= -483 \text{ psi}$$

Step 7 — Strand Estimate

In order to make an estimate of the strand requirements for the deck panels, the governing bottom tension in the deck panels must first be determined from the previous step. Using the governing bottom tension from Step 6, an estimate can be made of the number of strands that are necessary to bring the stresses within the allowable values prescribed in Step 4. In making this estimate, assume a 15 percent total loss of prestress as an approximate value, to be verified in a later step.

Deck Panel Tensile Stress Summary

Stresses from:	
Panel weight	−188 psi
Top slab weight	−313 psi
Future wearing surface weight	− 20 psi
Vehicle wheel load	−483 psi
	−1004 psi

Using ⅜ in. diameter 270K low relaxation strand with an assumed total prestress loss of 15 percent per strand:

$$P_{all\ losses} = (0.75)(0.085)(270)(0.85)$$
$$= 14.63 \text{ kips}$$

Required stress reduction =
Actual stresses − Allowable stresses

Using the deck panel tensile stress summary and the allowable stresses calculated in Step 4:

Required stress reduction =
$$1004 - 424 = 580 \text{ psi}$$

Solving for the number of strands required to provide this concentric stress, the required stress reduction equals:

$$\frac{(\text{Number of strands})\ (P_{all\ losses})}{\text{Panel unit area}}$$

$$580 = \frac{(\text{Number of strands})\ (14{,}630)}{(12)\ (3)}$$

Therefore, the number of strands is 1.43 strands per foot. For a 4 ft wide panel, this would give:

Total number of strands =
$$(4)(1.43) = 5.71 \text{ strands}$$

Use six strands as an initial estimate.

Step 8 — Check Panel Factored Moment Strength With Strand Estimate

Required flexural strength:

$$M_u = 1.3\ (M_{dead} + 1.67\ M_{LL+I})$$
$$= 1.3\,[282 + 469 + 210 + 1.67(5070)]$$
$$= 12{,}256 \text{ lb-ft/ft (required strength)}$$

Capacity using estimated strength:

$$\phi M_n = \phi A^*_s f^*_{su} d \left(1 - 0.6\ \frac{p^* f^*_{su}}{f'_c}\right) \quad (9\text{-}13)$$

$$f^*_{su} = f'_s \left(1 - \frac{0.5\, p^* f'_s}{f'_c}\right) \quad (9\text{-}17)$$

but not greater than:

$$f^*_{su} = \frac{l_x}{D} + \frac{2}{3} f_{se} \quad (9\text{-}19)$$

A^*_s = area of prestressing steel per foot
$= (6)(0.085)/(4)$
$= 0.1275$ in.²/ft

d = distance from extreme compression fiber to centroid of prestressing force
$= 6.5$ in.

p^* = ratio of prestressing steel
$= A^*_s/bd$
$= (0.1275)/(12)(6.5) = 0.0016$

f'_c = 4500 psi (top slab)
f'_s = 270 ksi
f_{se} = (0.85)(0.75)(270)
= 172 ksi (based on 15 percent prestress loss assumption)
D = nominal diameter of strand
= 0.375 in.
l_x = distance from end of prestressing strand to center of panel = 48 in.

Substituting into Eq. (9-17):

$$f^*_{su} = 270 \left[1 - \frac{(0.5)(0.0016)(270)}{(4.5)}\right]$$
$$= 257 \text{ ksi}$$

Checking this value against that of Eq. (9-19):

f^*_{su} = (48)/(0.375) + ⅔(172)
= 243 ksi

indicates that the average stress in the prestressing steel at ultimate load (f^*_{su}) must be limited to 243 ksi due to the limiting development length provided by the panel.

Solution of Eq. (9-13) yields:

$$\phi M_n = 1.0\,(0.1275)(243)(6.5)\left[1 - 0.6\ \frac{(0.0016)(243)}{4.5}\right]$$
$$= 15{,}912 \text{ lb-ft/ft}$$

A ϕ factor of 1.0 must be applied to this moment as specified in Article 9.14 for factory produced members.

This value is greater than the calculated required strength of 12,256 lb-ft; hence, six strands are adequate for factored moment strength. Also, note that the factored moment capacity using the reduced value of f^*_{su} is approximately 1.30 times greater than required. This indicates that development length is not important for this panel as it might be for a panel with a shorter span.

Step 9 — Check Maximum and Minimum Steel

Article 9.18.1 states that the maximum prestressing steel is limited to:

$$\frac{p^* f^*_{su}}{f'_c} \leq 0.3 \qquad (9\text{-}20)$$

Substituting for this example:

(0.0016)(243)/(4.5) = 0.0864

which is less than the prescribed maximum of 0.3.

Article 9.18.2 requires that the total amount of prestressed and nonprestressed reinforcement be adequate to develop a factored moment at least 1.2 times the cracking load calculated on the basis of modulus of rupture in accordance with Article 9.15.2.3 or:

$$\phi M_n \geq 1.2\, M_{cr\ tot}$$

The moment necessary to crack the composite slab section without considering the prestress force as defined in Article 8.13.3 is:

$$M_{cr} = f_r I/Y$$

where

f_r = modulus of rupture as specified in Article 8.15.2.1.1 as $7.5\sqrt{f'_c}$
= $7.5\sqrt{5000}$
= 530 psi
I = moment of inertia of composite section
= 499 in.⁴/ft (see Step 5)
Y = distance from centroidal axis to extreme fiber in tension of composite section
= 3.95 in. (see Step 5)

Therefore:

M_{cr} = (530)(499)/(3.95)(12)
= 5580 lb-ft/ft

In order to crack the concrete, however, the moment must be sufficient to overcome the prestress force. From the relationship $M = f\,S_{bottom}$, where f is the compressive stress in the concrete to be overcome:

$$f = \frac{(6 \text{ strands})(14{,}630)}{(3)(48)}$$
= 610 psi

Before calculating this moment, the tensile stress imparted by the dead load must be deducted. Hence:

f = 610 − 188 − 313
= 109 psi

and the moment necessary to overcome this remaining compressive stress is:

M_{ps} = (109)(126)/(12)
= 1145 lb-ft/ft

Therefore, the total moment necessary to crack the composite section is:

$M_{cr\ tot} = M_{cr} + M_{ps} + M_{DL}$
$= 5580 + 1145 + 282 + 469$
$= 7476$ lb-ft/ft

Multiplying 1.2 times this value yields:

$(1.2)(M_{cr\ tot}) = 1.2(7476) = 8971$ lb-ft/ft

Since $\phi M_n = 15{,}912$ lb-ft/ft, calculated earlier, exceeds this value, the requirements of Article 9.18.2 are satisfied.

