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DESIGN PROCEDURES FOR PRESTRESSED CONCRETE BRIDGE DECKS

By

R.W. Poston, A.R. Phipps, J.E. Breen,
and R.L. Carrasquillo

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16. Abstract

The objective of this study is to investigate the use of deck prestressing as a method of improving durability of bridge decks with the aim of developing suggested AASHTO Specification design requirements to facilitate its use.

In previous reports in this series, the idea of improving durability of bridge decks by prestressing was introduced as a design concept. The results from an experimental test series of prestressed concrete specimens subjected to an aggressive deicing salt exposure and from an experimental and analytical investigation of the structural behavior of a fully composite slab-girder bridge model indicated that deck prestressing is a viable corrosion-protection design alternative. It has already been used in numerous box girder bridges to improve structural efficiency.

Specific recommendations for improving durability of bridge decks by deck prestressing and for proper design of deck prestressing are included in this report. These recommendations are presented in a form suitable for inclusion in the AASHTO Bridge Specifications. Several design examples are included to illustrate the design recommendations and to compare costs to current bridge deck designs using conventional reinforced concrete.

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"The Application of Transverse Prestressing to Bridge Decks"

Conducted for

Texas

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CENTER FOR TRANSPORTATION RESEARCH
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There was no invention or discovery conceived or first actually reduced to practice in the course of or under this contract, including any art, method, process, machine, manufacture, design of composition of matter, or any new and useful improvement thereof, or any variety of plant which is or may be patentable under the patent laws of the United States of America or any foreign country.

PREFACE

This report is the third and final report in a series which summarizes a detailed evaluation of the use of prestressing in bridge decks to improve durability and structural efficiency. The first report summarized an experimental investigation of the behavior of prestressed concrete deck specimens subjected to an aggressive deicing salt exposure. The second report summarized the experimental and analytical investigation of the structural behavior of prestressed concrete bridge decks. This report draws on the experimental and analytical results presented in both earlier reports to develop suggested AASHTO Bridge Design Specification requirements for the design of durable prestressed concrete bridge decks. This report also contains several examples to illustrate the application of the design provisions for prestressed decks and to compare costs to current bridge deck designs using conventionally reinforced concrete.

This work is part of Research Project 3-5-80-316, entitled "The Application of Transverse Prestressing to Bridge Decks." The research was conducted at the Phil M. Ferguson Structural Engineering Laboratory as part of the overall research program of the Center for Transportation Research of The University of Texas at Austin. The work was sponsored jointly by the Texas State Department of Highways and Public Transportation and the Federal Highway Administration under an agreement with The University of Texas at Austin and the State Department of Highways and Public Transportation.

Liaison with the State Department of Highways and Public Transportation was maintained through the contact representative, Mr. James C. Wall; the Area IV Committee Chairman, Mr. Robert L. Reed; and the State Bridge Engineer, Mr. Wayne Henneberger. Mr. J. W. Bowman was the contact representative for the Federal Highway Administration.

The overall study was co-directed by Dr. John E. Breen, who holds the Nassar I. Al-Rashid Chair in Civil Engineering, and Dr. Ramon F. Carrasquillo, who is an Associate Professor of Civil Engineering. The project was under the immediate supervision of Dr. Randall W. Poston, Research Engineer. He was assisted by Dr. Riyadh A. Almustafa, Mr. Rafael Mora, Mr. Alan R. Phipps, and Mrs. Mary Lou Ralls, Assistant Research Engineers.

SUMMARY

The objective of this study is to investigate the use of deck prestressing as a method of improving durability of bridge decks with the aim of developing suggested AASHTO Specification design requirements to facilitate its use.

In previous reports in this series, the idea of improving durability of bridge decks by prestressing was introduced as a design concept. The results from an experimental test series of prestressed concrete specimens subjected to an aggressive deicing salt exposure and from an experimental and analytical investigation of the structural behavior of a fully composite slab-girder bridge model indicated that deck prestressing is a viable corrosion-protection design alternative. It has already been used in numerous box girder bridges to improve structural efficiency.

