SPECIAL REPORT

Design of Transversely Prestressed Concrete Bridge Decks

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Synopsis

Prestressing of bridge decks is a concept with potential benefits in both economy and improved durability. This paper summarizes the major design related observations and conclusions from an extensive experimental and analytical study conducted to develop criteria for design of durable prestressed bridge decks.

General provisions for design of prestressed composite girder-slab bridge decks are presented. Although the experimental research and testing was directed at cast-in-place post-tensioned decks on precast concrete girders, these provisions are valid for prestressed decks on steel girders and panelized deck systems on either concrete or steel girders.

The design provisions are presented in the form of suggested AASHTO Bridge Design Specification changes based on a synthesis of the findings from both the durability and structural phases of the study. An example showing the practical application of the proposed recommendations is included.

CONTENTS

INTRODUCTION

This paper is the last of a three-part series summarizing a study conducted at the Ferguson Structural Engineering Laboratory at The University of Texas at Austin investigating the use of prestressing as a method of improving bridge deck design. The first paper[1] summarized the results from the durability phase of the research program which emphasized the experimental investigation of post-tensioned concrete specimens subjected to aggressive deicing salt exposure. In the second paper,[2] a summary of a series of interrelated physical tests and computer analyses which were conducted to provide necessary information for development of design criteria for prestressed concrete bridge decks was presented.

The durability phase of the program[1-3] studied the effect of concrete quality and cover on corrosion, the relationship between prestressing and chloride ion penetration in cracked and uncracked concrete, and the specific detailing requirements necessary to reduce the risk of corrosion. In the experimental program of the structural phase,[2,4,5] an approximately half-scale slab and girder bridge model utilizing transverse post-tensioning was tested to determine transverse prestress distribution in the deck slab considering the effects of variables such as presence of diaphragms, lateral girder stiffness, and tendon profile as well as to investigate its behavior under vertical wheel loads. An extensive series of two- and three-dimensional linear elastic finite element computer analyses were then used to generalize the results considering the effect of slab thickness, diaphragm stiffness, and bridge skew.[2,4,6,7]

In this paper the findings from the structural and durability studies are translated into specific design recommendations and suggested AASHTO Bridge Design Specification provisions. The overall research project primarily addressed prestressing of composite cast-in-place bridge decks over multiple girders although a limited analytical study of the transverse prestressing effects in box girder bridges was also included.[4,6,7]

TRANSVERSE PRESTRESSING EFFECTS

One of the principal concerns identified at the beginning of the research study was the influence of the lateral restraint of girders on transverse prestress distribution in the deck. As shown in Fig. 1, the basic question is how much of the edge prestressing would be effective in the interior regions of the deck. The results from the finite element analysis of the slab-girder bridge without diaphragms presented in an earlier paper[2] indicate that the transverse stress distribution in a composite slab-girder bridge deck is not affected significantly by the lateral stiffness of the girders if the girders rest on flexible neoprene pads, as is the usual case.

In box-girder and in slab-girder bridges with fixed support conditions, there is a restraint problem from the girders which needs to be considered. However, for slab-girder bridges current practice is to almost exclusively use flexible neoprene bearings, with occasional use of steel rocker bearings. Both of these bearings should allow for sufficient relative girder movement during transverse prestressing. This finding suggests that the lateral stiffness effects of girders in composite slab-girder bridges will not have to be considered in design although the effect of the restraint of the webs must be considered in box-girder bridges.[4,6,7]

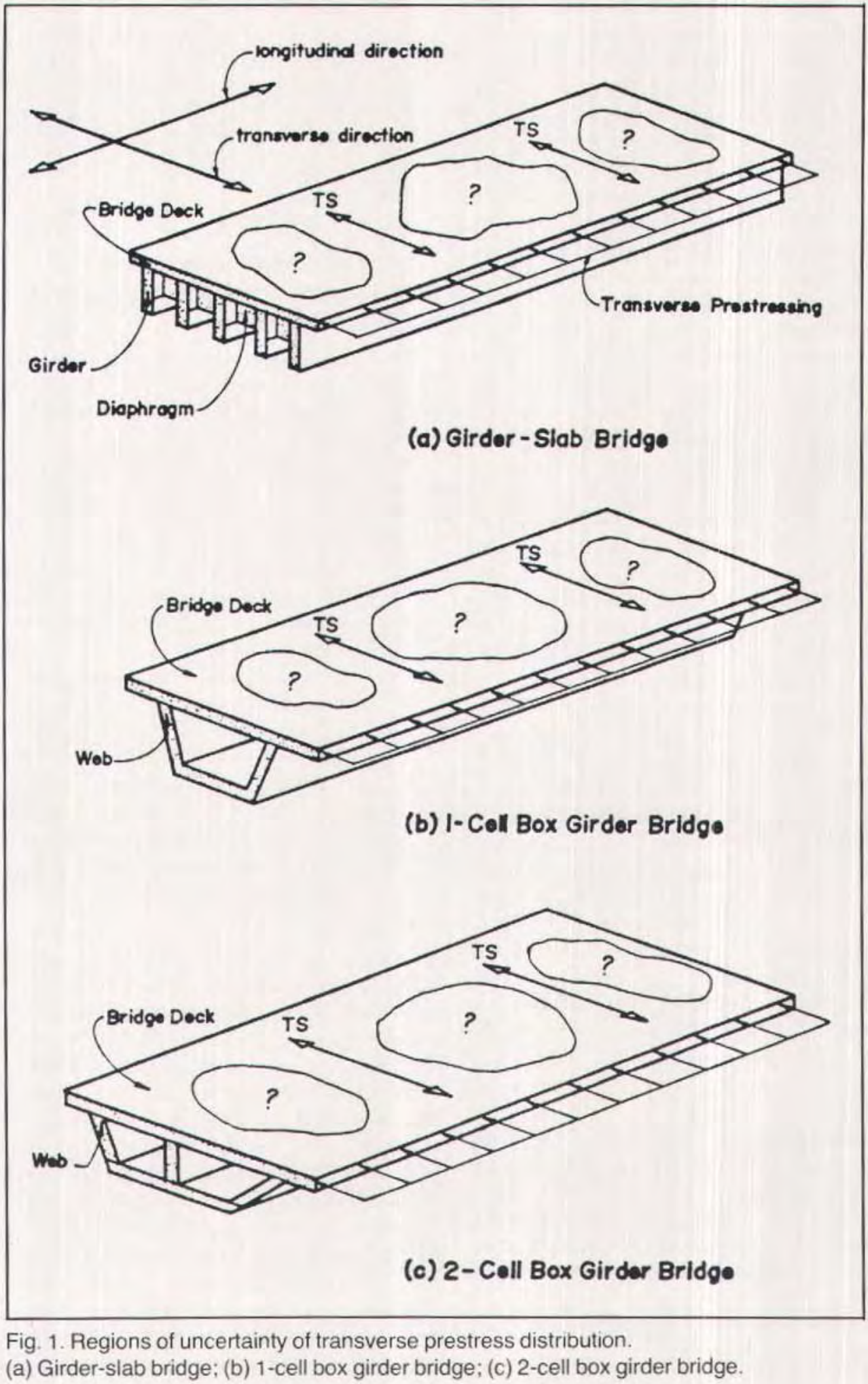

Fig. 1. Regions of uncertainty of transverse prestress distribution.
(a) Girder-slab bridge; (b) 1-cell box girder bridge; (c) 2-cell box girder bridge.

In contrast to girder restraint considerations in slab-girder bridges, the analytical and experimental results presented previously[2] clearly indicate that there are significant reductions in transverse slab prestress in both slab-girder bridges and box-girder bridges because of the presence of diaphragms. Therefore, the effect of diaphragms on the prestress distribution in a transversely prestressed bridge deck must be considered in design.

For practical design considerations there are two methods which can be used to compensate for diaphragm restraining effects. The first method involves prestressing the diaphragms with a supplementary force equal to the force attracted by them due to transverse prestressing of the deck. This would permit approximately equal shortening in the slab and diaphragms. Consequently, the deck transverse prestress distribution would be relatively unaffected by the diaphragms. Thus, the prestress force that is applied to the diaphragms to overcome the restraining effects will be some factor times the transverse prestress force applied to the slab.

The second method which can be used to compensate for diaphragm restraining effects involves amplifying the transverse prestressing in the slab by using more closely spaced tendons in regions near the diaphragms. To use this method in design, two things need to be known. They are:

1. What amplification of the prestress force is required to overcome the restraining effects; and

2. Over what area should the force be applied.

The following recommendations for the analysis of transverse prestressing restraint effects in slab-girder bridges assume that a bridge deck basically behaves compositely as an elastic slab continuous over the supporting girders.

Bridge With No Diaphragms

For a nonskew or skew bridge which will not include diaphragms, or for those cases in which the diaphragms will not be present at the time of transverse prestressing, the transverse prestress distribution for design purposes can be assumed to be equal to the applied edge prestress less appropriate friction losses and time effects.

Compensating for Diaphragm Restraining Effects by Prestressing the Diaphragms

The basic diaphragm prestress force required to compensate for the diaphragm restraining effects is given by Eq. (1):

$$P_{Db} = 1.6 F_S \qquad (1)$$

where

- P_{Db} = basic prestress force applied to the diaphragms to compensate for diaphragm restraining effects
- 1.6 = factor to account for presence of diaphragms (unit of length is ft) (factor would be 0.49 if metric unit is meter)
- F_S = transverse slab prestress force per unit edge length required to resist effects of structural loads assuming no diaphragm restraining effects

To illustrate Eq. (1), if the design transverse prestress is 200 psi (1.38 MPa) in an 8 in. (20.3 cm) slab, then the transverse slab prestress force per unit edge length, F_S, would be 19,200 lbs per ft (280 kN/m), and the diaphragm force required to compensate for restraining effects would be 1.6 times 19,200 lbs (85.4 kN), which is about 30,700 lbs (137 kN). This basic equation [Eq. (1)] is applicable for both end and interior diaphragms. For this basic equation, the bridge slab thickness is assumed to be 8 in. (20.3 cm), the bridge skew 0 degrees, the diaphragm spacing 25 ft (7.6 m), and the diaphragm stiffness corresponds to that of standard concrete diaphragms

Table 1. Comparison between diaphragm prestress force required to compensate for diaphragm restraining effects determined by proposed basic equation and that computed by finite element analysis.

		P_D/F_S			
		End diaphragm		Interior diaphragm	
Strand profile	Diaphragm case	Finite element analysis	Proposed [Eq. (1)]	Finite element analysis	Proposed [Eq. (1)]
Straight	All	1.4	1.6	1.55	1.6
Straight	End only	1.4	1.6	—	—
Draped	All	1.55	1.6	1.75	1.6
Draped	End only	1.55	1.6	—	—

Assumptions for comparison:
Distance between interior diaphragms = 25 ft (7.6m).
Slab thickness = 8 in. (20.3 cm).
Standard concrete diaphragms: A = 160 in.^{2} (1032 cm^2).
Skew angle = 0 degrees.

[approximately 160 in.^{2} (1032 cm^2)]. Modifications to this basic equation will be required as suggested below for slab thickness, diaphragm stiffness, bridge length, and bridge skew.

Table 1 presents comparisons between the diaphragm prestress force required to compensate for diaphragm effects determined by Eq. (1) and that determined by a finite element analysis. The comparisons are presented in terms of the ratio of P_D to F_S. In general, the constant value of 1.6 is a reasonably conservative assessment of the values determined by finite element analysis. While the study basically considered diaphragms in skewed bridges as having a squared off arrangement, the recommendations made should be conservative for structures using skewed intermediate diaphragms.

Correction for Slab Thickness — As the slab thickness decreases, the relative restraint due to the diaphragms increases and hence the diaphragm force increases. The basic equation [Eq. (1)] is modified for the effect of slab thickness, as:

$$P_D = C_t\, 1.6 F_S \qquad (2)$$

where C_t is the correction factor for slab thickness.

The proposed slab thickness correction factor is:

$$C_t = 8/t \qquad (3)$$

where t is the slab thickness, in. (1 in. = 2.54 cm).

Table 2 presents a comparison between the diaphragm prestress force required to overcome diaphragm restraining effects predicted by Eq. (2) and that computed by finite element analysis for varying slab thickness. The comparisons are in terms of the ratio of P_D to F_S. The proposed slab thickness modification results in very reasonable and generally conservative estimates of the required diaphragm prestress force in all cases. Exceptionally good agreement exists for the interior diaphragm cases.

Correction for Diaphragm Stiffness — Current trends in bridge construction indicate that fewer diaphragms are being used, especially in the interior regions of bridges. If diaphragms are used, current practice calls for standard concrete diaphragms with an area of about

Table 2. Comparison between diaphragm prestress force required to compensate for diaphragm restraining effects determined by proposed basic equation modified for slab thickness and that computed by finite element analysis.

Slab thick-ness (in.)	Diaphragm case	P_D/F_S			
		End diaphragm		Interior diaphragm	
		Finite element analysis	Pro-posed [Eq. (2)]	Finite element analysis	Pro-posed [Eq. (2)]
6	All	1.9	2.1	2.1	2.1
6	End only	1.8	2.1	—	—
8	All	1.4	1.6	1.55	1.6
8	End only	1.4	1.6	—	—
10	All	1.1	1.3	1.25	1.3
10	End only	1.1	1.3	—	—

Assumptions for comparison:
Distance between interior diaphragms = 25 ft (7.6 m)
Standard concrete diaphragms: $A = 160$ in.^{2} (1032 cm^2).
Skew angle = 0 degrees.
Straight strand profile.

Table 3. Comparison between diaphragm prestress force required to compensate for diaphragm restraining effects determined by proposed basic equation modified for diaphragm stiffness and that computed by finite element analysis.

Cross-sectional diaphragm stiffness (EA) (k-in.^{2}/in.^{2})	Strand profile	Diaphragm case	P_D/F_S			
			End diaphragm		Interior diaphragm	
			Finite element analysis	Proposed [Eq. (4)]	Finite element analysis	Proposed [Eq. (4)]
320,000	Straight	All	0.8	0.8	0.8	0.8
	Straight	End only	0.8	0.8	—	—
	Draped	All	1.0	0.9	1.0	0.8
	Draped	End only	1.0	0.8	—	—
640,000* (Standard concrete diaphragms; $A = 160$ in.^{2})	Straight	All	1.4	1.6	1.55	1.6
	Straight	End only	1.4	1.6	—	—
	Draped	All	1.55	1.6	1.75	1.6
	Draped	End only	1.55	1.6	—	—
960,000	Straight	All	2.2	2.4	1.7	2.4
	Straight	End only	2.2	2.4	—	—
	Draped	All	2.0	2.4	2.4	2.4
	Draped	End only	2.2	2.4	—	—

Assumptions for comparison:
Distance between interior diaphragms = 25 ft (7.6 m).
Slab thickness = 8 in. (20.3 cm).
Skew angle = 0 degrees.
*Concrete modulus assumed = 4000 ksi (27940 MPa).