Step 10 — Calculate Prestress Losses

Article 9.16 describes the procedures for calculating loss of prestress due to shrinkage, elastic shortening, creep, and relaxation. The general equation for total loss is:

$$\Delta f_s = SH + ES + CR_c + CR_s \qquad (9\text{-}3)$$

where

Δf_s = total loss excluding friction in psi

SH = loss due to concrete shrinkage in psi

ES = loss due to elastic shortening in psi

CR_c = loss due to creep of concrete in psi

CR_s = loss due to relaxation of prestressing steel in psi

Article 9.16.2.1.1 defines the stress loss due to shrinkage as

$$SH = 17000 - 150RH \qquad (9\text{-}4)$$

where RH is the mean annual ambient relative humidity in percent as defined as AASHTO Fig. 9.16.2.1.1. For this example, assume RH = 70 percent; thus:

$SH = 17000 - 150\,(70)$
$= 6500$ psi

Article 9.16.2.1.2 defines the stress loss due to elastic shortening as:

$$ES = \frac{E_s}{E_{ci}}\,(f_{cir}) \qquad (9\text{-}6)$$

where

E_s = modulus of elasticity of prestressing steel strand
$= 28{,}000{,}000$ psi (assumed)

E_{ci} = modulus of elasticity of concrete at stress transfer calculated thus:

$E_{ci} = 33w^{3/2}\sqrt{f'_{ci}}$
$= 33(150)^{3/2}\sqrt{4000}$
$= 3{,}830{,}000$ psi

f_{cir} = concrete stress at center of gravity of prestressing steel due to prestressing force and dead load of member immediately after stress transfer

$$= \frac{(0.92)(0.75)(270)(6)(0.085)(1000)}{(48)(3)}$$
$= 660$ psi

Note that the concrete stress at panel center of gravity due to the dead load moment is zero because the strand is at the center of the symmetrical section.

Substituting these values into Eq. (9-6):

$$ES = \frac{28{,}000{,}000}{3{,}830{,}000}(645)$$
$= 4715$ psi

Article 9.16.2.1.3 defines the stress loss due to concrete creep as:

$$CR_c = 12 f_{cir} - 7 f_{cds} \qquad (9\text{-}9)$$

where

f_{cds} = the concrete stress at the center of gravity of the prestressing steel due to all dead loads except the dead load present at the time the prestressing force is applied

The only dead load that results in a stress other than zero at the center of gravity of the strand is the future wearing surface. Thus:

$f_{cds} = -(5 - 20)/2 = -12.5$ psi

and

$CR_c = (12)(660) - 7(12.5)$
$= 7833$ psi

Article 9.16.2.1.4 defines the stress loss due to relaxation of the low relaxation prestressing steel as:

$$CR_s = 5000 - 0.1(ES) - 0.05(SH + CR_c) \qquad (9\text{-}10A)$$

Therefore substituting the previously calculated values into Eq. (9-10A) yields:

$$CR_s = 5000 - (0.1)(4715) - 0.05(6500 + 7833)$$
$$= 3812 \text{ psi}$$

Stress Loss Summary

SH = Shrinkage	6500 psi
ES = Elastic shortening	4715 psi
CR_c = Concrete creep	7833 psi
CR_s = Strand relaxation	3812 psi
Total loss	22860 psi

Therefore, the effective stress after all losses is:

$$f_{se} = (0.75)(270{,}000) - 22{,}860$$
$$= 179{,}640 \text{ psi}$$

Step 11 — Recheck Stresses

In Step 7, 15 percent prestress loss was assumed. The resulting stresses were evaluated with the calculated prestress loss. This condition must be checked at the time of release, during placement of the cast-in-place top slab, and under final composite conditions.

At stress transfer the initial stress is $(0.92)(0.75)(f'_s)$. Note that the factor 0.92 results from the assumption that 8 percent of the jacking stress is lost through stress transfer.

$$f_{top\ precast} = f_{bottom}$$
$$= \frac{(0.92)(0.75)(270{,}000)(6 \text{ strands})(0.085)}{(3)(48)}$$
$$= 660 \text{ psi}$$

Under panel dead load:

$$f_{top\ precast} = 660 + 188 = 848 \text{ psi}$$
$$f_{bottom} = 660 - 188 = 472 \text{ psi}$$

Note that a multiplier as large as 3.5 (660/188) could be applied to stresses for dynamic effects at stripping with the section still remaining in compression.

At the time of casting of the top slab, all losses may conservatively be assumed to have occurred. Thus, the uniform stress in the deck panel is:

$$= \frac{(f_{se})(\text{strand area})(\text{no. of strands})}{\text{panel area}}$$
$$= \frac{(179{,}640)(0.085)(6)}{(48)(3)}$$
$$= 636 \text{ psi}$$

Under panel self weight, construction loads, and the weight of the top slab:

$$f_{top\ precast} = 636 + 188 + 313 + 250$$
$$= 1387 \text{ psi (1137 psi without construction load)}$$
$$f_{bottom} = 636 - 188 - 313 - 250$$
$$= -115 \text{ psi (135 psi without construction load)}$$

Under wheel load plus impact:

$$f_{top} = 495 \text{ psi (from Step 6)}$$
$$f_{top\ precast} = 1137 - 116 \text{ (from Step 6)}$$
$$= 1021 \text{ psi}$$
$$f_{bottom} = -483 \text{ (from Step 6)} + 135$$
$$= -348 \text{ psi}$$

With the future wearing surface in place:

$$f_{top} = 495 + 20 = 515 \text{ psi}$$
$$f_{top\ precast} = 1021 - 5 = 1016 \text{ psi}$$
$$f_{bottom} = -348 - 20 = -368 \text{ psi}$$

Note that all these values are within the prescribed limits defined in Step 4.

Step 12 — Distribution Reinforcing Steel in Deck Panel

Article 9.23.2 requires a minimum reinforcement transverse to the direction of the strands of 0.11 in.^{2} per ft of panel width. This may be accomplished using deformed reinforcing bars or welded wire fabric. This specified quantity of distribution reinforcement is a minimum value. Higher percentages of reinforcement may be required to resist stripping, handling, erection, or in-service stresses. If additional reinforcement is required to resist stripping, handling, or erection stresses, it should be included on the fabricator's drawings.

* * *

APPENDIX B — SAMPLE DESIGN CURVES

The design curves which follow were developed using a computer program written by the consultant. These curves are intended to give the designer an accurate initial estimate of strand requirements for a given construction configuration. These curves can be used to replace Step 7 (Strand Estimate) in Appendix A. The ability of the graphical strand estimate to satisfy AASHTO requirements must be verified by the steps outlined in Appendix A.

The curves plot jacking force in kips per foot of panel width versus simple span as determined by AASHTO Article 3.24.1. Two curves for allowable tensile stress of $6\sqrt{f'_c}$ are presented and are differentiated by the ratio *C/S*. This factor is to account for the variation in the continuous span length as determined by AASHTO based on the various girder flange widths used throughout the United States. A ratio of continuous to simple span of 1.0 and 1.1 should cover the majority of situations encountered.

The jacking force in kips per foot of width is the force necessary to apply at the time of jacking so that after all long term losses have occurred, as determined by the AASHTO Specifications, the final effective prestress force is sufficient to yield the allowable stress indicated. In order to plot these curves, the following assumptions were made:

Concrete unit weight = 150 pcf

Future wearing surface load = 35 psf

Panel transfer compressive strength = 4000 psi

Panel 28-day strength = 5000 psi

Prestress loss due to concrete shrinkage = 6500 psi (RH = 70 percent)

Construction load = 50 psf

Prestressing strand modulus of elasticity = 28,000,000 psi

Prestressing force is concentric in panels

The upper limit of the curves is typically controlled by the AASHTO code deflection control criteria specified in Article 8.9.2. These requirements are shown graphically in Fig. B1.

How to Use the Design Aids

Refer to the design example configuration in Appendix A (Fig. A1). The jacking force required to resist the loads will be determined.

The continuous span length for this configuration is 8.42 ft and the simple span length is 7.75 ft. The ratio of continuous to simple span is 1.086. As previously mentioned, separate curves have been developed for *C/S* = 1.0 and *C/S* = 1.1. Note that *C/S* represents the ratio: continuous span length to simple span length.