Specific recommendations for improving durability of bridge decks by deck prestressing and for proper design of deck prestressing are included in this report. These recommendations are presented in a form suitable for inclusion in the AASHTO Bridge Specifications. Several design examples are included to illustrate the design recommendations and to compare costs to current bridge deck designs using conventional reinforced concrete.

IMPLEMENTATION

This report is the final in a series which summarizes a major experimental and analytical investigation of the use of deck prestressing as a method of improving durability and structural efficiency of bridge decks. The detailed recommendations for design of durable prestressed concrete bridge decks in a form suitable for inclusion in the AASHTO Bridge Specification are included in this report.

This report contains background information of interest both to those responsible for deciding on specifications and codes and to bridge designers. In addition, it contains detailed design examples of the application of prestressing to bridge decks. These examples should aid engineers in the design of prestressed concrete bridge decks.

This report shows deck prestressing to be a cost-competitive solution for durable bridge decks. In most cases with slab-girder bridges the initial costs of a prestressed deck will be slightly higher than for a conventionally reinforced concrete deck. On the other hand, for most box girder bridges deck prestressing can reduce construction costs. In either case, however, the long-term benefits of its proper use in terms of improved durability and decreased maintenance costs should be substantial.

C O N T E N T S

T A B L E S

CHAPTER 1

INTRODUCTION

1.1 General

The accelerated deterioration of bridge decks has become a critical problem for design engineers and maintenance forces over the last two decades. Bridge decks designed for a service life of 40 years are requiring major maintenance 5 to 10 years after construction; and often times total deck replacement is necessary after only 15 years of service.

Deterioration of concrete bridge decks occurs by two basic mechanisms. The first mechanism involves moisture which penetrates concrete through surface cracks and subsequently freezes. Large expansive pressures are generated within the crack, widening it even further. Later more water can seep into this widened crack and freeze. This progressive cycle results in surface spalling of the concrete deck. The second mechanism involves corrosion of embedded steel reinforcement in bridge decks. While reinforcing steel is normally passive in the highly alkaline concrete environment, the presence of chloride from deicing salts or marine spray combined with moisture and oxygen will cause steel to corrode rapidly. Since corrosion products occupy 2 to 15 times the volume of the original steel, large expansive pressures develop within the concrete. This leads to cracking, spalling and delamination of the deck.

The use of low quality, poorly compacted, permeable concrete or inadequate concrete cover over steel reinforcement will allow moisture and chlorides to penetrate over some time, thus initiating steel corrosion and concrete deterioration. However, even with high quality concrete and substantial covers, penetration of aggressive corrosion-producing substances is facilitated by cracking in the concrete slab. Moreover, such cracking is very likely in a conventional reinforced concrete deck under normal service loads.

1.2 Problem Statement

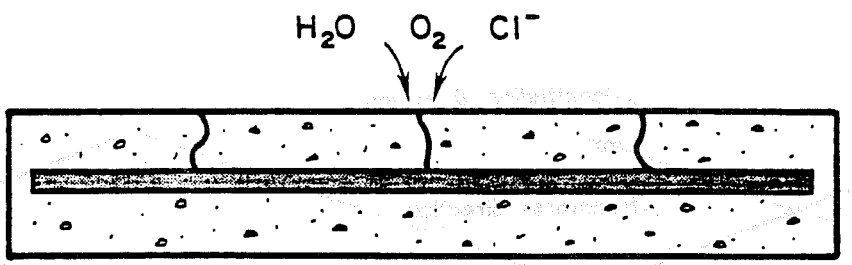
Numerous techniques have been proposed to extend the service life of concrete bridge decks. These include the use of: higher quality concrete, improved compaction, greater concrete cover over the reinforcing steel, polymerized concrete, waterproofing membranes, coated reinforcing steel, active cathodic protection of reinforcing, and prestressed concrete [1]. Special concretes, deck surface coatings, and increased concrete cover all help prevent infiltration of moisture and chlorides into the deck through the concrete to the level

of the reinforcing. Since corrosion producing elements can penetrate uncracked concrete with insufficient cover, concrete quality or compaction, it is assumed throughout this report that all normal precautions involving provision of adequate cover and concrete quality will be observed. Even in such as well designed and constructed conventional slabs, they are not effective wherever the concrete has cracked. Reinforcing coatings and cathodic protection inhibit corrosion of the reinforcing steel even when chlorides, water, and oxygen are present. However, they can be expensive and furthermore offer no protection against freeze-thaw damage where water has seeped into concrete cracks. Thus, the basic weakness of most of the proposed methods of protection for concrete decks is the cracking of the concrete which must occur in a conventionally reinforced deck in order for substantial moment resistance to be mobilized.