Metric conversion factor:
1 kip 4.45 kN.
(1 kip-in.^{2}/in.^{2} = 4.45 kN-cm^2/cm^2).

Table 4. Comparison between diaphragm prestress force required to compensate for diaphragm restraining effects determined by proposed basic equation modified for spacing between interior diaphragms and that computed by finite element analysis.

Spacing between interior diaphragms (ft)	P_D/F_S*			
	End diaphragm		Interior diaphragm	
	Finite element analysis	Proposed [Eq. (6)]	Finite element analysis	Proposed [Eq. (6)]
18	1.9	2.2	2.4	2.2
21	1.7	1.9	2.0	1.9
25	1.4	1.6	1.55	1.6
28	1.25	1.4	1.4	1.4

*Applicable only for bridges with interior diaphragms.

Assumptions for comparison:

Slab thickness = 8 in. (20.3 cm).

Standard concrete diaphragms; A = 160 in.² (1032 cm²).

Skew angle = 0 degrees.

Straight strand profile.

160 in.² (1032 cm²). For this case, the basic equation [Eq. (1)] does not need modification. However, if nonstandard concrete diaphragms or if steel diaphragms similar to those used for steel girder bridges are called for in design, the following modification to the basic ratio is proposed:

$$P_D = C_K\, 1.6 F_S \quad (4)$$

where C_K is the correction factor for diaphragm stiffness.

The correction factor for diaphragm stiffness is defined as follows:

$$C_K = (EA)_D / 640{,}000 \quad (5)$$

where

E = modulus of elasticity of the diaphragm materials, ksi (1 ksi = 6.894 MPa), and

A = effective diaphragm cross-sectional area resisting axial deformations, in.² (1 in.² = 6.45 cm²)

The term $(EA)_D$ represents the effective cross-sectional diaphragm axial stiffness.

Table 3 presents a comparison between the diaphragm prestress force required to overcome diaphragm restraining effects predicted by Eq. (4) and that predicted by finite element analysis for varying diaphragm stiffness. The comparisons are again based on ratios of P_D to F_S. The proposed modification for diaphragm stiffness roughly approximates that obtained by finite element analysis and is generally conservative.

Correction for Interior Diaphragm Spacing — The number of interior diaphragm locations varies with bridge length. Current practice indicates that for bridge lengths up to 55 ft (16.8 m), one line of interior diaphragms at midspan is used. From 55 to 95 ft (16.8 to 29.0 m), two lines of interior diaphragms at third points are used. For bridge lengths greater than 95 ft (29.0 m), three diaphragm lines at quarter points are used. Thus, the spacing between diaphragms in bridges which include interior diaphragms varies from about 18 to 32 ft (5.5 to 9.8 m). As the distance between interior diaphragms decreases, the restraining force in both end and interior diaphragms increases and hence the force required to overcome diaphragm restraining effects increases. To account for the interior diaphragm spacing effect (i.e., bridge length effect), the following equation is proposed:

$$P_D = C_L\, 1.6 F_S \qquad (6)$$

where C_L is the correction factor for the stiffness effect due to interior diaphragm spacing.

If no interior diaphragms are used, this correction is not required. To determine C_L, the following equation is proposed:

$$C_L = 25/S_D \qquad (7)$$

where S_D is the spacing between interior diaphragms or between end and interior diaphragms, ft (1 ft = 0.305 m).

Table 4 presents a comparison between the proposed diaphragm prestress force modified for spacing between interior diaphragms and that computed by finite element analysis. Again, the comparisons are made in terms of the ratio of P_D to F_S. In general, the values calculated by Eq. (6) are reasonably close to those determined by finite element analysis.

Correction for Bridge Skew Angle — The results from finite element analyses indicate that as the skew angle of the bridge increases from zero degrees, the restraining force in the diaphragm due to transverse prestressing decreases. This implies that the diaphragm prestress force required to overcome restraining effects also decreases. It would be conservative to ignore any decrease in diaphragm force for bridges with skew. However, detailing considerations suggest that prestressing diaphragms on a skew bridge is probably not practical. Thus, the factor (C_{SK}) to correct the basic equation for effects of skew will be taken as 1.0 in the proposed design recommendations and no other numerical table is required.

Multiple Corrections — It is proposed that the correction factors be multiplied as illustrated by Eq. (8) for multiple corrections to the basic equation:

$$P_D = C_t C_K C_L C_{SK}\, 1.6\ F_S \qquad (8)$$

A parametric study with a mix of variables revealed that Eq. (8) could be as much as 20 percent unconservative if more than two of the correction factors used had a value less than 1. Therefore in using Eq. (8) no more than two correction factors with values less than one can be used. However, the two lowest correction factors may be used. Table 5 compares the results obtained for Eq. (8) and those obtained by finite element analysis for several cases with a mix of variables. It appears that the proposed simplified procedure for determining the diaphragm prestress force required to overcome the restraining effects should produce reasonable yet conservative results.

The development of the equations utilized in this method assumes that the prestress force was applied at the centroid of the diaphragm cross section. In practice, this may not always be possible. If the height of the diaphragm is small compared to the total height of the bridge superstructure, the exact location of the diaphragm tendon may not affect the stresses in the slab significantly. The opposite is true, however, when the diaphragm height is nearly equal to that of the superstructure. Regardless of the height of the diaphragm, the prestressing force should never be located such that it may induce tension stress in the diaphragm.

Taking into consideration the above constraints, a reasonable allowable eccentricity of the diaphragm prestress force is ½ the distance to the kern point of the diaphragm, or 1/12 the height of the diaphragm. If the prestress eccentricity in the diaphragms exceeds this amount, a more detailed analysis of the effect on the stresses in the bridge deck should be carried out.

Table 5. Comparison between diaphragm prestress force calculated by Eq. (8) and that determined by finite element analysis for a mix of variables.

Case	Bridge variables	Correction factors	Proposed [Eq. (8)]	Finite element analysis
1	Bridge length = 75 ft Diaphragm spacing = 25 ft θ = 40 degrees Slab thickness = 10 in. $(EA)_D$ = 320,000 kip-in.²/in.²	$C_L = 1$ $^*C_{SK} = 0.77$ $C_t = 0.8$ $C_K = 0.5$	(0.77)(0.5)1.6 = 0.62 Limit imposed that only two correction factors can be taken less than 1	0.59
2	Bridge length = 60 ft Diaphragm spacing = 20 ft θ = 0 degrees Slab thickness = 6.5 in. $(EA)_D$ = 960,000 kip-in.²/in.²	$C_L = 1.25$ $C_t = 1.23$ $^*C_{SK} = 1$ $C_K = 1.5$	(1.25)(1.23)(1.5)(1.6) = 3.69	2.49
3	Bridge length = 76 ft θ = 20 degrees Slab thickness = 7 in. $(EA)_D$ = 640,000 kip-in.²/in.²	$C_L = 1$ $^*C_{SK} = 0.94$ $C_t = 1.14$ $C_K = 1$	(0.94)(1.14)(1.6) = 1.71	1.71
4	Bridge length = 76 ft Diaphragm spacing = 25 ft θ = 0 degrees Slab thickness = 9 in. $(EA)_D$ = 640,000 kip-in.²/in.²	$C_L = 1$ $C_K = 1$ $C_t = 0.89$ $^*C_{SK} = 1$	(0.89)(1.6) = 1.42	1.33

$^*C_{SK}$ — Correction for bridge skew.
Not included in actual design recommendations.
Metric conversion factors: 1 ft = 0.305 m; 1 in. = 2.54 cm; 1 kip = 4.45 kN.

Compensating for Diaphragm Restraining Effects by Applying Extra Prestressing in the Slab in Regions Near the Diaphragms

The results from the laboratory model bridge tests revealed that applying extra prestressing in the form of more closely spaced tendons in a 4 ft (1.2 m) region around the diaphragms was a viable and expeditious method to overcome the restraining effects of the diaphragms.[2,3] In the case of the model bridge tested, the tendon spacing was conservatively cut in half from that used in nondiaphragm slab regions. This resulted in twice the prestressing force per unit edge length in the diaphragm regions as compared to nondiaphragm regions. However, the results from the experimental tests as well as from the finite element studies revealed that a somewhat lower value of prestressing force in diaphragm regions would have been adequate.

For design, two equations are proposed for determining the amplified prestress force required in diaphragm regions. For 0 to 10 degree skew bridges, Eq. (9) is proposed:

$$F_e = 1.6\ F_S \tag{9}$$

where

F_e = amplified transverse slab prestress force per unit edge length applied in regions near diaphragms in order to compensate for diaphragm restraining effects

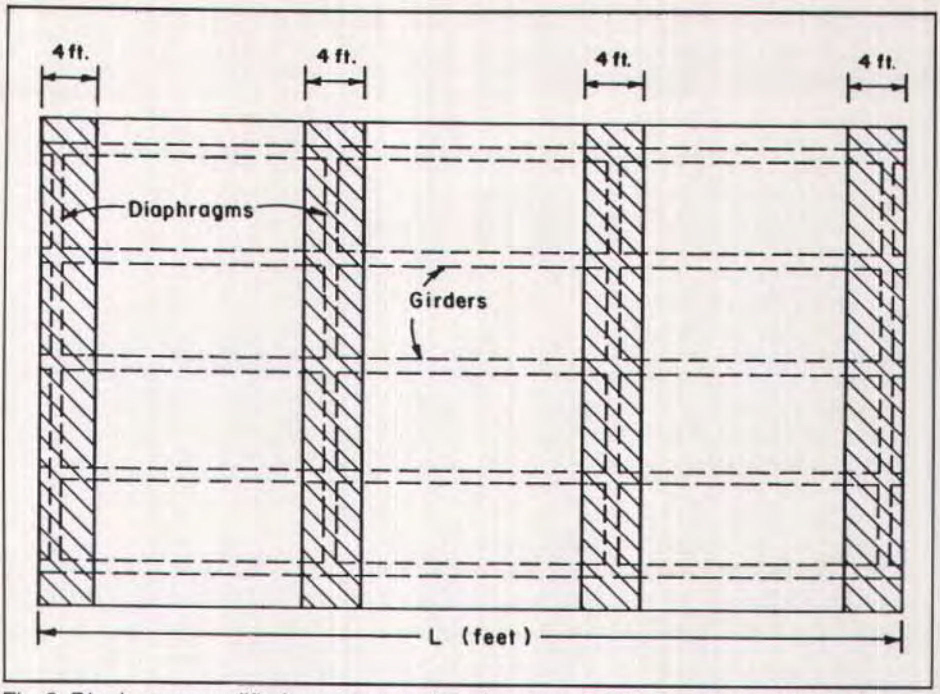

Fig. 2. Diaphragm amplified prestress regions for a nonskew bridge (1 ft = 0.305 m).

F_S = tranverse slab prestress force per unit edge length required to resist effects of structural loads assuming no diaphragm restraining effects

This amplified prestressing would be applied over an edge length of 4 ft (1.2 m) centered on the diaphragms. For bridges with greater than 10 degree skew, Eq. (10) is proposed:

$$F_e = 1.2 F_S \qquad (10)$$

Thus, for bridges with greater than 10 degree skew, less amplified prestress force per unit edge length would be required; however, it will need to be applied over a wider region of the slab than the 4 ft (1.2 m) edge strip used with nonskew bridges.

The slab edge length over which this amplified prestressing force must be applied is given by Eq. (11):

$$x = W \tan\theta + 4 \leq (L + W \tan\theta)/N \qquad (11)$$

where

x = slab edge length at diaphragms over which F_e will be required, ft (1 ft = 0.305 m)

W = width of bridge slab, ft (1 ft = 0.305 m)

θ = bridge skew angle as measured between the transverse edge of the deck slab and the normal to the longitudinal edge of the deck slab, degrees (see Fig. 3)

L = span length, ft (1 ft = 0.305 m)

N = number of diaphragm lines per span (i.e., four for a span with two sets of interior diaphragms, and two for a span with only end diaphragms)

The limit of $(L + W \tan\theta)/N$ is imposed to ensure that the diaphragm regions do not overlap. This implies that for some skew bridges, the amplified

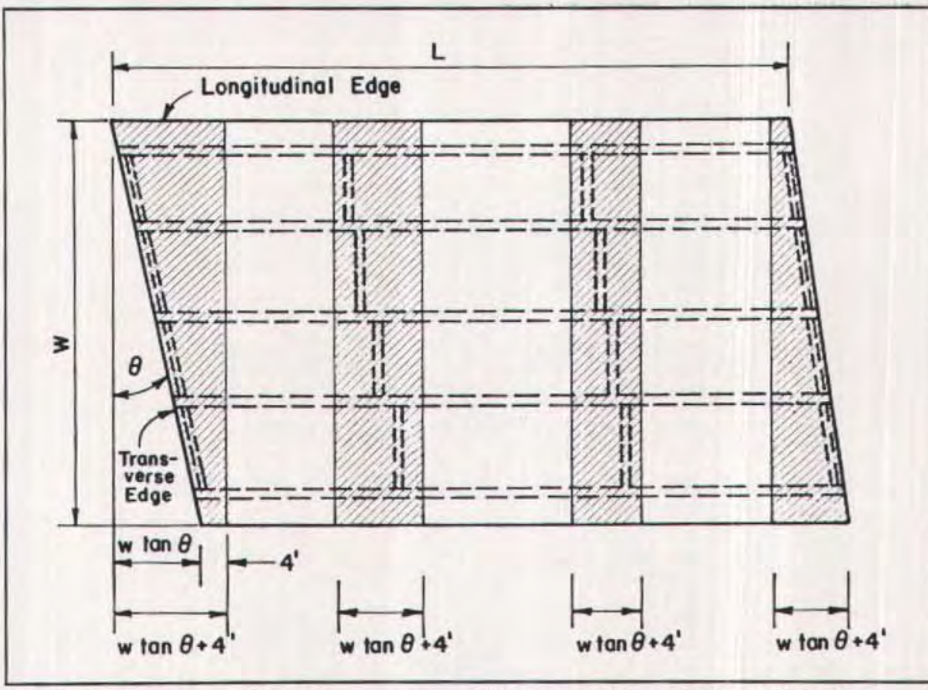

Fig. 3. Diaphragm amplified prestress regions for a skew bridge (1 ft = 0.305 m).

prestress force F_e may be required along the entire edge length of the bridge. For a bridge with no skew, the diaphragm amplified prestress region is 4 ft (1.2 m) wide, which was the equivalent distance used for the laboratory bridge model.[2,3] Figs. 2 and 3 show in shading the diaphragm amplified prestress regions for a bridge with no skew and with skew, respectively.