Fig. B2 depicts the conditions of the design example. Read the horizontal scale for simple span in feet of 7.75 ft. Move up the figure and read approximately 24.0 kips on the *C/S* = 1.0 curve and approximately 26.0 kips on the *C/S* = 1.0 curve and approximately 26.0 kips on the *C/S* = 1.1 curve. Interpolate between these values for *C/S* = 1.086 to obtain a jacking force of 25.72 kips/ft. The jacking force per foot of panel width in the design example is:

$(0.75)(270)(0.085)(6)/4 = 25.82$ kips/ft

* * *

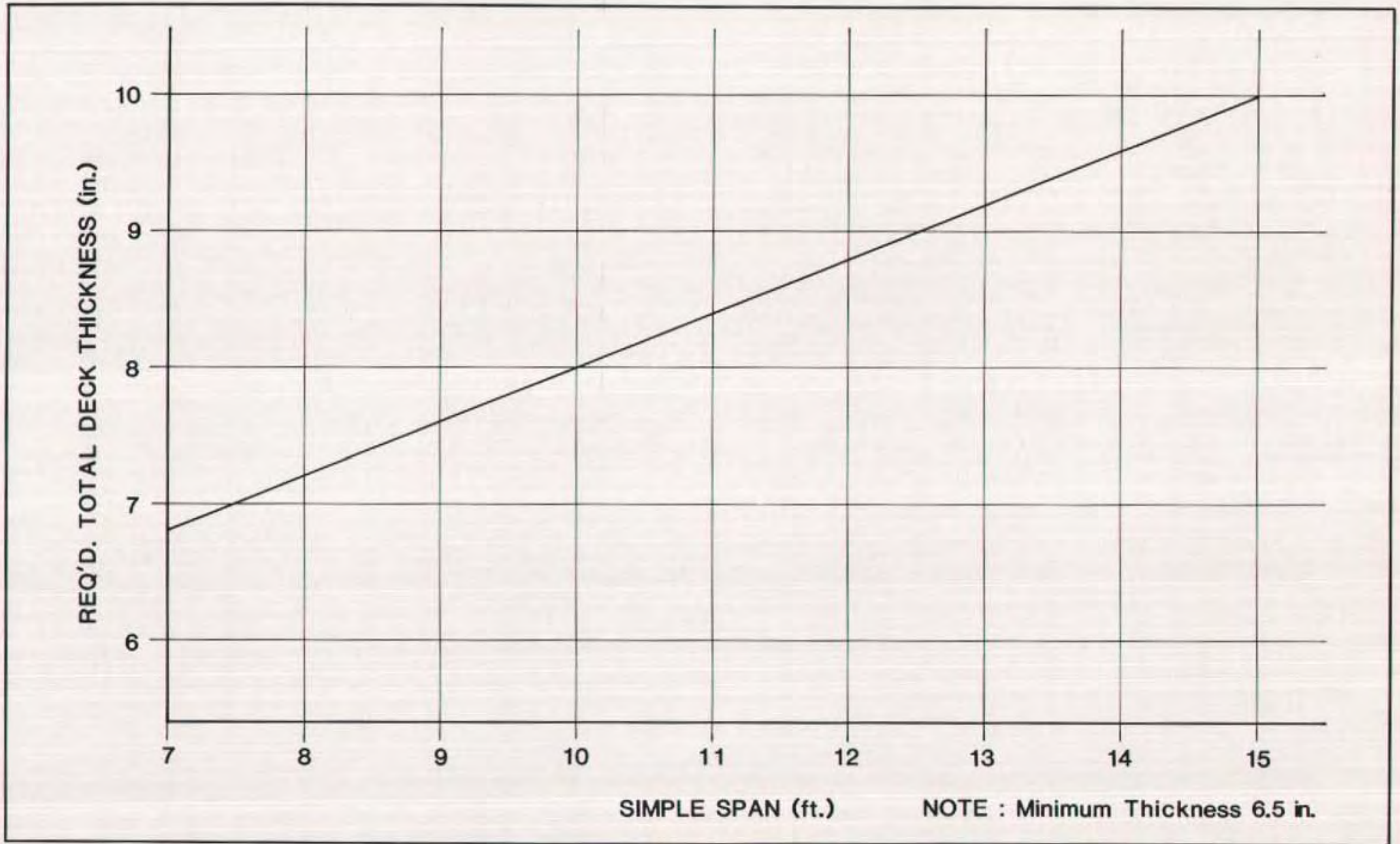

Fig. B1. AASHTO Article 8.9.2 span-depth requirements.