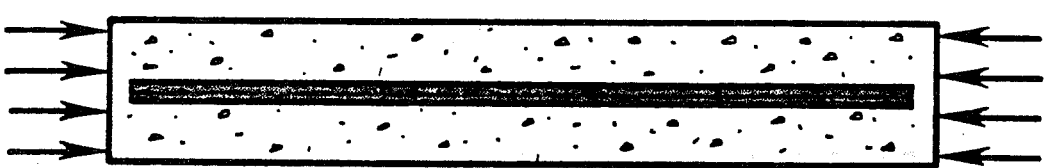
In contrast to a conventional reinforced concrete slab, an adequately designed prestressed slab will not crack under applied service loads. Thus, penetration of the corrosion-producing elements through channels provided by cracks is prevented (Fig. 1.1). Even if cracking should occur in a prestressed concrete member, the cracks will close once the load is removed. A main application of this principle to bridge decks would be prestressing the deck in the transverse direction, as illustrated in Fig. 1.2. However, it seems logical that if the basic philosophy is to control crack formation and crack width by active prestressing, a low level of prestressing of the deck in the longitudinal direction is also desirable to prevent possible transversely oriented slab cracking. 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.

Most of the corrosion protection alternatives which have been proposed are costly additions to the basic deck structure. Transverse prestressing, on the other hand, is an important active structural system which also enhances strength and thus offers potential economic advantages. Prestressed concrete utilizes materials more efficiently than reinforced concrete and may require a thinner concrete section or less reinforcement to carry the design loads, thereby saving materials and reducing superstructure weight. In many cases, the higher material costs associated with prestressing steel and higher strength concrete, together with the extra labor required for prestressing operations, should be more than offset by the structural efficiency and durability of transversely prestressed bridge decks.

Although the use of deck prestressing in bridges appears beneficial, there are very few documented studies and observations of such deck systems [2,3,4,5] to serve as a basis for implementation of the concept. Furthermore, the current AASHTO Design Specification [6] provisions for prestressed concrete were basically developed for