To examine the applicability of Eqs. (9) through (11), finite element analyses were used to examine the effects of the recommended prestress distributions of the prototype bridge of Ref. 2 for skew angles varying from 0 to 60 degrees. Figs. 4 and 5 present typical stress contours from the analysis for the cases of bridge skew of 10 and 40 degrees, respectively. The contours represent percentages of the stress induced along the slab edge by F_S. Ideally, it would be desirable to have a uniform stress distribution in the slab with all stress equal to the stress induced by F_S.

This is clearly not possible in practice. However, in all cases studied, the results indicate that a substantial portion of the deck area is between 95 and 120 percent, which suggests a reasonably uniform prestress distribution. There are a few "hot" spots up to 150 percent but this is not a problem with the low levels of slab prestress usually used. Thus, the use of Eqs. (9) through (11) resulted in reasonable, yet generally conservative slab prestress distributions for the wide range of skew angles examined for the study bridge. The prestress distributions which result from the use of these equations should be reasonably uniform and generally conservative.

Prestress Losses

Prestress losses such as friction losses

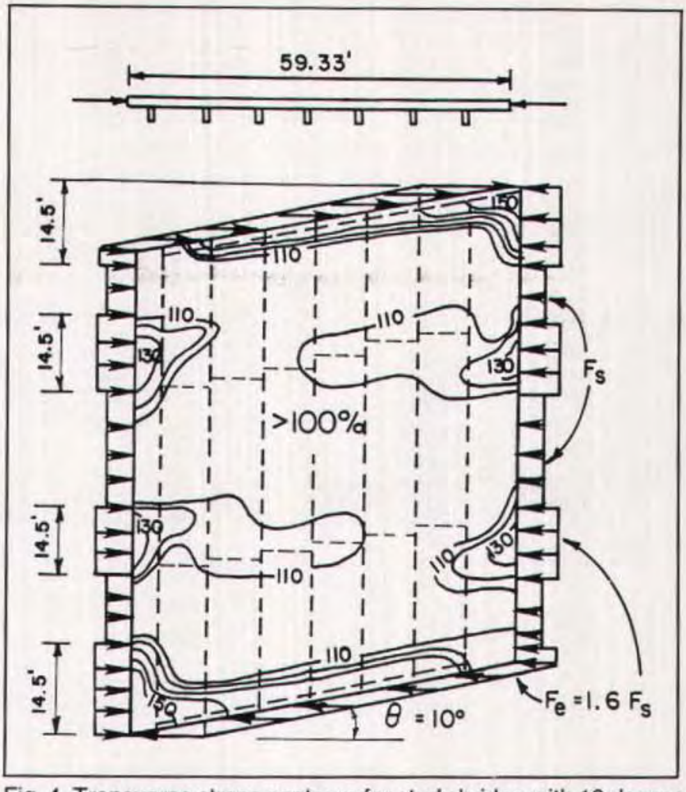

Fig. 4. Transverse stress contours for study bridge with 10 degree skew; amplified prestressing force applied in diaphragm regions; F_s produces edge stress = 100 percent (1 ft = 0.305 m).

in post-tensioning result in less compression to resist imposed loads and must be considered in design. Ralls[5] reported a tendon force loss due to friction of 30 percent for a post-tensioning system consisting of closely draped tendons in a full-depth test slab simulating draping for continuity over long longitudinal girders. This reduction is on the same order as that produced by the restraining effect of diaphragms. However, no additional rules are required since the loss of prestress is adequately covered in the current AASHTO Specifications,[8] Section 9.16.

Secondary Moment Effects

Draped tendons or any unsymmetrical placement of prestressing about the centroid of a bridge deck results in secondary moments in continuous transverse bridge slabs which are vertically restrained. The effect is to increase slab stresses due to prestressing at some locations and decrease these stresses at others. Thus, there is less effective compression at the locations where the stresses decrease due to secondary moments. In general, this secondary moment effect will probably not be very significant for thin transversely prestressed bridge decks. Draped tendons are probably not cost effective since only small eccentricities are possible within thin slabs. However, for those cases in which secondary moments can exist, the effects can be considered

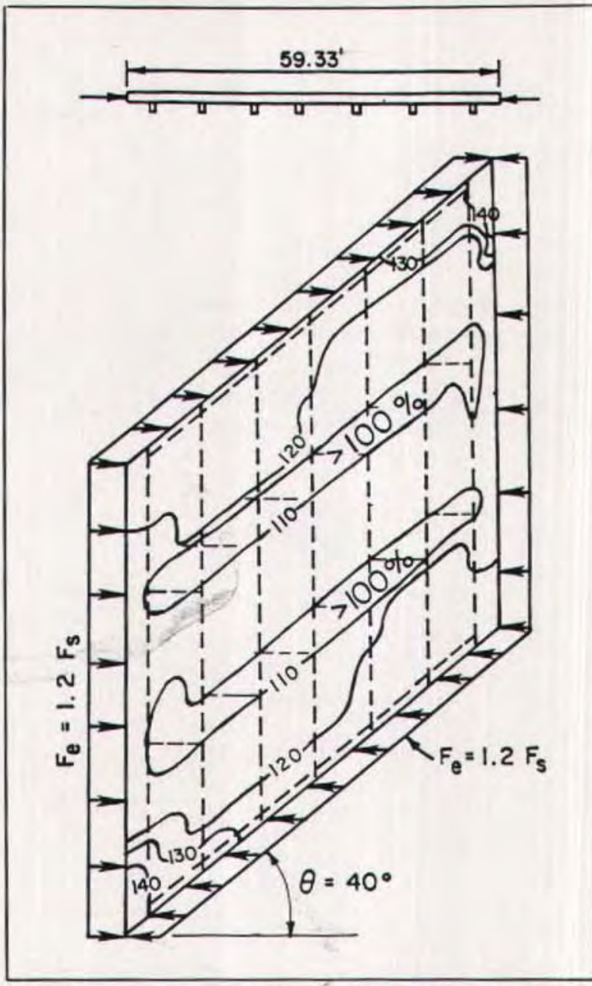

Fig. 5. Transverse stress contours for study bridge with 40 degree skew; amplified prestressing force applied along entire length of bridge; F_s produces edge stress = 100 percent (1 ft = 0.305 m).

using conventional continuous elastic beam theory.[9]

Maximum Tendon Spacing

The maximum spacing of transverse tendons is governed by two effects. First, if the tendons are spaced too far apart, shear lag in the slab will result in a nonuniform stress distribution in the interior regions. Second, the larger the tendon spacing, the larger the area of ineffectively stressed slab near the deck edge.

The shear lag effect seems to be well addressed by ACI provisions[10] for prestressed slab systems. The maximum allowable tendon spacing is the lesser of 8 times the slab thickness or 5 ft (1.5 m). This provision was set considering the load to be uniformly applied. However, it is believed that with adequate bonded distribution reinforcement, the ACI

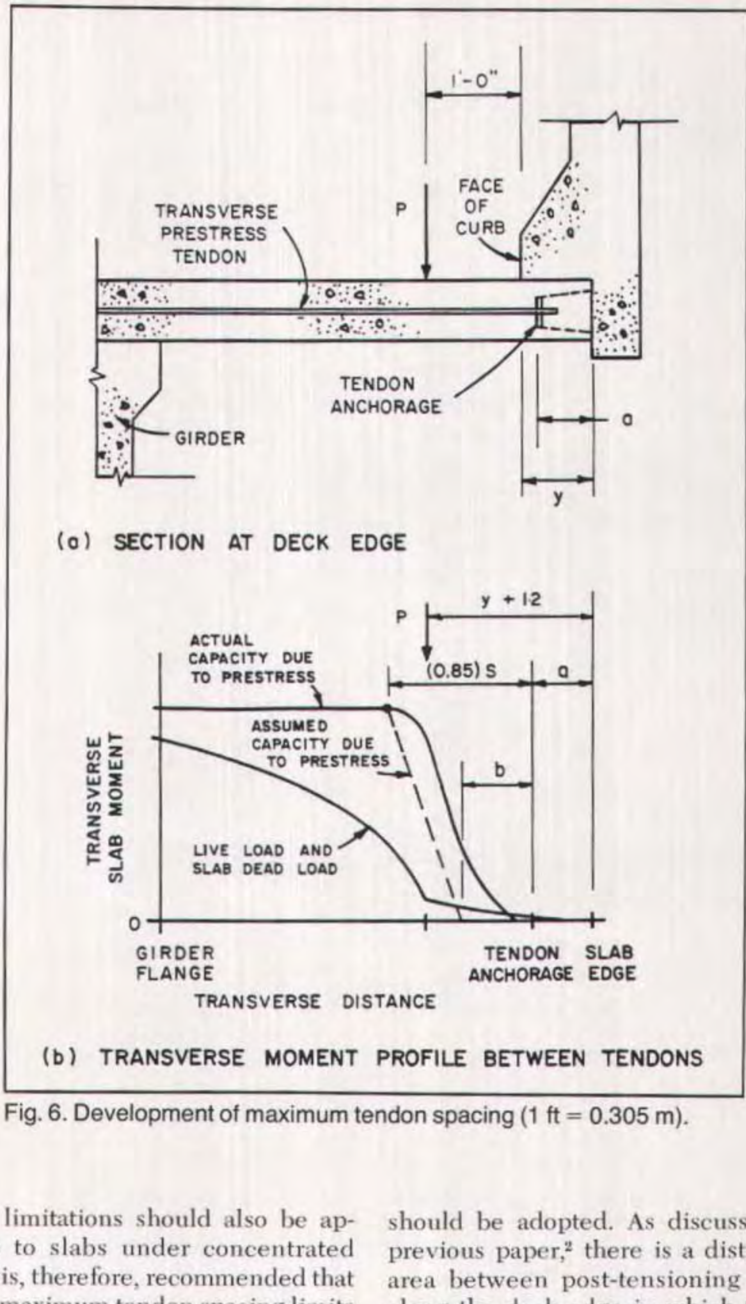

Fig. 6. Development of maximum tendon spacing (1 ft = 0.305 m).

spacing limitations should also be applicable to slabs under concentrated loads. It is, therefore, recommended that the ACI maximum tendon spacing limits be adopted as an upper limit for transversely prestressed bridge decks.

In addition to the shear lag consideration, a tendon spacing limit based on achieving an effective prestressing stress distribution at the deck edges should be adopted. As discussed in a previous paper,[2] there is a distribution area between post-tensioning strands along the deck edge in which the prestressing forces are not effective. Either the load must be kept off these areas, or resistance to the load must be provided by some other means. The position taken here is that it is preferable to prevent load application over these areas rather than providing passive reinforcement for local strengthening. This is because use of conventional reinforcement to carry service live loads would entail cracking of the concrete near the curb, where ponded water creates an especially corrosive environment.

Moments in the deck near the slab edge may be induced by either vertical loads or lateral rail impact loads. Only the vertical loads are considered in establishing the maximum transverse prestressing tendon spacing.

Fig. 6a shows a section through a deck at the longitudinal edge. A concentrated wheel load is located 1 ft (0.305 m) from the face of the guardrail, in accordance with AASHTO design specifications.[8] The distance from the edge of the deck to the bearing side of the tendon anchorage plate is represented as a, while y is the transverse distance from the deck edge to the inside face of the curb or rail. Fig. 6b shows the moment capacity and moment due to loading across this section, taken at midpoint between two tendons.

Referring to Fig. 6b, with the slab dead load moment of such small magnitude at the point of applied live load, the limiting acceptable design would be for the applied concentrated load to be located where the slab moment capacity due to prestressing alone reaches zero. It can be shown that for design purposes,[4] a reasonable limit for the tendon spacing to ensure an effective prestress distribution at the deck edges for resisting applied concentrated live load is given by Eq. (12):

$$S \leq 3\,(y - a + 12)\ \text{(in.)} \qquad (12)$$

where

- S = tendon spacing, in. (1 in. = 2.54 cm)
- y = transverse distance for the deck edge angle to inside face of curb or rail, in. (1 in. = 2.54 cm)
- a = distance from deck edge to the bearing side of the tendon anchorage plate, in. (1 in. = 2.54 cm)

Table 6. Maximum tendon spacings from Eq. (12).

y (in.)	Value of a (in.)		
	0	5	10 (upper limit)
0	36	21	6
6	54	39	24
12	72*	57	42
18	90*	75*	60

*Other limits will control maximum spacing.
Note: 1 in. = 2.54 cm.

The exact system of prestressing to be used will generally not be known at the time of design. A practical limit value for a should be recommended for those cases where the prestressing system is not known. From manufacturers' literature on bearing plates and pocket formers for tendon anchorages, it is found that the distance a may vary from zero, for anchorage plates bearing against the deck edge and covered by the railing concrete to 10¼ in. (26.0 cm), for a 1⅜ in. (3.5 cm) diameter threaded bar with a flat anchor plate. To account for all possibilities, the practical limit of a should be set at approximately the higher value. For simplicity of application, it is recommended that the upper limit of a be set at 10 in. (25.4 cm). Table 6 gives the maximum tendon spacings for various values of y and a as calculated using Eq. (13).

The maximum transverse prestressing tendon spacing allowed, then, should be the maximum given by the ACI limits of 8 times the slab thickness or 5 ft (1.5 m), or Eq. (12).