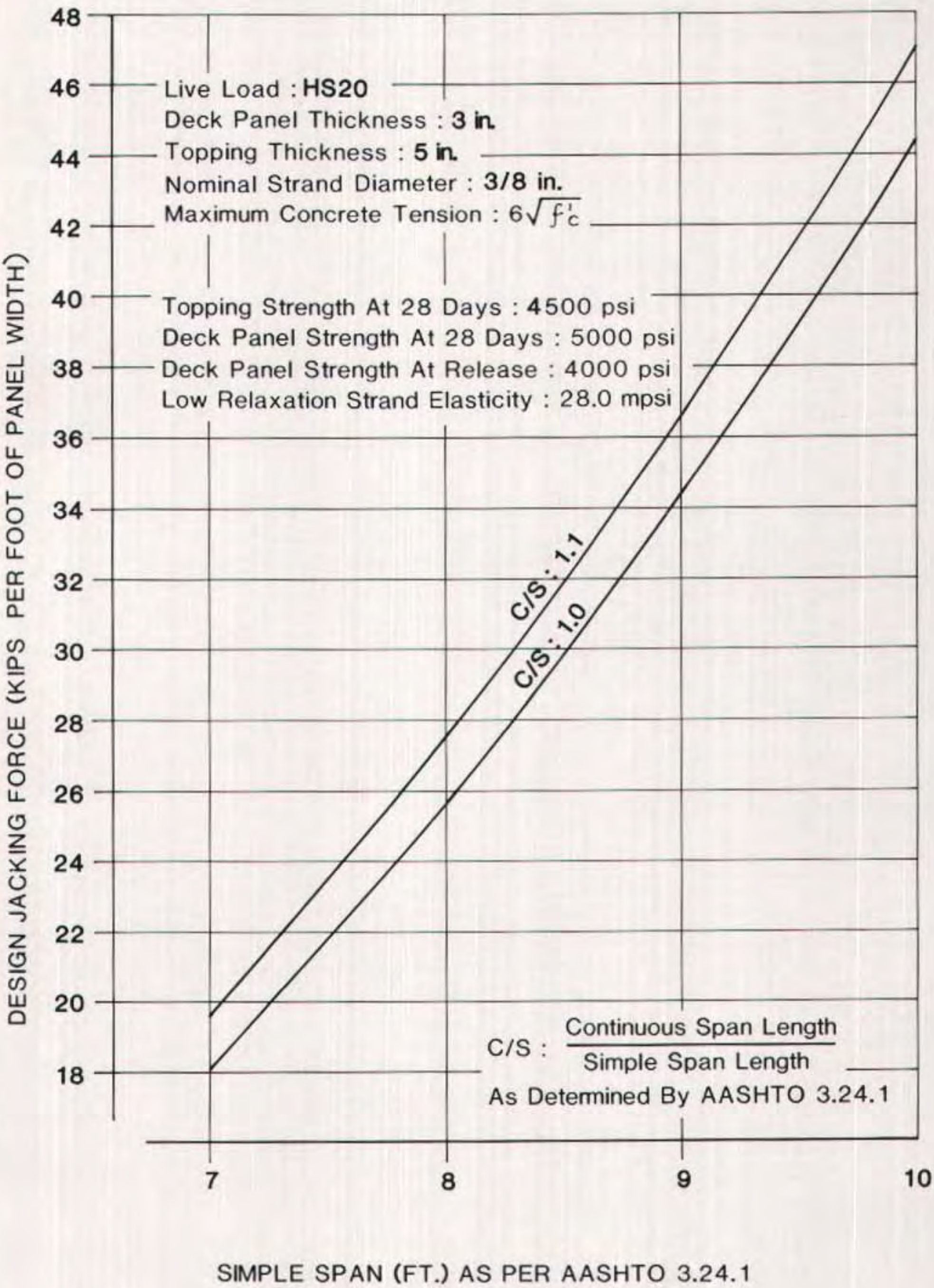

Fig. B2. Design curves — HS20 load; 3 in. deck panel; 5 in. topping.

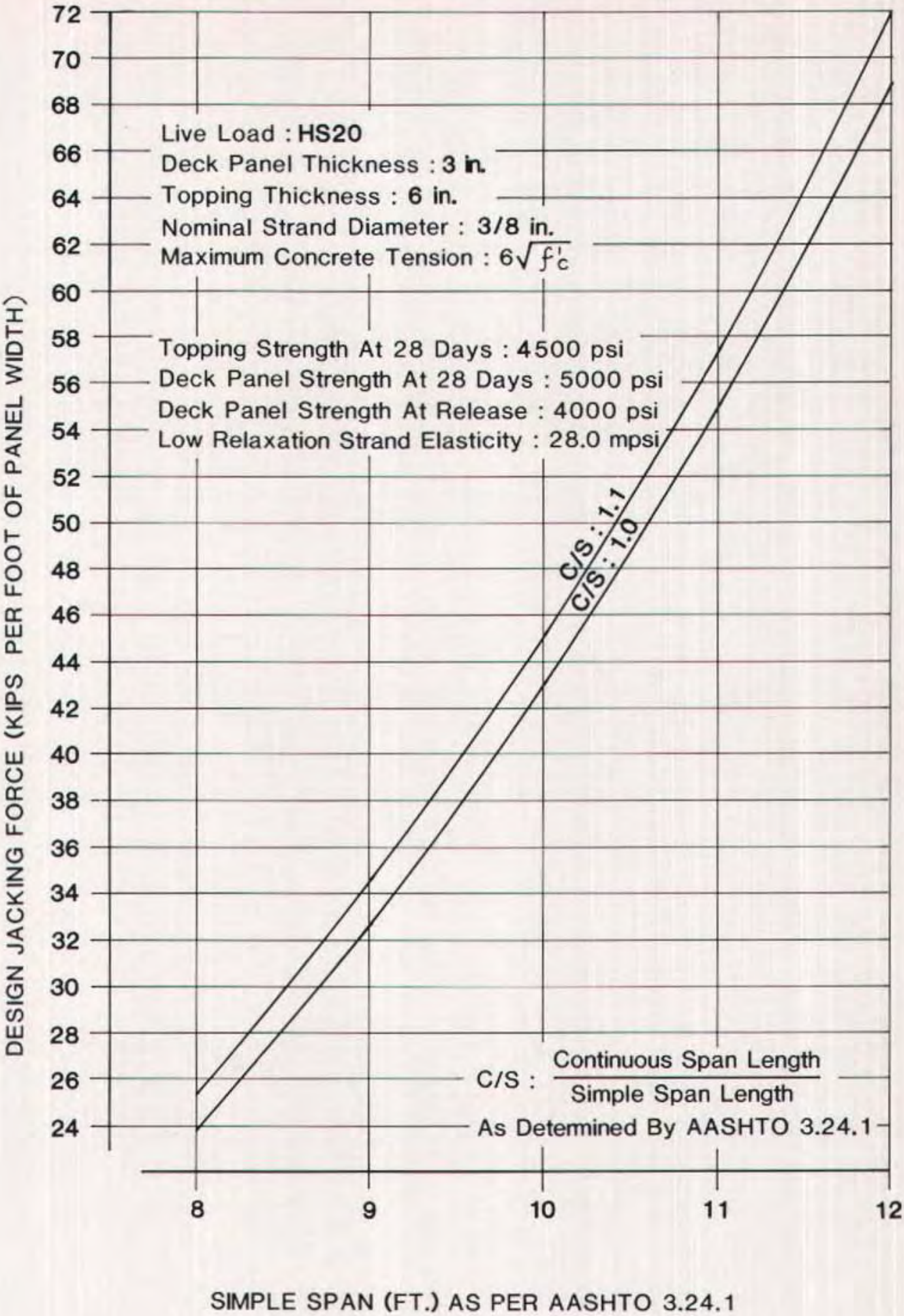

Fig. B3. Design curves — HS20 load; 3 in. deck panel; 6 in. topping.