Tendon Layout for Skewed Bridges

On a nonskew bridge, the transverse tendons may be distributed at the

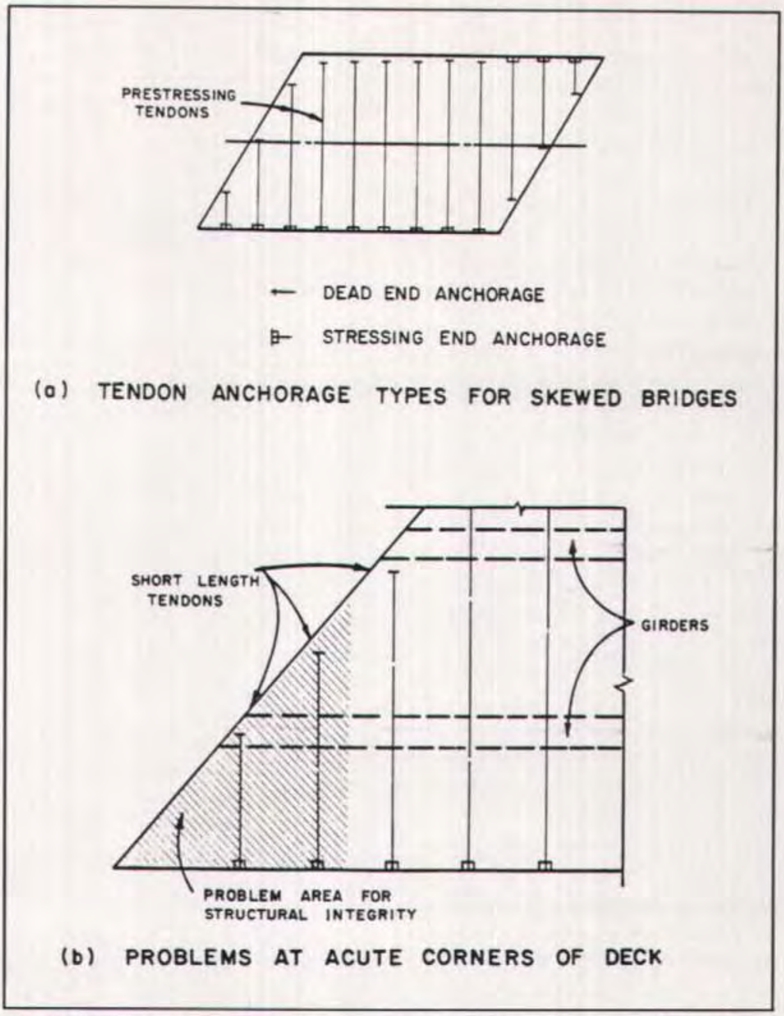

Fig. 7. Details of skewed bridge with perpendicular prestressing tendons.

specified spacings in the various zones along the entire bridge length. However, on a skewed bridge, complications arise near the abutments and expansion joints. In these regions, the use of tendons placed perpendicular to the girders results in varying tendon lengths. In addition, tendon anchorages would be required along the transverse edge of the deck.

It is generally not recommended that tendons be placed on a skew in these instances. The transverse prestressing force available to resist slab moments is reduced from the applied prestressing by the cosine of the skew angle. This amounts to nearly a 15 percent reduction for a bridge with a 30 degree skew, and thus would require the use of more prestressing steel. Furthermore, the effective transverse stress distribution in the slab is affected by the application of the post-tensioning forces on skew. Because of this reduced efficiency, the use of skewed tendons should generally be avoided wherever possible.

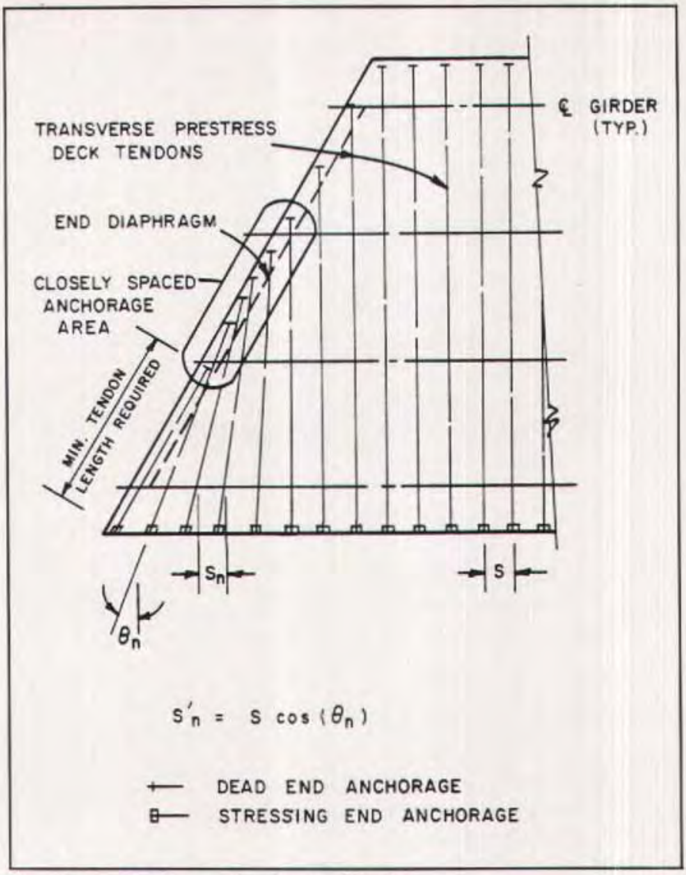

Fig. 8. Tendon layout for skewed decks.

For perpendicular tendons to be used on skewed bridges, several complications must be dealt with. Tendon anchorages along the transverse deck edge, required on a skewed bridge with perpendicular tendons, do not present a problem since dead end anchorages may be used and the tendons stressed from the longitudinal deck edge, as shown in Fig. 7a. The two major difficulties in this situation are at the acute corners of the deck, as illustrated in Fig. 7b. There the required tendon lengths become so short that losses due to anchorage seating are extreme. Tendon lengths shorter than, say, 12 ft (3.7 m) may be impractical since elongation during tensioning would be less than 1 in. (2.5 cm). In addition, the structural integrity of the extreme corner region is hard to maintain with transverse prestressing, especially for bridges with high skew angles, since it extends longitudinally beyond the end of the girder.

To avoid these problems, it is recommended that a fan arrangement of prestressing tendons be used at the acute corners of a skewed bridge deck as shown in Fig. 8. The tendons should be as long as possible to minimize wedge seating losses, and in any event, no less than say 12 ft (3.7 m) unless special precautions are taken to ensure adequate prestress after losses. The advantages of this tendon arrangement are that it provides a load path directly to the support, avoids high concrete stresses in the longitudinal direction due to live load, allows the use of longer prestressing tendons, and avoids closely spaced anchorages. When utilizing such a pattern, care should be taken not to extend the dead end of the tendons so far into the slab as to approach a fully skewed tendon layout.

The spacing of the fan tendons must be carefully detailed to account for the reduced spacing for these tendons, S_n', equal to the spacing for perpendicular tendons, S, multiplied by the cosine of the skew angle of the tendon. The resulting spacing is measured along the exterior girder.

At the transverse deck edge where fan tendon anchorages are spaced closely together (see Fig. 8), an integral end diaphragm should be provided to withstand the high compression stresses. If such a diaphragm is not used, an analysis, such as by the finite element method, should be made to determine the stresses at that location.

For those cases in which the slab is continuous over interior bridge bent locations, it is expected that all anchorages would be along the longitudinal edges of the slab.

Jacking Sequence

If all strands in a transversely prestressed bridge deck are stressed simultaneously, then there would not be any stress losses in the strands due to elastic shortening of the slab. However, stressing all tendons simultaneously is impractical. Successive stressing of tendons results in stress losses in all previously stressed tendons due to elastic shortening of the concrete slab. Maximum tendon stress losses would occur for each tendon post-tensioned individually. For the laboratory model bridge,[2,4] the maximum stress loss due to jacking sequence was calculated to be 3 percent. Ralls[5] reported the maximum stress loss as 3.8 percent, which is close to the calculated value. The effect of jacking sequence is insignificant when compared to other effects such as slab stress reductions due to the presence of diaphragms.

Variable Slab Thickness

There are cases in which a variable thickness or haunched slab might be used in a bridge deck. For purposes of determining transverse prestressing diaphragm restraint effects in these cases, it would be reasonable yet conservative to use the minimum slab thickness. As slab thickness decreases, the diaphragm restraining effects increase. Thus, using the minimum slab thickness would result in a higher calculated force required to overcome restraining effects.

Minimum Value of Compression

For most structural bridge applications envisioned, there is no need to specify a minimum desired value of compression which should be induced by the transverse prestressing, and hence no specific design recommendations will be proposed. However, should a unique occasion arise in which the deck slab may be extra thick, a reasonable minimum target value of compression which should be induced is 150 psi (1.0 MPa).

* * *

SERVICEABILITY, STRENGTH AND STRUCTURAL INTEGRITY

Besides the effect of lateral restraint on transverse prestress distribution, other design considerations for durable post-tensioned bridges were also evaluated in the overall study. The structural design implications of serviceability, strength and structural integrity are discussed in the following sections.

Crack Control

From the viewpoint of corrosion risk, cracks must be limited whether caused by structural loads or other factors such as temperature and shrinkage stresses in concrete.[1-3] Current crack control recommendations are assumed to be adequate in limiting nonstructural cracking.

Shrinkage and Temperature Reinforcement — The provisions of the current AASHTO Specifications[8] are assumed to be adequate with regard to minimizing concrete cracking due to shrinkage and temperature stresses. However, as written, these provisions imply that for a transversely prestressed bridge deck, the prestressing would be adequate as temperature and shrinkage reinforcement. This is not true, especially for a deck with unbonded tendons. It is recommended that the minimum temperature and shrinkage requirement for reinforced concrete in the AASHTO Specification Section 8.20 be met in the form of bonded auxiliary nonprestressed reinforcement at both top and bottom slab surfaces in both the transverse and longitudinal direction of all transversely prestressed bridge decks.

Allowable Tension Stresses — The durability study results[1-3] indicated that corrosion risk was reduced for crack widths limited to about 0.002 in. (0.05 mm) by the use of prestressing. However, even though little corrosion occurred for small crack widths, the Cl^- levels at the reinforcement level at crack locations exceeded the chloride corrosion threshold. On the other hand, in uncracked concrete, the Cl^- levels at reinforcement depth were below the threshold. This suggests that the prudent approach would be to eliminate cracking altogether under normal loading conditions. Thus, for a transversely prestressed bridge deck which is exposed to chlorides in service, any cracking would constitute a damage limit state. It is implicit that such a "crack free" design can only ensure corrosion protection if adequate thickness of concrete cover, adequate concrete quality and adequate compaction exist so that the "uncracked" concrete provides the necessary barrier to inhibit the corrosion mechanism.

Using a limit state design philosophy and considering the statistical dispersion of concrete cracking strength as well as potential fatigue problems[11] and likelihood of overloads on a bridge, the proposed design recommendation is to limit the extreme fiber deck slab tensile stresses under full service load to $2\sqrt{f_c'}$. The value $2\sqrt{f_c'}$ seems to be a reasonably conservative tension limit, yet has significant economic advantages over a zero tensile stress limit.

Bonded Transverse Reinforcement — When unbonded transverse prestressing is used, supplementary bonded reinforcement is needed to control cracking under overloads, and to ensure overall structural integrity. The amount of such bonded reinforcing, A_B, in the transverse direction recommended for each slab surface per foot width of deck follows from ACI 318 requirements:[10]

$A_B = 0.024\,t$ (in.^{2}) (1 in.^{2} = 6.45 cm^2)
where t is the overall thickness of deck (in.) (1 in. = 2.54 cm).

This amount of bonded transverse reinforcement should be placed in both the top and bottom of the deck when unbonded transverse prestressing is used and distributed uniformly.

If bonded transverse prestressing is used, supplementary transverse nonprestressed bonded reinforcement need be provided only for temperature and shrinkage control as previously described.

Bonded Longitudinal Distribution Reinforcement — As discussed in a companion paper,[2] the longitudinal moment in a typical composite I-beam and slab bridge deck due to concentrated wheel loads is approximately one-quarter of the transverse slab moment at that location. For typical bridge decks, this level of moment results in concrete tensile stresses on the bottom of the deck of less than $2\sqrt{f_c'}$. Such low stress values are much less than the tensile strength of the concrete. However, in case the concrete does become cracked, and because of the possibility of overloads, some longitudinal reinforcement must be provided to resist these moments. The amount of reinforcement required will be governed by either the design moment or the minimum reinforcing requirements to ensure ductile failure.

In view of the low values of longitudinal moment in the slab due to a wheel load determined in the laboratory study,[4] determination of the longitudinal distribution reinforcing in the bottom of the slab should be made by direct design. The design longitudinal moment should be one-quarter of the transverse live load plus impact moment, and the amount of reinforcement should conform to the minimum requirements of AASHTO Section 8.17.1.[8] However, to expedite the design process, a design value of (0.03) t (sq in. per ft width of deck) (1 in. = 2.54 cm; 1 ft = 0.305 m) for longitudinal reinforcement in slab-girder bridges appears adequate if a more exact determination is not desired.

For the reinforcing arrangement shown in Fig. 9, the maximum spacing of the longitudinal distribution bars allowed by AASHTO requirements for flexural reinforcement distribution (AASHTO Section 8.16.8.4) is 9.8 in. (24.9 cm). This is overly restrictive in this case since the longitudinal tensile stresses in uncracked concrete on the bottom of the slab are less than $2\sqrt{f_c'}$. Instead, a maximum spacing of 12 in. (30.5 cm) is recommended for longitudinal distribution reinforcing, which provides nearly three bars in the cone of load influence beneath a 20 in. (51 cm) wide wheel.

Transverse Cracking — A transversely prestressed bridge deck designed in accordance with the recommendations for transverse prestressing presented in this paper should be free of deck cracks running in the longitudinal direction. The great advantage of the absence of these cracks is that one mechanism by which corrosion of the reinforcement and freeze-thaw deterioration of the concrete takes place is eliminated. However, if slab cracking should occur running in the transverse direction across the deck and thus parallel to the transverse prestressing, the potential for substantial early deck deterioration will still be present and the highly stressed tendons may be exposed to corrosion attack. In Texas, the Texas State Department of Highways and Public Transportation (TSDHPT) reports that the primary cracking in their bridge decks occurs in the transverse direction, and consequently, transverse prestressing would not be particularly beneficial. This is the reason that as the study progressed, the need for minimum levels of longitudinal deck prestressing was also evaluated.

For slab and girder bridges, the two most promising methods for dealing with transverse cracking in the deck are the use of epoxy-coated reinforcement and longitudinal post-tensioning of the bridge deck. Because adequately thick, high quality, uncracked concrete pro-

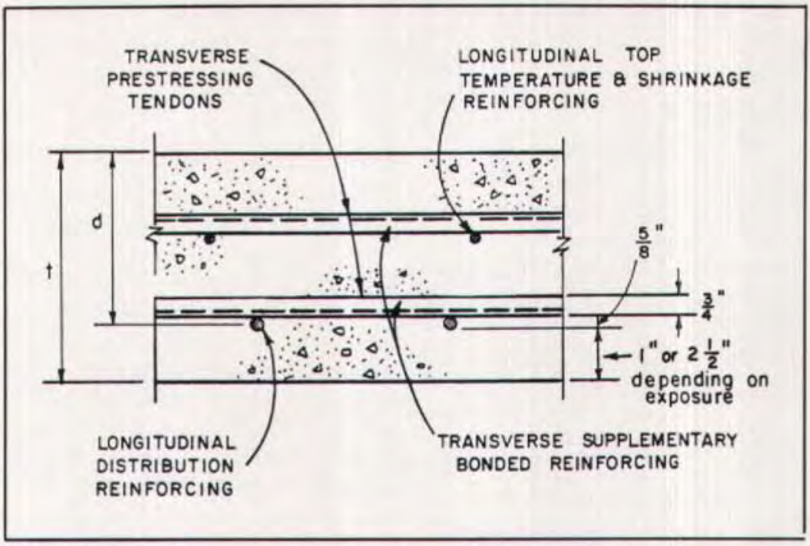

Fig. 9. Transverse section of deck showing depth to longitudinal distribution reinforcing (1 in. = 2.54 cm).

tects the reinforcement against corrosion, resists freeze-thaw deterioration, and has a low susceptibility to fatigue, longitudinal post-tensioning of the bridge deck is the more promising method. If a deck is not longitudinally post-tensioned, epoxy-coated reinforcement and bonded transverse prestressing should be used as a minimum level of protection. When epoxy-coated reinforcement is used, all reinforcing located within 4 in. (10.2 cm) of concrete surfaces exposed to an aggressive environment should be coated.