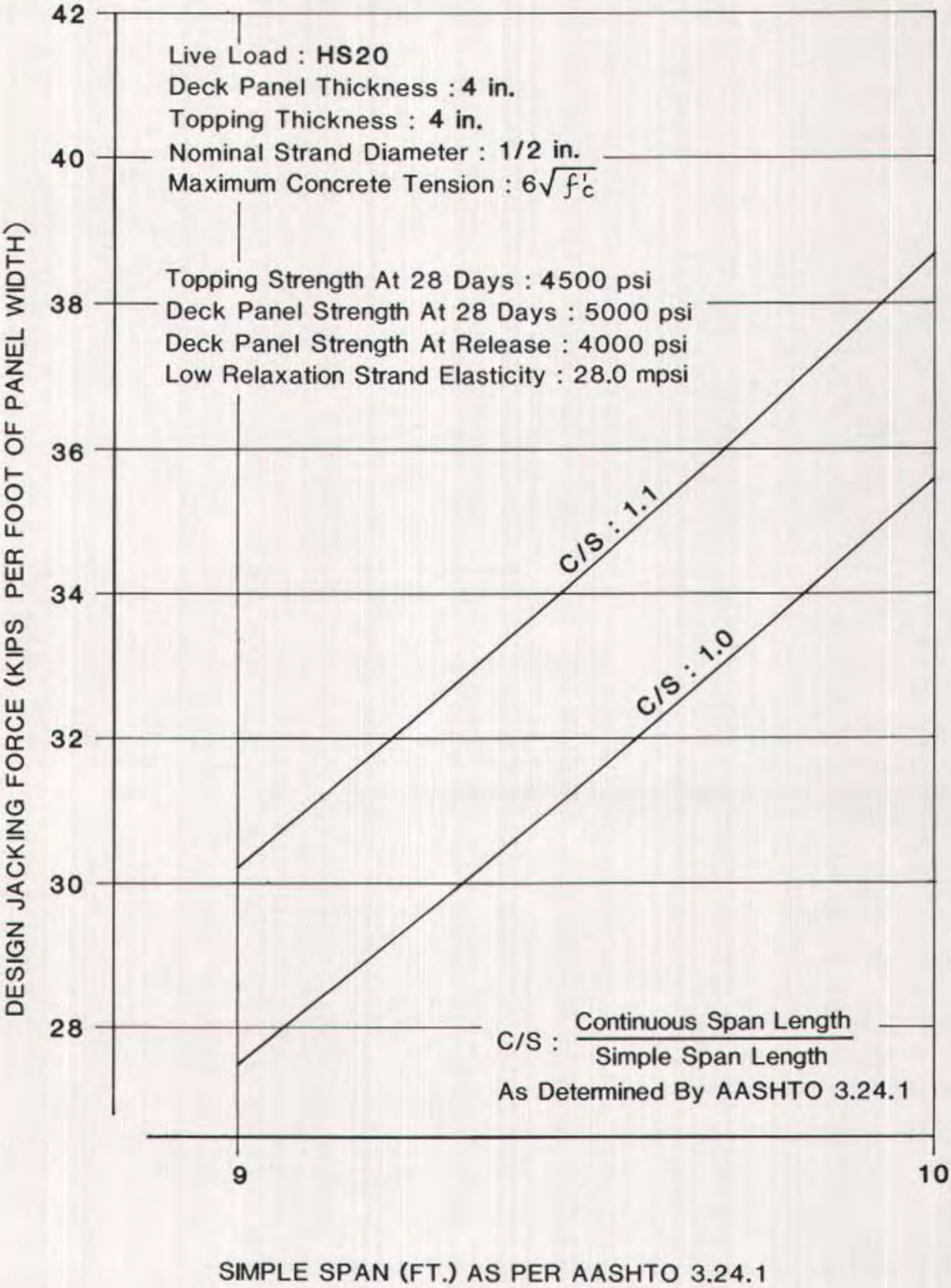

Fig. B4. Design curves — HS20 load; 4 in. deck panel; 4 in. topping.

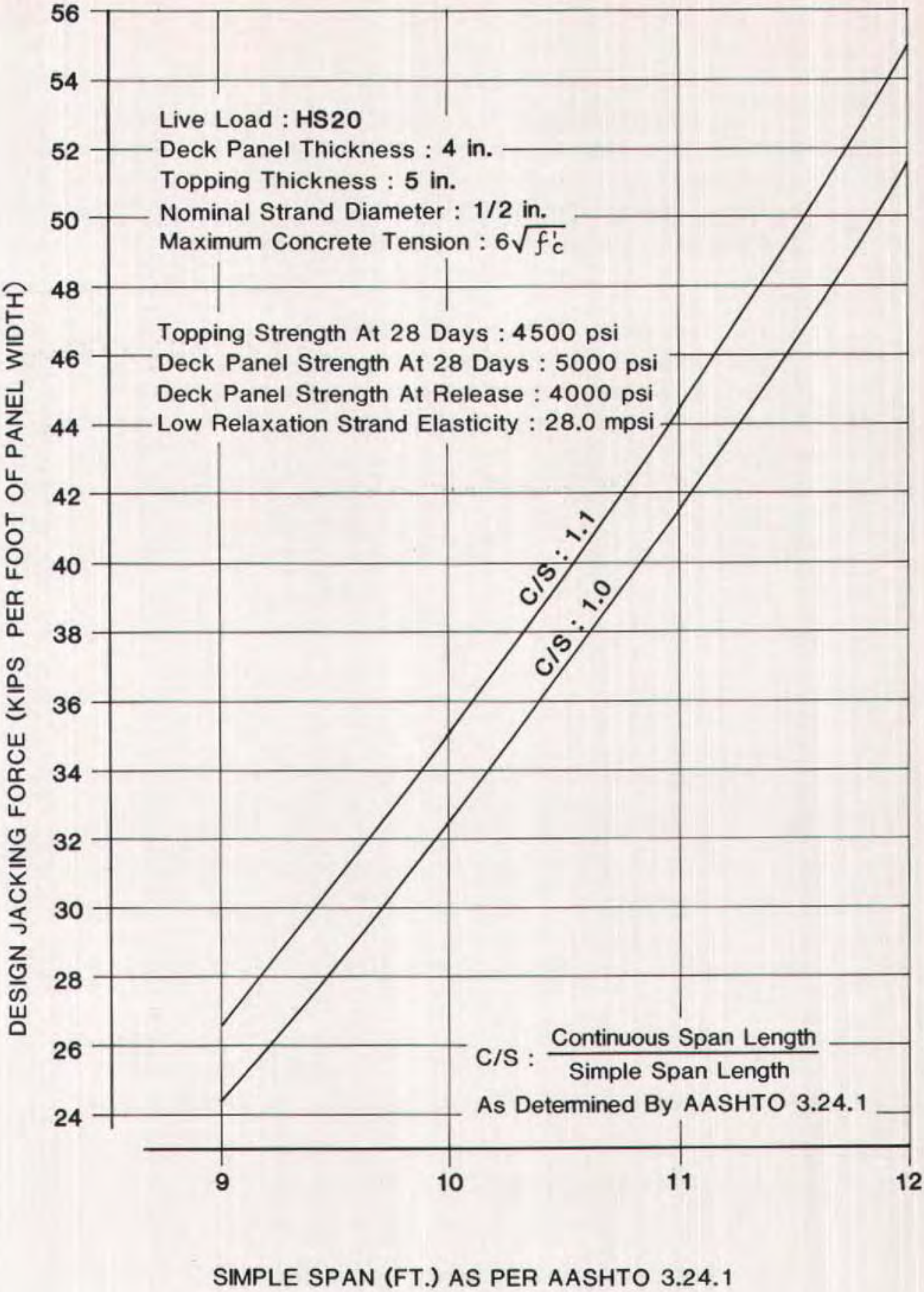

Fig. B5. Design curves — HS20 load; 4 in. deck panel; 5 in. topping.