Longitudinally post-tensioning a bridge superstructure is a viable method of preventing transverse cracking in bridge decks. The ACI Building Code[10] recommends that if post-tensioning is used to counteract temperature and shrinkage stresses, a minimum average compressive stress of 100 psi (0.69 MPa) due to the effective prestress (after losses) on gross concrete area should be provided.

Several possibilities exist for longitudinally post-tensioning slab and girder bridge superstructures to eliminate transverse temperature and shrinkage cracking in the deck. These include: longitudinal post-tensioning of only the slab for the full length of the bridge; longitudinal post-tensioning of only the slab in the end quarters of a span in conjunction with using shored construction; and designing pretensioned girders for construction loads only, then post-tensioning the completed structure for the full design loads plus the desired compression in the deck.

Regardless of the particular longitudinal prestressing scheme used, the same protection provided for transverse prestressing tendons and anchorages must also be provided for longitudinal tendons. The minimum bonded nonprestressed temperature and shrinkage reinforcement should still be provided when the deck is longitudinally post-tensioned. Also, girders for a bridge using this method of construction must be designed to accept the additional stress the longitudinal post-tensioning imposed. Details for designing the longitudinal post-tensioning for simple and continuous span bridges as well as detailed design examples may be found in Ref. 6.

Deflection Control

The use of prestressing generally decreases live load deflections and thus live load deflection problems should not be a concern for a transversely prestressed bridge deck. A companion paper[2] shows that live load deflections of a transversely prestressed deck slab are negligible. However, at the start of the present study, there was also concern for camber and deflection effects from transverse prestressing. Ralls[5] reported that the maximum upward camber and downward deflection was less than 0.01 in. (0.25 mm) due to prestressing the model bridge deck. This value represents a camber or deflection to slab span ratio of about 0.02 percent. Thus, these small deflections are of no practical concern.

Ultimate Strength

In a companion paper,[2] it was shown that the experimental results of both this study and others conclusively confirm that the failure mode of the interior portion of a deck slab is punching shear. Most current practices (except for the Ontario slab design procedure) calculate the ultimate capacity of the bridge deck assuming one-way flexural behavior. This ignores in-plane forces (arching action) and redistribution of load in the longitudinal direction. This will normally result in an underestimation of strength by a factor of at least 6 in interior regions where membrane action is able to develop. A lower and more reasonable value will be found elsewhere, such as for the deck overhangs. If a simplified shear strength analysis of slabs including arching effects were available, the use of simple middepth tendons for transversely prestressed bridge decks could be expedited. In the absence of such a method, however, it is recommended that the current procedure for checking the deck strength be used for transversely prestressed bridge decks.

Bonded Versus Unbonded Tendons

There are both advantages and disadvantages in using either an unbonded or a bonded post-tensioning system. The results from the durability study[1,3] indicated that both an unbonded tendon completely surrounded by grease with an integral plastic duct, and a bonded tendon completely surrounded by grout with a rigid galvanized duct provided adequate corrosion protection in the length between anchorages. The unbonded tendon surrounded by grease and a plastic duct is more vulnerable to corrosive attack if the plastic duct is not completely assembled and joined to protected anchorages or is damaged before concrete is cast. The bonded system seems to have an additional corrosion protection because moisture must penetrate the concrete cover, duct, and the grout before corrosion can occur. In both systems, it is necessary to maintain continuous protection where the duct and anchors join.

The ultimate strength behavior of unbonded and bonded prestressing systems also has important design implications. The principal difference in the tendon behavior is the steel stress at failure. Since the tendon is free to slip in an unbonded system, the strain is more or less equalized along its length, and the strain at the critical section is lessened. Consequently, when the concrete crushing stress is reached, stress in the steel is often far below its ultimate strength.[9] Thus, for the same amount of prestressing steel in an unbonded and a bonded member, the ultimate strength of the bonded member will be 10 to 30 percent greater.[9]

In comparing the cost of an unbonded single-strand system to that of a grouted single-strand system, it is usually found that the unbonded system is less expensive. The additional cost with use of a bonded system is a result of the cost of grouting hardware and the cost of grouting labor operations. However, these costs are basically constant whether there is a single or whether there are multiple tendons. Thus, the cost of multiple tendons in a single duct with a single pair of anchorages approaches the cost of several unbonded single tendons with the associated several pairs of anchorages. From a cost standpoint, a grouted multistrand system can be just as economical as an unbonded single-strand system.

Anchorage Design

Anchoring a prestress tendon at the edge of the thin bridge deck induces large bursting and spalling stresses which could lead to substantial cracking or even violent failure of the concrete at the anchorage zone. To control these stresses, sufficient amounts of concrete and confining reinforcement must be provided. Currently, there are several methods available for the design of prestress tendon anchorage zones. However, none of these methods seem adequate for the analysis of multiple anchorages in thin slabs. It is, therefore, recommended that provisions be included in the project specifications requiring the contractor to show by some appropriate means that the proposed anchorage detail is adequate prior to its approval for use.

Railing Attachment

As has been discussed, there are areas along the sides of a transversely prestressed bridge deck which are ineffectively stressed since the post-tensioning is applied at discrete locations and tendon anchorages are often recessed into the edge of the slab. Traffic rails with continuous attachment to the bridge deck such as concrete barriers, as well as railing utilizing posts anchored directly to the deck, can impose concentrated stresses and transverse moments near the slab edge where moment capacity due to prestressing is not present. It is, therefore, recommended that decks with traffic railings located within a distance equal to the spacing of the transverse tendons from the slab edge should be provided with nonprestressed reinforcement adequate to resist lateral railing impact loads. This reinforcement should provide the full required moment capacity for a transverse distance from the deck edge equal to the tendon spacing.

DURABILITY CONSIDERATIONS

The design implications for improving the durability of bridge decks with the use of transverse prestressing are discussed in the following sections.

Concrete Cover and Concrete Quality

Even though deck prestressing should reduce cracks in a bridge deck, there is still a risk of corrosion due to the long-term exposure of chlorides which penetrate slowly through uncracked concrete. The durability study results[1,3] indicate that the combination of a 2 in. (5.1 cm) clear cover and a water-cement ratio (w/c) of 0.45 was adequate for corrosion protection in uncracked concrete to resist the aggressive exposure of the relatively short durations (5 to 7 months) accelerated tests. However, Weed[12] found that in actual construction the depth of cover over bridge deck steel was approximately normally distributed with a mean value close to the specified value, but with a standard deviation of approximately ⅜ in. (1 cm) for a 2 in. (5.1 cm) cover. This implies that about 15 percent of the steel could be expected to have a cover less than 1⅝ in. (4.1 cm). The durability study results indicate that at this depth the Cl^- levels would be greater than the corrosion chloride threshold, and thus would be at a high corrosion risk.

The current AASHTO Section 9.25.1.2[8] value is 2 in. (5.1 cm) when deicers are used. This is very marginal as a practical all-inclusive requirement. Setting a minimum clear cover value of 2.5 in. (6.4 cm) would ensure that most steel would have at least a 2 in. (5.1 cm) cover, which would be at an acceptable corrosion risk. Therefore, it is proposed that a minimum 2.5 in. (6.4 cm) cover over all top reinforcement with a maximum water-cement ratio of 0.45 be used for transversely prestressed bridge decks exposed to chlorides in service.

This combination is in complete agreement with the provisions for reinforced concrete slabs in the current ACI Building Code.[10] The current AASHTO Section 9.25.1 recommendation of 1 in. (2.5 cm) for concrete cover under bottom slab reinforcement is assumed to be adequate when the chloride exposure is limited to the top of the bridge deck. However, if there is any threat of salt exposure at the bottom slab surfaces, such as would occur in a marine environment, 2.5 in. (6.4 cm) of bottom cover is also recommended.

Protection of Prestressing

The consequences of corrosion of the prestressing steel in a transversely prestressed bridge deck would be quite severe. It is recommended that prestressing tendons be protected by an impenetrable barrier which extends the full length between anchorages and is physically attached to the anchorages. This would completely eliminate any moisture path to the tendons between anchorages. A duct with complete grouting would provide the best protection against corrosion; however, a rugged grease-filled plastic duct could also provide adequate protection as long as no defects exist in the duct. The most current information on appropriate materials for corrosion protection of post-tensioning tendons is found in Refs. 13 to 16. It is essential that the duct be examined for any damage after the tendon is placed and before the concrete is cast. Any damage must be repaired by appropriate measures.

Anchorage Protection

Maintaining a minimum 2.5 in. (6.4 cm) concrete cover around all surfaces of an anchorage would normally provide adequate corrosion protection. However, a minimum 2.5 in. (6.4 cm) cover over the prestressing ducts will likely result in less concrete cover over some areas of the anchorage. For the durability specimens[1-3] with a concrete cover of 2 in. (5.1 cm) over the prestressing, only ¾ in. (1.9 cm) of cover was provided over the top anchorage surfaces. The heavy corrosion which resulted in some of these anchorages clearly suggests that reliance on positive measures other than concrete cover must be used for anchorage corrosion protection.

In unbonded post-tensioning, the anchorage is critical throughout the entire life of the structure. Therefore, it is proposed that the anchorage must be completely sealed against moisture. This sealing can be achieved with the use of a suitable coating material such as an epoxy-resin compound, or a specially made covering of plastic or other suitable materials which completely encapsulates the anchorage, jaws, and strand extensions. Providing a physical barrier to moisture around the anchorages as well as the prestressing tendon effectively results in an "electrically isolated" tendon which will be at low risk to corrosion, as suggested by Schupack.[16]

It is also proposed that external anchorages shall not be used even if protected by an auxiliary protective barrier. All protected anchorage components, including the strand extensions, must be surrounded by not less than 1½ in. (3.8 cm) of concrete or mortar.

After stressing the tendons and sealing the anchorages, stressing pockets should be filled with a suitable chloride-free mortar with low shrinkage properties. As was done for the durability specimens, it is recommended that the pocket be painted with an epoxy-resin bond agent to improve adhesion of the fresh mortar to the hardened concrete.

Cl^- Content

The durability study test results[1,3] indicate that in order to minimize the risk of corrosion, the maximum water soluble Cl^- content in concrete by weight of cement should be limited to 0.06 percent. The limit on Cl^- content would be verified by trial mix on test samples.

OTHER APPLICATIONS

Although the emphasis of the experimental testing program was for cast-in-place post-tensioned decks on precast concrete girders, the general findings of the study are appropriate for other applications. Specifically, the proposed recommendations are equally valid for prestressed concrete decks on steel girders.

Likewise, the use of precast stay-in-place panels with subsequent prestressing of the deck is not precluded from the general design procedures which were derived. The use of precast panelized systems could shorten erection time. The use of this type of system may be particularly attractive for redecking of an existing bridge.

SUGGESTED REQUIREMENTS FOR TRANSVERSELY PRESTRESSED BRIDGE DECKS

Because the AASHTO Specifications[8] are minimum requirements for bridge design, some of the ideas included in this paper are not fully represented in the suggested provisions. Specifically, neither longitudinal post-tensioning of the deck nor epoxy-coated reinforcement are expressly required. In addition, design details for continuous bridges and tendon placement in skewed slabs have been omitted. Guidance for designing the longitudinal prestressing and details for continuous bridges may be found in Ref. 7.

The proposed design recommendations follow the limit states design concept. When a structure becomes unfit for its intended use, it is said to have reached a limit state.[17] There are basically three limit states for a transversely prestressed bridge deck that are considered by the proposed design recommendations. They are:

1. Ultimate limit state which might be evidenced by a flexural failure or a punching shear failure;

2. Damage limit state in the form of premature or excessive cracking which might allow penetration of corrosive agents;

3. Durability limit state in the form of unacceptable corrosion of reinforcing steel and deterioration of concrete which would impair the performance

and integrity of the prestressed bridge deck.

The proposed AASHTO provisions assume that all other portions of the AASHTO Specifications[8] are applicable. Some of the provisions could be directly included in existing sections of the AASHTO Specifications, while others would require the formation of new sections.

SUGGESTED SPECIFICATION PROVISIONS

Metric Conversion Factors

1 in. = 2.54 cm; 1 ft = 0.305 m; 1000 psi = 6.894 MPa; 1 kip = 4.45 kN.

1.0 Notation

a = distance from slab edge to the bearing side of transverse tendon anchorage, in.

A_B = area of bonded nonprestressed transverse reinforcement per foot width of slab, in.^{2}

A_L = area of bonded nonprestressed longitudinal distribution reinforcement per foot width of slab, in.^{2}

C_K = correction factor of diaphragm stiffness

C_L = correction factor for diaphragm spacing; applied only when interior diaphragms are present

C_t = correction factor for bridge deck thickness

$(EA)_D$ = cross-sectional diaphragm stiffness where E is the modulus of elasticity of diaphragm material (ksi) and A is the cross-sectional area of diaphragm resisting axial deformation in.^{2}

f'_c = specified compressive strength of concrete, psi

$\sqrt{f'_c}$ = square root of specified compressive strength of concrete, psi

F_e = amplified transverse slab prestress force per unit edge length required to overcome web restraining effects in box-girder bridges and diaphragm restraining effects in slab-girder bridges

F_S = transverse slab prestress force per unit edge length required to resist effects of structural loads assuming no restraining effects

L = longitudinal span length of the superstructure, ft

N = number of line of diaphragms

P_D = prestress force required in diaphragms to overcome diaphragm restraining effects in slab-girder bridges, units of force

S_D = interior diaphragm spacing, ft

t = bridge deck slab thickness, in.

W = bridge slab width, ft

y = distance from slab edge to inside face of railing or barrier wall, in.

θ = bridge skew angle as measured between the transverse edge of the deck slab and the normal to the longitudinal bridge centerline, degrees

1.1 Scope

These provisions shall apply for decks of composite slab-girder bridges and of box-girder bridges which utilize transverse prestressing.

1.2 Design Assumption

The bridge deck shall be designed assuming that it behaves as an elastic slab continuous over the supporting girders in a slab-girder bridge and as an elastic slab continuous over the webs in a box-girder bridge.