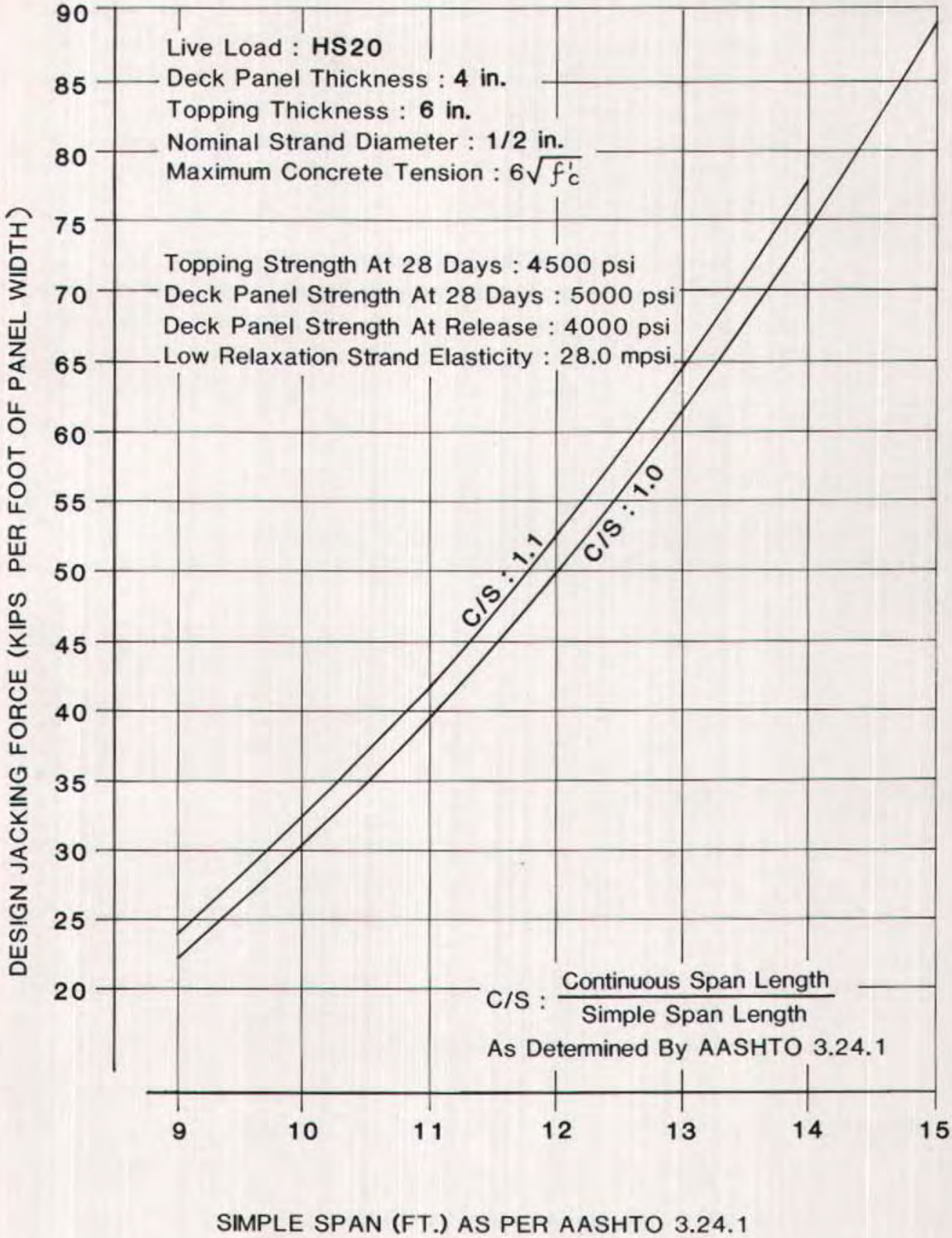

Fig. B6. Design curves — HS20 load; 4 in. deck panel; 6 in. topping.

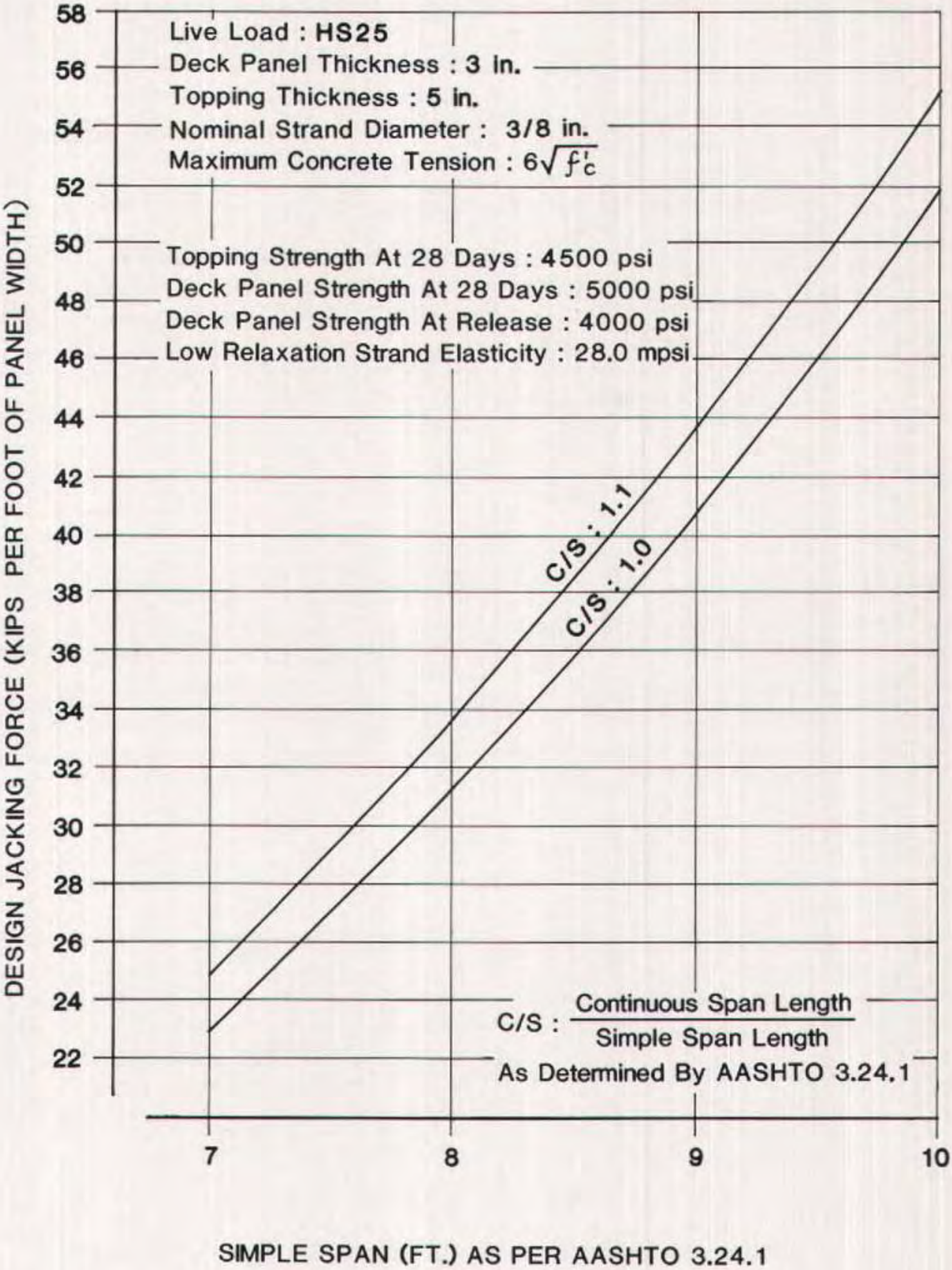

Fig. B7. Design curves — HS25 load; 3 in. deck panel; 5 in. topping.

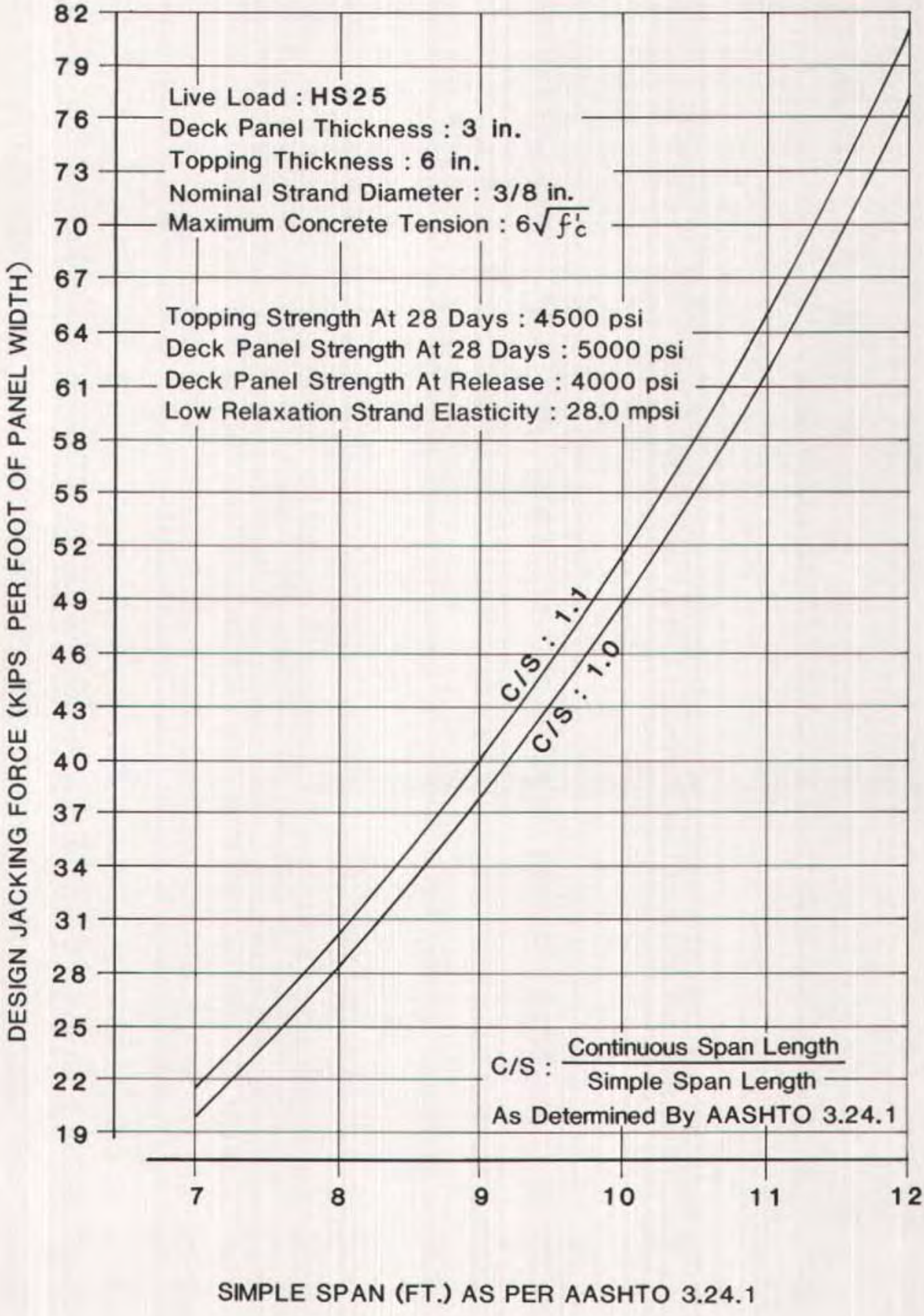

Fig. B8. Design curves — HS25 load; 3 in. deck panel; 6 in. topping.

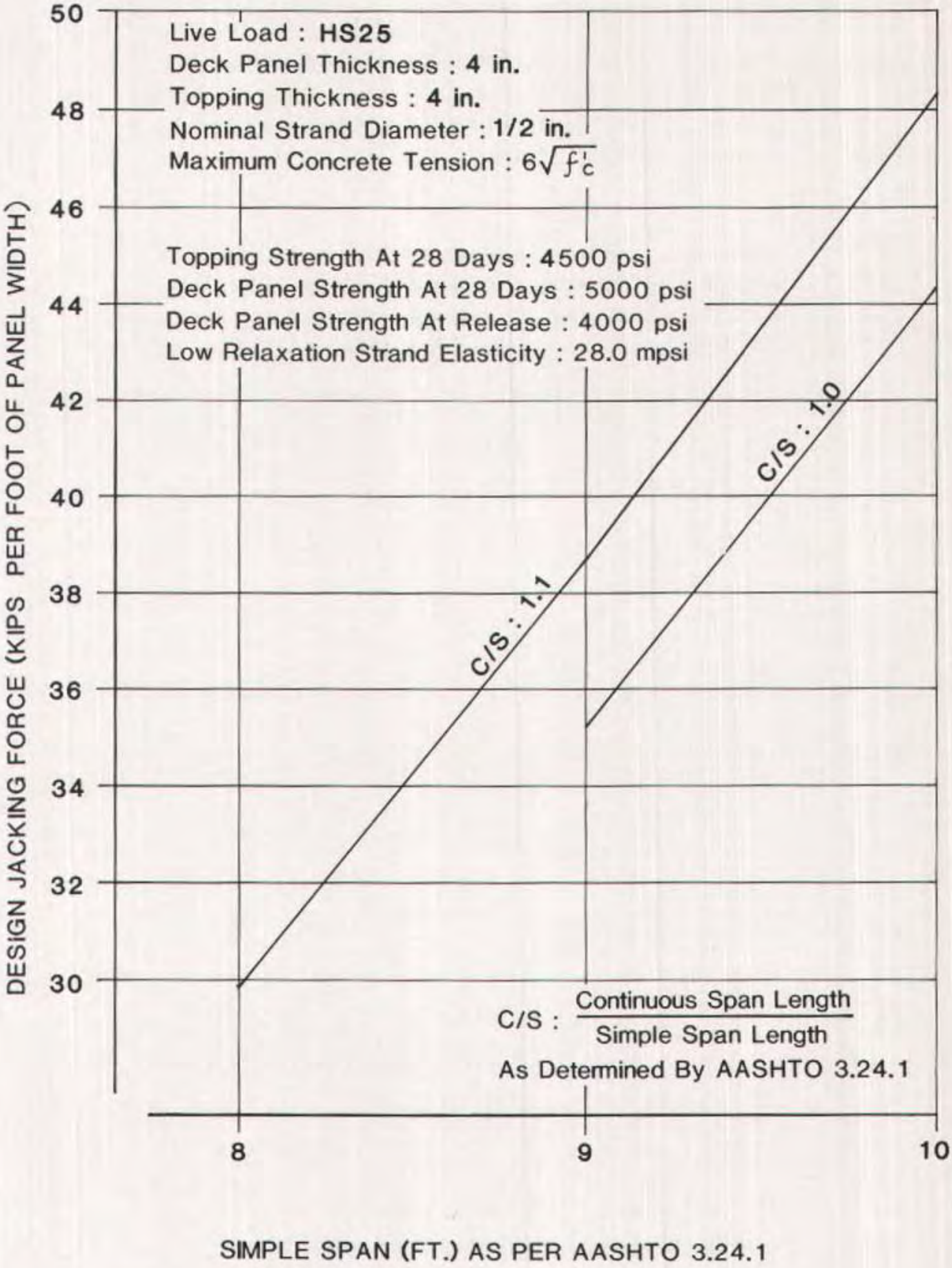

Fig. B9. Design curves — HS25 load; 4 in. deck panel; 4 in. topping.