1.3 Transverse Prestressing Effects

1.3.1 Box Girder Bridges

1.3.1.1 Transverse prestressing shall be considered effective in all regions of top slabs of box-girder bridges only if diaphragms are not present at the time of transverse prestressing or if the diaphragms are transversely prestressed to a level consistent with the deck prestressing.

1.3.1.2 Design of a bridge deck which utilizes transverse prestressing shall take into account the influence of web restraint, losses in prestressing, and secondary slab moments on transverse prestress distribution. The effects of transverse prestress on transverse moments and shears in the webs and soffits of the box-girder section shall be considered in the analysis.

1.3.1.3 In lieu of a more exact analysis, the restraining effect of webs on transverse prestress distribution may be accounted for in accordance with the approximate procedure presented in Section 1.3.1.4.

1.3.1.4 The amplified transverse prestress force per unit edge length required at all slab locations to overcome web restraining effects shall be not less than:

One-Cell Box Section:

$$F_e = 1.1 F_S \qquad (1.3.1.4\text{-}1)$$

$$F_e = 1.15 F_S \qquad (1.3.1.4\text{-}2)$$

Three (or greater)-Cell Box Section:

$$F_e = 1.4 F_S \qquad (1.3.1.4\text{-}3)$$

1.3.2 Slab-Girder Bridges

1.3.2.1 Design of a bridge deck which utilizes transverse prestressing shall take into account the influence of diaphragm restraints, losses in prestressing, and secondary slab moments on transverse prestress distribution. The influence of diaphragms needs to be considered only if the diaphragms will be in-place at the time of transverse prestressing.

1.3.2.2 In lieu of a more exact analysis, the effect of diaphragms on transverse prestress distribution may be accounted for in accordance with the approximate procedures presented in either Section 1.3.2.3 or Section 1.3.2.4.

1.3.2.3 The prestress force required in the diaphragms of nonskew bridges to overcome diaphragm restraining effects shall be not less than:

$$P_D = C_t C_K C_L 1.6\ F_S \qquad (1.3.2.3\text{-}1)$$

where

$$C_t = 8/t \qquad (1.3.2.3\text{-}2)$$
$$C_K = (EA)_D / 640{,}000 \qquad (1.3.2.3\text{-}3)$$
$$C_L = 25/S_D \qquad (1.3.2.3\text{-}4)$$

No more than two values for C_t, C_K, and C_L shall be taken less than 1 in Eq. (1.3.2.3-1). In Eq. (1.3.2.3-1), F_S shall be computed for a 1 ft length of slab.

Unless an analysis is carried out in accordance with Section 1.3.2.2, the prestress force calculated by Eq. (1.3.2.3-1) shall be applied at a distance not exceeding 1/12 the height of the diaphragm from the centroid of the diaphragm.

1.3.2.4 The amplified transverse prestressing force per each 1 ft edge length of slab in the diaphragm regions required to overcome diaphragm restraining effects shall be not less than:
For bridges with $\theta \leq 10$ degrees:

$$F_e = 1.6 F_S \qquad (1.3.2.4\text{-}1)$$

For bridges with $\theta > 10$ degrees:

$$F_e = 1.2 F_S \qquad (1.3.2.4\text{-}2)$$

The amplified transverse prestress force per unit edge length required by Eqs. (1.3.2.4-1) or (1.3.2.4-2) shall be applied along the slab edge with a uniform distribution at diaphragm locations for an edge length of:

$$x \geq W \tan\theta + 4 \text{ ft} \leq (L + W \tan\theta)/N \tag{1.3.2.4-3}$$

where W and L are in units of feet. For end diaphragm regions, x shall be measured from the transverse slab edge on nonskew bridges, and from the acute slab corner on skewed bridges. For intermediate diaphragm regions, the length x shall be considered centered over the intersection of the longitudinal bridge centerline and the overall centerline of that set of diaphragms.

1.4 Maximum Transverse Tendon Spacing

The maximum spacing of individual transverse tendons or groups of tendons shall not exceed eight times the deck slab thickness, 5 ft, nor 3 $(y - a + 12)$. Without more precise information, the value of a may be taken as 10 in.

1.5 Stresses at Service Loads After Losses Have Occurred

The tensile concrete stress in precompressed tensile zones of transversely prestressed bridge decks after all allowances for losses shall not exceed $2\sqrt{f'_c}$.

1.6 Minimum Bonded Reinforcement

For a transversely prestressed bridge deck which utilizes unbonded construction, the minimum area of top and bottom uniformly distributed supplementary bonded reinforcement per foot width of slab in the transverse direction shall be computed by:

$$A_B = 0.024\,t \tag{1.6-1}$$

1.7 Distribution Reinforcement for Slab-Girder Bridges

1.7.1 For slab and girder bridges, bonded longitudinal distribution reinforcement in the bottom of a transversely prestressed bridge deck shall be provided to resist at least ¼ the maximum design transverse live load plus impact slab moment.

1.7.2 The requirements of Section 1.7.1 may be considered satisfied if distribution reinforcement is provided in accordance with the following formula:

$$A_L \geq (0.03)\,t \tag{1.7.2-1}$$

1.7.3 The specified amount of distribution reinforcement shall be uniformly spaced between girder flanges. Individual bars shall not be spaced farther apart than 12 in.

1.8 Shrinkage and Temperature Reinforcement

1.8.1 For all transversely prestressed bridge decks, reinforcement for shrinkage and temperature stresses shall be provided near the top and bottom slab surfaces not otherwise reinforced with sufficient bonded nonprestressed reinforcement, in accordance with AASHTO 8.20.

1.8.2 Prestressing tendons used to control shrinkage and temperature stresses in the longitudinal direction shall be proportioned to provide a minimum average compressive stress of 100 psi in the slab after all losses. Use of such tendons does not negate the requirements of Section 1.8.1.

1.9 Tendon Anchorage Zones

Post-tensioning anchorages and supporting concrete in transversely prestressed bridge decks shall be designed to resist bursting, splitting, and spalling stresses induced by the maximum tendon jacking force, for strength of concrete at time of prestressing. Adequacy of the anchorage zone design shall be demonstrated prior to its acceptance for use.

1.10 Traffic Railings

Transversely prestressed bridge decks with traffic railings located within a distance equal to the spacing of the transverse tendons from the slab edge shall be provided with nonprestressed reinforcing to resist transverse railing loads. The full moment capacity required shall be provided for a distance from the slab edge equal to the tendon spacing.

1.11 Special Exposure Requirements

1.11.1 For corrosion protection of transversely prestressed bridge decks exposed to deicing salts, marine environments or any other corrosive environments, the maximum water-cement ratio of concrete shall not exceed 0.45.

1.11.2 For corrosion protection of transversely prestressed bridge decks exposed to chlorides in service, the maximum water soluble chloride ion concentrations in test samples of hardened concrete taken from a trial mix shall not exceed 0.06 percent by weight of cement.

1.11.3 For corrosion protection, the minimum clear concrete cover over all reinforcement in a transversely prestressed bridge deck directly exposed to chlorides in service shall be 2½ in.

1.11.4 For corrosion protection, all anchorages, prestressing, and strand extensions shall be fully encapsulated by a durable protective barrier which prevents the penetration of moisture. Protective measures for unbonded single strands shall conform to "Specification for Unbonded Single Strand Tendons" (PTI, 1984, Ref. 15).

1.11.5 After placement of prestressing tendons and anchorages and of conventional reinforcement and before concrete is cast, any damage to the protective barrier surrounding the tendons and anchorages shall be repaired.

1.11.6 All anchorage components including strand extensions shall be covered by not less than 1½ in. of concrete or mortar as measured from any exposed surface.

1.11.7 Stressing pockets shall be filled with a suitable chloride-free low-shrinkage mortar. Before placing the mortar, the sides of the pocket shall be painted with a suitable resin bond agent to improve adhesion.

COST ANALYSES

A primary criterion for selecting a particular structural system is most often cost. The costs considered are the initial construction cost and the life cycle cost of the system. The bridge of the design example presented in the Appendix was constructed in Texas as a conventionally reinforced deck, and thus, a more direct cost comparison between prestressed and conventionally reinforced decks could be made.

Based primarily on average 1984 Texas State Department of Highway and Public Transportation (TSDHPT) bid tabulations, initial cost estimates for the example bridge in the Appendix are summarized in Table 7 for four construction options. The costs associated with the post-tensioning in the transversely and longitudinally prestressed options were obtained from a post-tensioning supplier. It is evident from Table 7 that any type of design which incorporates features which should appreciably increase the durability of bridge decks increases the construction cost somewhat. However, the increase in cost is fairly close for the different construction options designed to increase durability.

The construction cost increase for the designs with enhanced durability fea-

Table 7. Initial cost figures for example bridge of Appendix.

Cost	Conventionally reinforced	Conventionally reinforced w/top bars epoxy coated	Transverse prestressed w/top bars epoxy coated	Transverse and longitudinally prestressed
Total deck cost ($)	13,000	15,200	15,900	
Unit deck cost ($/ft^2)	6.57	7.68	7.83	8.03
Normalized deck cost	1.00	1.17	1.19	1.22
Total unit bridge cost ($/ft^2)	25.00	26.11	26.26	26.46
Normalized bridge cost	1.00	1.04	1.05	1.06

Metric conversion factor: 1 ft^2 = 0.093 m^2.

tures on the simple span bridge of the Appendix ranges from 17 to 22 percent of the deck cost, or 4 to 6 percent of the total bridge cost. If life cycle costs are considered, the cost of bridge decks constructed with features for increased durability is less than that for a conventionally reinforced deck assuming a 5 to 10 year longer service life. The amount of savings could be 20 percent or more for the simple span bridge example in the Appendix.[4] Between the three construction options for increased durability, the life cycle cost is fairly uniform, and thus, cost competitive with each other.

PROTOTYPE TRANSVERSELY PRESTRESSED BRIDGE DECK

In 1986, the Texas State Department of Highway and Public Transportation (TSDHPT) constructed a trial transversely post-tensioned slab and girder bridge using single strand unbonded tendons in LaGrange, Texas. Compared to their vast experience with conventionally reinforced deck slabs, they found the construction process for this trial bridge both time and labor intensive.

The use of single strand tendons resulted in a very large number of stressing operations. Repeated handling of the stressing rams along the edge of the bridge was burdensome and time consuming.

The contractor's actual costs were greater than anticipated. This in part was most likely due to the unfamiliarity of the construction crew to this new method and relatively little indoctrination provided to the constructor, constructor's crew and inspecting field forces. Another disadvantage noted by the TSDHPT was that their choice of using unbonded tendons instead of bonded tendons most likely precluded the future widening of this particular bridge.

CONCLUSIONS

Premature deterioration of concrete bridge decks has become a major problem in the last 20 years. The primary causes of this deterioration are corrosion of the reinforcing steel and freeze-thaw action. This investigation focused on the application of prestressing to bridge decks for the prevention of concrete cracking, thereby sealing out chlorides and water which initiate reinforcing corrosion and concrete deterioration. It is implicit that such a "crack free" design can only ensure corrosion protection if adequate thickness of concrete cover, adequate concrete quality and adequate compaction exist so that the "uncracked" concrete provides the necessary barrier to inhibit the corrosion mechanism. The primary objectives of the research program were to determine the effect of major variables on corrosion protection in concrete slabs, evaluate the structural effects of prestressing bridge decks, and develop design recommendations for the implementation of prestressing for bridge decks.

The major conclusions from the study are as follows:

1. A desirable approach for the design of concrete bridge decks exposed to aggressive environments is to minimize cracking under normal loading conditions through the use of prestressing. This is supported by the results of the durability study[1,3] which show that while proper concrete cover, quality and compaction are absolutely essential to ensure durability of uncracked concrete, concrete quality and cover have little effect on chloride penetration in cracked concrete. The test specimens showed that even in such high quality slabs, corrosion of reinforcement in slabs initiates after cracking occurred. The corrosion then occurs primarily at crack locations. Prestressing, however, reduces the potential for corrosion and possibly freeze-thaw damage by limiting the penetration of chlorides, moisture, and oxygen through the cracks commonly associated with reinforced concrete, as long as adequate concrete quality and cover are provided.

2. Transverse prestressing of a slab-girder bridge can effectively develop compressive stresses in the slab to counteract tensile stresses that occur due to live loads, as demonstrated by the lateral post-tensioning stress distribution tests.[2,4] The same was found to be the case for box-girder bridges through analytical studies. The desired transverse stress distribution in a transversely prestressed deck is mainly affected by the restraining actions of the diaphragms. These restraints may be effectively compensated for by prestressing the slab before diaphragms are installed, increasing the amount of transverse prestressing in the deck near the diaphragms, or post-tensioning the diaphragms themselves.

3. A prestressed bridge deck requires approximately the same level of design effort, should need less maintenance, and should have a longer service life than a conventionally reinforced slab with uncoated reinforcing steel. Since construction forces will probably be unfamiliar with slab prestressing techniques, initial uses of such systems will require extra indoctrination for construction and inspector field forces. Except for such added costs on initial projects, initial construction cost of a prestressed deck should be competitive with that of a conventionally reinforced deck with coated steel and will increase the total construction cost of the bridge approximately 5 to 10 percent.

4. A prestressed deck designed in accordance with the recommendations presented in this paper and the AASHTO slab live load moments[8] should exhibit essentially linear elastic behavior through factored load levels.[2,4] If a more "exact" method is used to determine the slab live load moments, the

deck should still behave elastically beyond service load levels. Failure of a prestressed deck is expected to be by punching shear at a *minimum* factor of safety against live load plus impact of seven. This high factor of safety suggests that excluding the effects of compressive membrane forces in the structural analysis may lead to excessively conservative deck designs.

DESIGN RECOMMENDATIONS

The concept of prestressing bridge decks has been shown to be viable as well as advantageous. The design recommendations and suggested AASHTO Specification provisions found in this paper should give ample guidance for the design of prestressed bridge decks. For brevity, detailed recommendatons concerning design of longitudinal prestressing and continuous span bridges have been omitted from this paper, but may be found in Ref. 7.

It is recommended that for highway bridges located in areas where exposure to deicing salts and freeze-thaw conditions are expected, as in a marine environment, a prestressed bridge deck following the recommendations presented in this paper should be considered as a viable option. Such decks should preferably be prestressed in both the longitudinal and transverse directions. If only transverse prestressing is utilized, however, the nonprestressed reinforcement in the top of the deck should be epoxy-coated.

For maximum effectiveness in corrosion resistance of prestressed decks utilizing post-tensioning, the post-tensioning tendon system must be completely encapsulated in a corrosion resistant barrier. This requires careful placement and inspection. Overall structural integrity must be ensured by provision of an adequate amount of auxiliary bonded reinforcement if unbonded tendons are utilized. Thus, the provision of both an adequate corrosion barrier and improved structural integrity indicate that grouted, bonded tendons are highly preferable for deck prestressing.

* * *

REFERENCES

1. Poston, R. W., Carrasquillo, R. L., and Breen, J. E., "Durability of Post-Tensioned Bridge Decks," *ACI Materials Journal*, V. 87, No. 4, July-August 1987, pp. 315-326.
2. Poston, R. W., Phipps, A. R., Almustafa, R. A., Breen, J. E., and Carrasquillo, R. L., "Effects of Transverse Prestressing in Bridge Decks," *Journal of Structural Engineering*, ASCE, V. 114, No. 4, April 1988, pp. 743-764.
3. Poston, R. W., Carrasquillo, R. L., and Breen, J. E., "Durability of Prestressed Bridge Decks," *Report 316-1*, Center for Transportation Research, The University of Texas at Austin, July 1985, 176 pp.
4. Phipps, A. R., Almustafa, R. A., Ralls, M. L., Poston, R. W., Breen, J. E., and Carrasquillo, R. L., "Structural Effects of Transverse Prestressing in Bridge Decks," *Report 316-2*, Center for Transportation Research, The University of Texas at Austin, July 1985, 246 pp.
5. Ralls, M. L., "Stress Distribution in a Transverse Prestressed Slab-Girder Bridge Deck," MS Thesis, The University of Texas at Austin, May, 1983.
6. Almustafa, R., "The Analysis of Transverse Prestressing Effects in Bridge Decks," PhD Dissertation, The University of Texas at Austin, December 1983.
7. Poston, R. W., Phipps, A. R., Breen, J. E., and Carrasquillo, R. L., "Design Procedures for Prestressed Concrete Bridge Decks," *Report 316-3F*, Center for Transportation Research, The University of Texas at Austin, November 1985, 192 pp.
8. American Association of State Highway and Transportation Officials, *Standard Specifications for Highway Bridges*, 13th Edition, 1983.
9. Lin, T. Y., and Burns, N. H., *Design of Prestressed Concrete Structures*, 3rd Edition, John Wiley & Sons, New York, 1981.
10. ACI Committee 318, "Building Code Requirements for Reinforced Concrete (ACI 318-83)," American Concrete Institute, Detroit, Michigan, 1983.
11. Overman, T. R., Breen, J. E., and Frank, K. H., "Fatigue Behavior of Pretensioned Concrete Girders," *Report 300-2F*, Center for Transportation Research, The University of Texas at Austin, 382 pp.
12. Weed, M. R., "Recommended Depth of Cover for Bridge Deck Steel," *Transportation Research Record No. 500*, Transportation Research Board, 1974.
13. Ad Hoc Committee of the FIP Commission on Prestressing Steels and Systems, "Tentative Recommendations for the Corrosion Protection of Unbonded Tendons," PCI JOURNAL, V. 28, No. 1, January-February 1983, pp. 40-49.
14. Etienne, C., Binnekamp, D., Copier, W., Hendrickx, R., and Smit, C., "Corrosion Protection of Unbonded Tendons," *HERON*, V. 26, No. 3, 1981.
15. PTI Committee, "Specification for Unbonded Single Strand Tendons," Ad-Hoc Committee for Unbonded Single Strand Tendons, Post-Tensioning Institute, Phoenix, Arizona, October 1984.
16. Schupack, M., "Protecting Post-Tensioning Tendons in Concrete Structures," *Civil Engineering*, ASCE, December 1982, pp. 43-45.
17. MacGregor, J. C., "Safety and Limit State Design for Reinforced Concrete," *Canadian Journal of Civil Engineering*, V. 3, No. 4, 1976.

Note: The full text of Refs. 3, 4 and 7 can be obtained through the National Technical Information Service, Springfield, VA 22161.

* * *

APPENDIX — DESIGN EXAMPLE

Metric conversion factors: 1 in. = 2.54 cm; 1 ft = 0.305 m; 1000 psi = 6.894 MPa; 1 kip = 4.45 kN.

Throughout the design example, references to the provisions proposed in this paper are prefixed by a "P" for "Proposed Specification Provisions," while those cited from the current AASHTO Specification[8] are so designated.

Fig. A1 shows the major features of the bridge for which a post-tensioned bridge deck is to be designed. The span is 55 ft long and skewed approximately 21 degrees. The bridge deck is to be supported on five prestressed concrete girders. Diaphragms are provided only at the supports.

The use of an unbonded monostrand prestressing system of the transverse prestressing of this deck would require very close strand spacing. Not only is this inefficient, but the strand layout at the acute corners of the deck becomes cluttered. The shorter length tendons needed in the slab corners also make multistrand systems unattractive because of high seating losses. The most advantageous prestressing system for this deck is, therefore, high strength threaded bars. Insufficient room is available in a thin slab for two layers of threaded bar tendons, so middepth tendons will be used. Other prestressing systems could be used with minor changes.

Materials selected for this design include concrete with a compressive strength of 4 ksi, Grade 60 nonprestressed reinforcement, 1 in. diameter Grade 150 bonded threaded rods for transverse prestressing, and ½ in. diameter Grade 270 seven-wire extrusion coated unbonded strand for longitudinal prestressing of the deck.

The minimum deck thickness needed with middepth transverse tendons and longitudinal prestressing in the top of the slab is 8 in., as shown in Fig. A2. The 2½ in. concrete cover on top surface is required by P.1.11.3.

Slab Loads — The effective transverse slab span, S, is calculated as (AASHTO 3.24.1.2):

S = clear span

= girder spacing — girder top flange width

For a Texas Type 54 girder, the top flange is 16 in. wide.

$$S = 7.5 - 16/12 = 6.167 \text{ ft}$$

Dead load consists of the 8-in. slab and a 2-in. asphalt overlay. The uniform dead load per ft width on the deck, W_D, is then:

$$W_D = (0.150)\,8/12 + (0.140)\,2/12 = 0.123 \text{ kip/ft/ft}$$

The corresponding dead load moment, M_{DL}, for the continuous slab is:

$$M_{DL} = W_D S^2/10 = (0.123)(6.167)^2/10 = 0.469 \text{ kip-ft/ft}$$

The transverse slab live load moment per ft width of slab including impact, M_{LL+I}, is (AASHTO 3.24.3.1):

$$M_{LL+I} = (\text{Impact factor})\,[(S+2)/32]\,P\,(\text{Continuity factor})$$

where

Impact factor = 1.3 (AASHTO 3.8.2)

P = load on one rear wheel group of truck

= 16 kips for HS20 design loading (AASHTO 3.24.3)

Continuity factor = 0.8 (AASHTO 3.24.3.1)

$$M_{LL+I} = (1.3)\,[(6.167+2)/32]\,16\,(0.8) = 4.247 \text{ kip-ft/ft}$$

Total service load moment in the deck, M_S, is then:

$$M_S = M_{DL} + M_{LL+I} = 0.469 + 4.247 = 4.716 \text{ kip-ft/ft}$$

Transverse Prestress Design — The allowable extreme fiber concrete stresses, f_c and f_t, are:

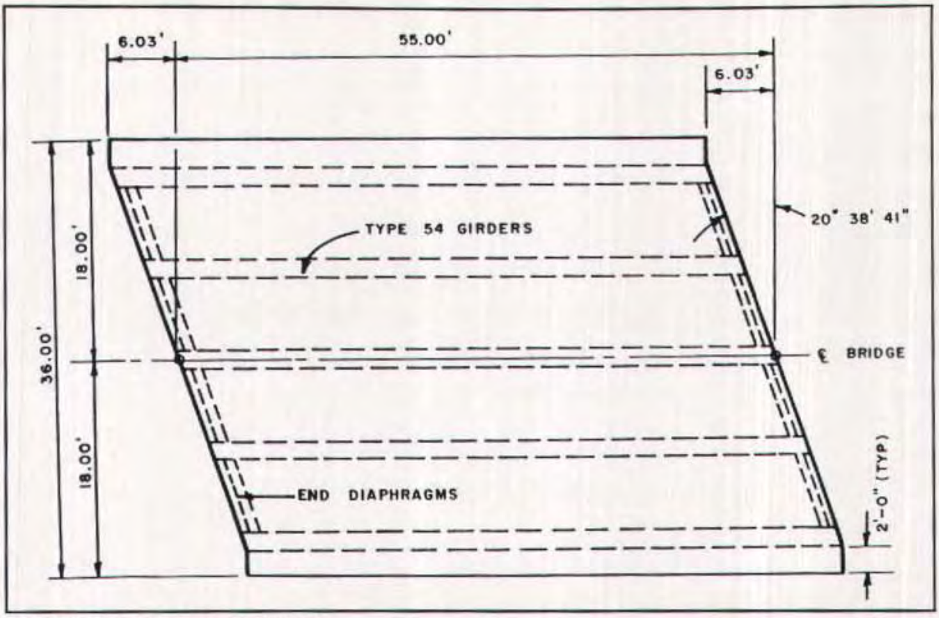

Fig. A1. Plan of example bridge.

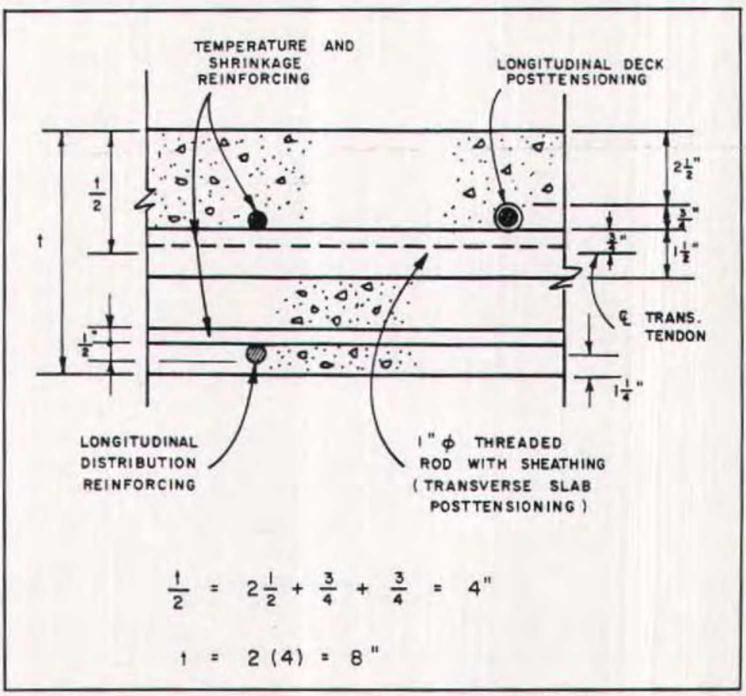

Fig. A2. Transverse section of deck showing determination of deck thickness for design example.

(a) Compression

$f_c = (0.4) f'_c$ (AASHTO 9.15.2.2)

$= (0.4)\,4 = 1.60$ ksi

(b) Tension

$f_t = 2\sqrt{f'_c}$ (P-1.5)

$= 2\sqrt{4000}/1000 = 0.126$ ksi

Cross section area, A, and section modulus, S', of a 1-ft wide strip of slab are calculated as:

$A = 8(12) = 96$ in.^{2}/ft

$S' = 2t^2 = 2(8)^2 = 128$ in.^{3}/ft

The required transverse prestress slab force per unit edge length, F_S, is then first found if governed by tension stresses.

$f_t = F_S/A - M_S/S'$

$-0.126 = F_S/96 - [(4.716)12]/128$

$F_S = 30.35$ kips per ft

Assuming tensile stress controls, check the compression stress:

$f = F_S/A + M_S/S'$

$= 30.35/96 + [(4.716)12]/128$

$= 0.758 \text{ ksi} < 1.6 \text{ ksi} = f_c$ (ok)

This deck will require amplified prestressing in the slab at the bridge ends to compensate for the restraint of the diaphragms. The required transverse slab prestress force per unit edge length in these areas, F_e, is found as:

$F_e = 1.2 F_S$ (P-1.3.2.4)

$= 1.2\,(30.35)$

$= 36.42$

This amount of transverse prestressing must be applied for an edge distance at each end of the bridge of x, where:

$x \geq W \tan\theta + 4 \leq (L + W \tan\theta)/N$ (P-1.3.2.4)

$W \tan\theta + 4 = 36 \tan(21^\circ) + 4$

$= 17.8$ ft

$(L + W \tan\theta)/N = [55 + 36 \tan(21^\circ)]/2$

$= 34.4$ ft

Use $x \geq 17.8$ ft

To determine the transverse tendon spacing, the effective force per tendon after all losses must be known. Initial stress for the threaded bars, T_o, is $(0.8) f'_s$ during jacking and should not exceed $(0.7) f'_s$ after seating of the anchorage, where f'_s is the ultimate strength of the prestressing steel (AASHTO 9.15.1).

If the tendon were jacked to $(0.8) f'_s$, the stress at the jacking end would be:

$T_o = 0.8\,(150) = 120$ ksi

For a straight tendon, the stress at the far end of the tendon, T_x, is calculated by:

$T_x = T_o e^{-(KL')}$ (AASHTO 9.16.1)

where

K = wobble coefficient

$= 0.0003$ (AASHTO 9.16.1)

L' = length of tendon, ft

$T_x = (120)\; e^{-[0.0003(36)]}$

$= 118.7$ ksi

This must be no greater, however, than $0.7 f'_s$.

$(0.7 f'_s = (0.7)\,150$

$105.0 \text{ ksi} < 118.7 \text{ ksi}$

Therefore, $T_x = 105.0$ ksi.

Prestress losses due to concrete creep and shrinkage and steel relaxation may be estimated as 22 ksi (AASHTO 9.16.2.2). Thus, the final effective stress in the prestressing tendon, T_e, is:

$T_e = T_x - 22 = 105 - 22 = 83.0$ ksi

A 1-in. diameter high strength threaded bar has a cross-sectional area of 0.85 sq in. The effective prestressing force per tendon, F_T, is then:

$F_T = T_e A_t = (83.0)\,0.85 = 70.555$ kips

where A_t is the area of prestressing steel for one tendon, in.^{2}

The spacing of the transverse tendons may now be calculated.

(a) Non-diaphragm regions:

Spacing $= F_T/F_S = 70.55/30.35$

$= 2.32$ ft $= 27.9$ in. (use 28 in.)

(b) Diaphragm regions:

Spacing $= F_T/F_e = 70.55/36.42$

$= 1.94$ ft $= 23.2$ in. (use 23 in.)

Maximum spacing of the tendons must be checked (P-1.4).

Spacing $\leq 8t = 8(8) = 64$ in.

≤ 5 ft $= 60$ in.