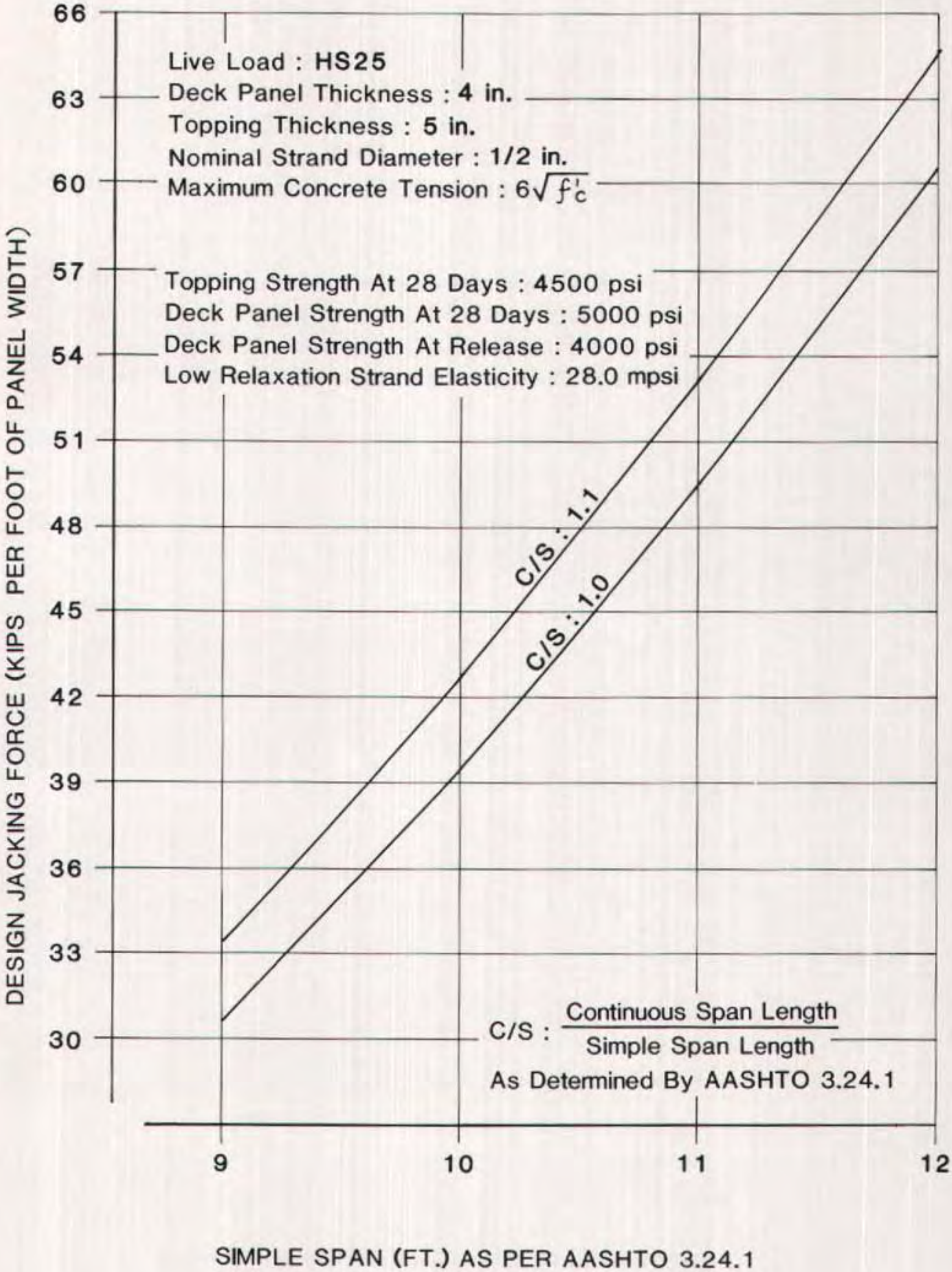

Fig. B10. Design curves — HS25 load; 4 in. deck panel; 5 in. topping.

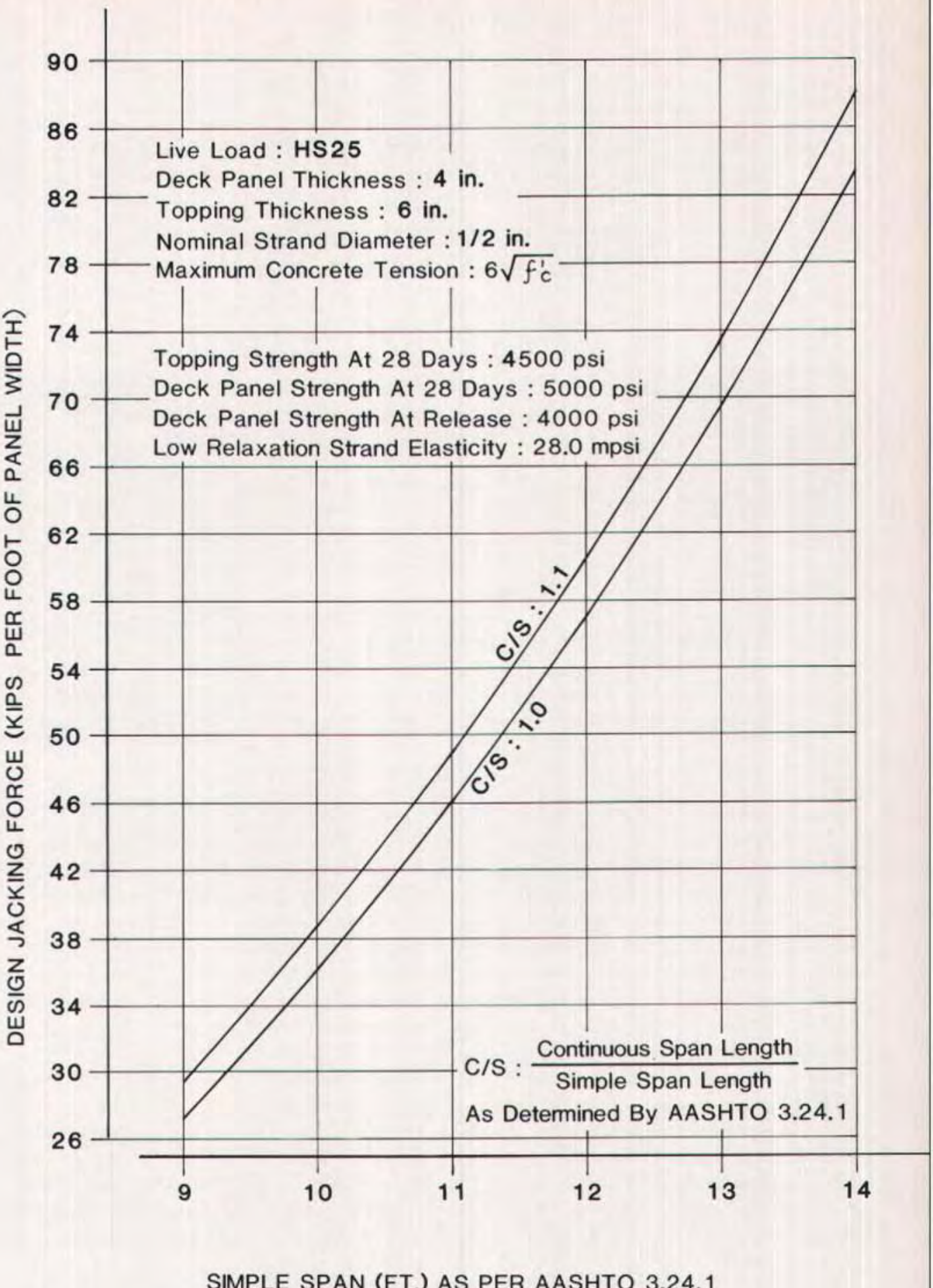

Fig. B11. Design curves — HS25 load; 4 in. deck panel; 6 in. topping.