$\leq 3\,(y - a + 12)$

The distance from the slab edge to the inside face of the rail, y, is 12 in. From manufacturers' literature, the distance for a 1-in. diameter threaded rod with a plate anchorage is 8.25 in. Thus:

$\text{Spacing} \le 3(12 - 8.25 + 2) = 47.3$ in.

Since the maximum design spacing of 28 in. is less than 47 in., the design tendon spacings meet the requirements.

Concrete stresses have also been checked for the conditions at the time of initial tendon stressing and were within acceptable limits (AASHTO 9.15.2.1). This step, however, is omitted here for brevity.

Supplementary Bonded Reinforcement — Since the threaded bars will be grouted after stressing, supplementary bonded reinforcing is not required (P 1.6).

Ultimate Moment Check — The factored transverse slab moment, M_u, is calculated as (AASHTO 3.22):

$$M_u = (\text{load factor}) \times [(\text{dead load coefficient}) \times M_{DL} + (\text{live load coefficient}) \times M_{LL+I}]$$

$$M_u = 1.3[1.0\,(0.469) + 1.67\,(4.25)] = 9.83 \text{ kip-ft/ft}$$

Nominal flexural strength of the deck, M_n, is calculated by the formula (AASHTO 9.17.2):

$$M_n = A_s^* f_{su}^* d\,[1 - 0.6\,(p^* f_{su}^*)/f_c'\,]$$

where

- A_s^* = area of prestressing steel, in.^{2}
- d = distance from extreme compressive fiber to centroid of prestressing force, in.
- f_{su}^* = average stress in prestressing steel at ultimate load, ksi
- p^* = ratio of prestressing steel = A_s^*/bd where b is width of section

For bonded tendons:

$$f_{su}^* = f_s'\,(1 - 0.5\,(p^* f_s'\,)/f_c' \qquad \text{(AASHTO 9.17.4.1)}$$

The nondiaphragm area of the deck is critical for strength requirement since it has less prestressing steel. For a 1-ft wide strip of slab in the nondiaphragm region:

$$A_s^* = 12/28\,(0.85) = 0.364 \text{ in.}^2/\text{ft}$$

$$d = 4 \text{ in.};\ b = 12 \text{ in.}$$

$$p^* = A_s^*/bd = 0.364/[12(4)] = 0.00759$$

$$f_{su}^* = 150\{1 - 0.5[(0.00759)150]/4\} = (128.7 \text{ ksi}$$

$$M_n = (0.364)128.7(4)\{1 - 0.6[(0.00759)128.7]/4\} = 160.0 \text{ kip-in./ft } (= 13.33 \text{ kip-ft/ft})$$

$$M_n \ge M_u/\phi$$

For post-tensioned cast-in-place members, $\phi = 0.95$ (AASHTO 9.14):

$$13.33 \ge 9.83/0.95 = 10.35 \text{ kip-ft/ft (ok)}$$

Reinforcing Limits — The maximum steel allowed is such that:

$$p^* f_{su}^*/f_c' \le 0.30 \qquad \text{(AASHTO 9.18.1)}$$

$$(0.00759)\,128.7/4 = 0.24 < 0.3 \qquad \text{(ok)}$$

The minimum amount of reinforcement must be able to develop an ultimate flexural capacity of at least 1.2 times the cracking moment, M_{CR}, based on a tensile stress of $7.5\sqrt{f_c'}$ (AASHTO 9.18.2.1).

$$f = M_{CR}/S'$$

$$(7.5\sqrt{4000})/1000 = M_{CR}/128$$

$$M_{CR} = 60.72 \text{ kip-in./ft} = 5.06 \text{ kip-ft/ft}$$

$$\phi M_n \ge 1.2 M_{CR}$$

$$\phi M_n = (0.95)\,13.33 = 12.7 \text{ kip-ft/ft}$$

$$1.2 M_{CR} = 1.2\,(5.06) = 6.07 \text{ kip-ft/ft} \qquad \text{(ok)}$$

Distribution Reinforcement — Longitudinal distribution reinforcement in the bottom of the slab is taken as:

$$A_L \ge (0.03)\,t \qquad \text{(P-1.7.2)}$$

$$A_L \ge (0.03)\,8 = 0.24 \text{ in.}^2/\text{ft}$$

Spacing of these bars must be less than 12 in. (P 1.7.3).

Use #4 bars spacesd at 10 in. (A_L = 0.24 in.^{2}/ft).

Shrinkage and Temperature Reinforcement

Nonprestressed Reinforcement — By P-1.8.1, bonded nonprestressed steel will be needed in both directions in the top of the slab, and in the transverse direction in the bottom of the slab. Use #4 bars spaced at 18 in. (0.133 in.^{2}/ft) in all these locations.

Longitudinal Deck Prestressing — Since this deck is considered as exposed to a corrosive environment, and also to protect the concrete from freeze-thaw

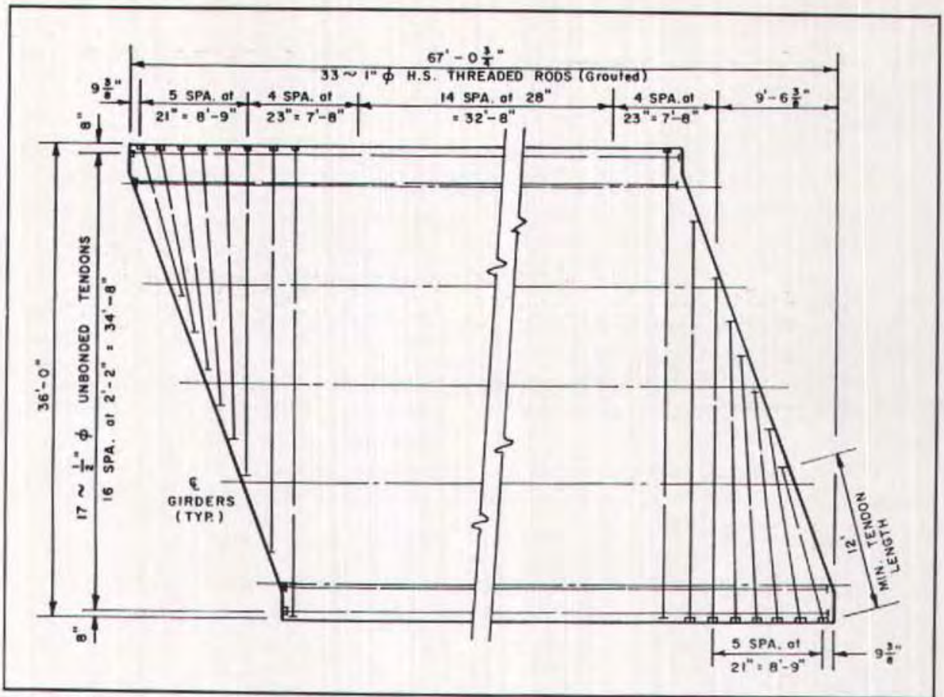

Fig. A3. Prestressing tendon layout in deck for design example.

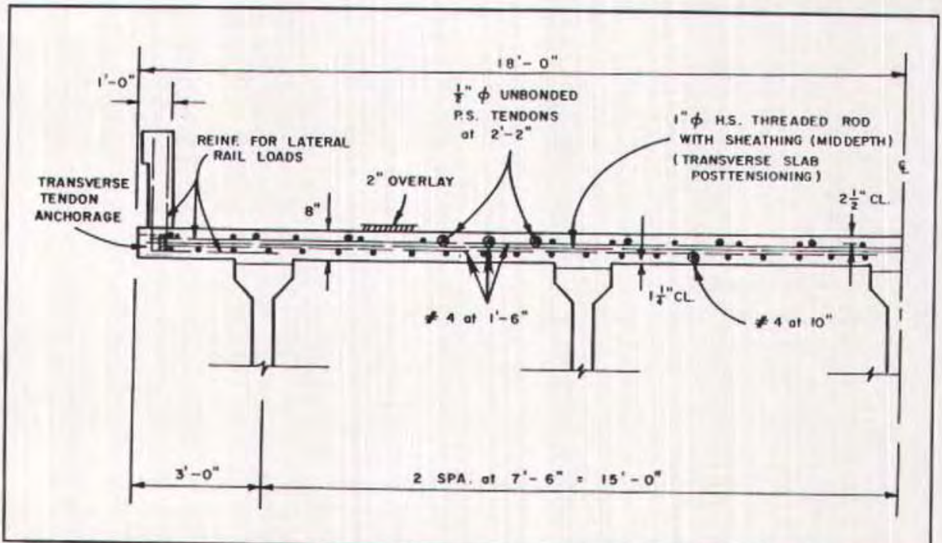

Fig. A4. Transverse section of deck for design example.

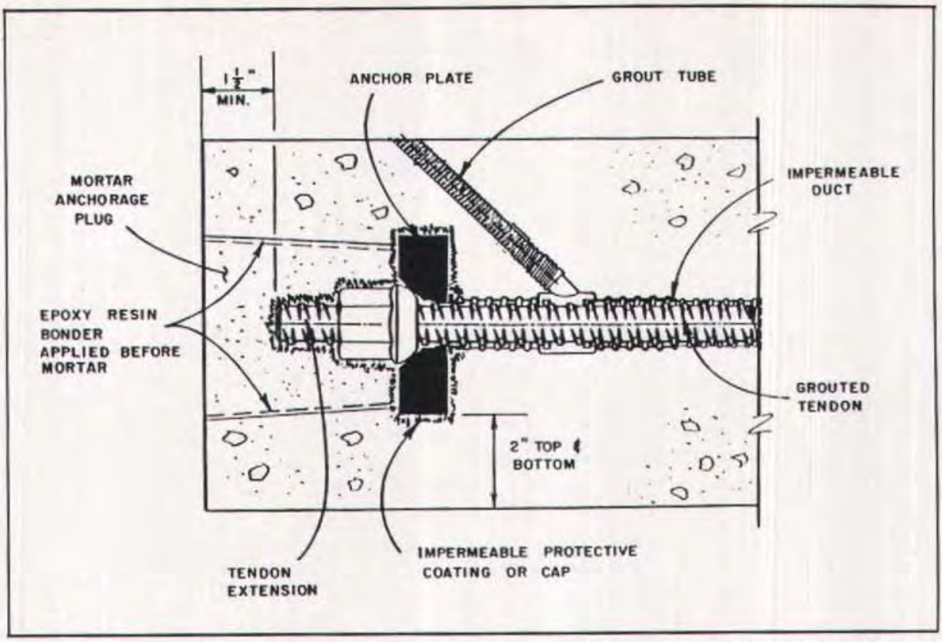

Fig. A5. Vertical section through slab showing special detailing requirements at tendon anchorages. As detailed, this would be an electrically isolated tendon which is a patented concept (see Ref. 16).

deterioration, it is desirable to prevent transverse cracking of the slab. Longitudinal prestressing of the deck will therefore be used. A minimum average compressive stress in the slab of 100 psi must be provided to counteract longitudinal tensile stresses (P-1.8.2).

Neglecting girder haunches and the difference in modulus of elasticity between the slab and girders, the *composite* section properties for one girder are found to be:

$A = 1213$ in.^{2}

$I = 476{,}500$ in.^{4}

$y_t = 17.19$ in.

$y_b = 44.81$ in.

Ignoring any compression in the slab due to composite dead loads, the longitudinal prestress force, P_L, required to obtain 100 psi at the slab middepth is determined in these calculations:

From Fig. A2, it can be seen that the center of the longitudinal prestressing is located 2.88 in. below the top of the slab. The eccentricity of the longitudinal prestressing tendon, e, is then:

$$e = y_t - 2.88 = 17.19 - 2.88 = 14.31 \text{ in.}$$

$$f = P_L/A + P_L \frac{ec}{I}$$

where c is the distance from the composite neutral axis to the center of the slab.

$$0.10 = P_L/1213 + [P_L\,(14.31)\,(17.19 - 4)/476{,}500]$$

$$P_L = 81.93 \text{ kips/girder}$$

$$= 81.93/7.5 = 10.92 \text{ kip/ft width}$$

The effective force per strand must be found to determine the tendon spacing. The maximum tendon stress at the jacking end during stressing is:

$T_o = (0.8) f_s' = (0.8)\,270 = 216$ ksi
and from AASHTO Section 9.16.1 the stress at the far end of the tendon is:

$T_x = T_o e^{-(KL')}$

where

K = 0.002 k/ft for extrusion coated strand

$T_x = (216) e^{-(0.002(55))} = 193.5$ ksi

But

$T_x \leq (0.7) f_s' = 0.7\,(270) = 189$ ksi

Since

$(0.7) f_s' < T_x$, let $T_x = 189$ ksi

Losses from all other sources are taken as 32 ksi (AASHTO 9.16.2.2), so that the final effective stress in the longitudinal tendons is:

$T_e = T_x - 32 = 189 - 32 = 157$ ksi

The effective prestress force per tendon for a ½-in. seven-wire strand with an area of 0.153 sq in. is then:

$F_T = T_e A_t = 157\,(0.153)$
$= 24.0$ kips

Spacing of the longitudinal tendons may now be calculated.

Spacing $= F_T / P_L = 24.0/10.92$
$= 2.20$ ft (= 24.4 in.)

Use 26 in. (17 strands total).

The effect of the longitudinal deck prestressing on the precast girders must be accounted for. The tensile stress in the bottom of the girder due to the longitudinal deck prestressing is calculated as:

$f = P_L/A - (P_L\, e\, y_b)/I$

$f = 81.93/1213 - [81.93(14.31)44.81]/476{,}500$

$= -0.043$ ksi

This additional tensile stress in the bottom of the precast girders may be easily accommodated by slightly lowering the pretensioned strand eccentricity, increasing the concrete strength, or adding two more pretensioned strands.

Final Details — Other details of the transversely prestressed deck design which must be addressed are the tendon layout at the skewed ends of the deck, reinforcing for lateral railing loads, and corrosion protection of the tendons and anchorages.

A fan tendon arrangement will be used at the acute corners of the deck. The spacing of the tendons on skew must be reduced by the cosine of the skew angle. In this instance, the reduced spacing is found as:

Fan tendon spacing = (diaphragm region spacing) $\cos\theta$
$= 23 \cos(21°)$
$= 21.5$ in.

Use 21 in. as shown in Fig. A3.

By P-1.10, nonprestressed reinforcement will have to be provided at the slab longitudinal edges to resist the moment from lateral railing loads. Since the calculations required to find the amount of reinforcement needed are the same as for a conventional slab, they will be omitted here.

Corrosion protection of the prestressing tendons and anchorages is required by P 1.11. A detail for these protective measures is shown together with a plan and section view of the transversely prestressed deck design in Figs. A3 through A5.

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NOTE: Discussion of this paper is invited. Please submit your comments to PCI Headquarters by June 1, 1990.

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