a) conventional



b) prestressed

Fig. 1.1 Corrosion protection mechanism of prestressing

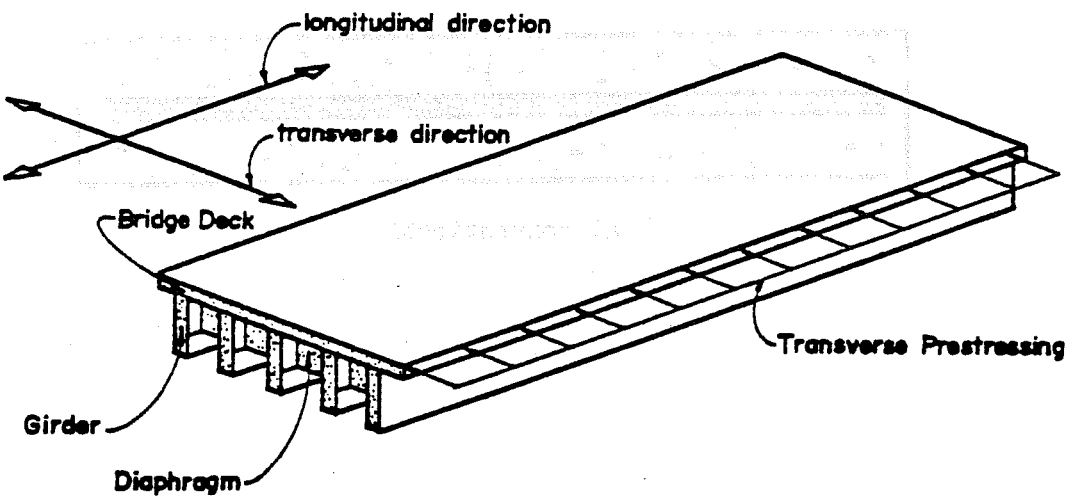


Fig. 1.2 Transverse prestressing of a conventional slab-girder bridge

longitudinal prestressing, and offer little guidance for many important aspects of the design of transversely prestressed bridge decks. Such aspects include the distribution of prestress across the slab and the durability requirements of the system. For instance, the specifications are silent on how girder or web stiffness, transverse diaphragms, and bridge skew angle affect the actual distribution of the transverse compressive stresses applied along the edge of the bridge slab. Similarly, no guidance is given as to the allowable concrete tensile stresses for precompressed tension zones if one considers the possibility of corrosion-inducing solutions penetrating the thin zone of concrete over top deck reinforcement. The influence of the greater stiffness of a prestressed slab on the behavior of the deck under concentrated wheel loads is not defined.

1.3 Objectives and Scope of the Study

The present study was undertaken to address some of these questions regarding the design and behavior of prestressed concrete bridge decks. Specifically, the principal objectives of the study were to:

1. Study the effect of major variables on corrosion protection in concrete slabs.

2. Evaluate the structural behavior and restraint effects in prestressed bridge decks.

3. Formulate design recommendations for the economic application of deck prestressing considering the interrelationship between structural and durability aspects.

The early focus of this study was principally on transverse prestressing. As the study progressed the importance of two way prestressing became apparent and longitudinal deck prestressing was incorporated.

Report 316-1 summarizes an experimental investigation to study the effect of deck prestressing on chloride-induced corrosion. Report 316-2 summarizes an experimental and analytical investigation of the structural behavior of transversely prestressed bridge decks. In this report the findings from the structural and durability studies are translated into specific design recommendations and draft AASHTO Specification requirements. Design applications and cost comparisons using these proposed recommendations are then presented.

CHAPTER 2

DESIGN OF BRIDGE DECK PRESTRESSING

2.1 Introduction

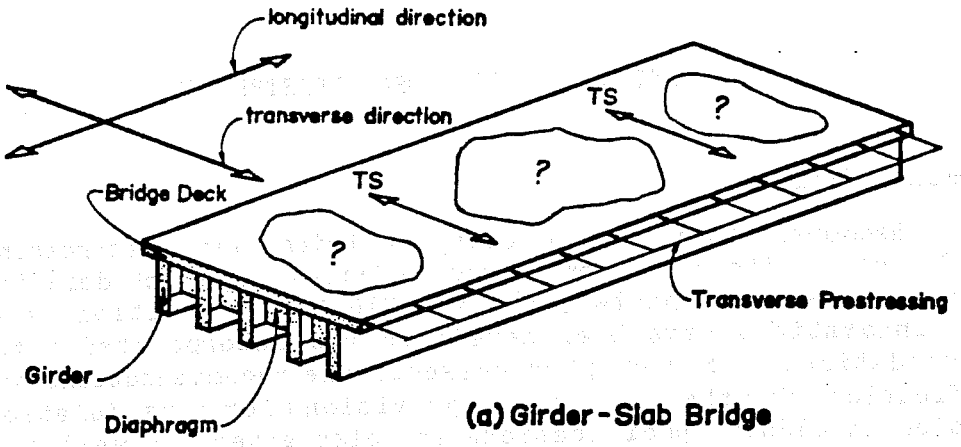
Recommendations for the design of bridge deck prestressing are based largely on the experimental and analytical research described in Companion Research Reports 316-1 and 316-2. In addition, several design innovations have been developed and incorporated into the recommendations. This chapter presents the recommendations and the justifications for them. Proposed provisions for consideration for inclusion in AASHTO Specifications are also given, as well as some direction for innovation in the design of bridge decks.

2.2 Structural Considerations

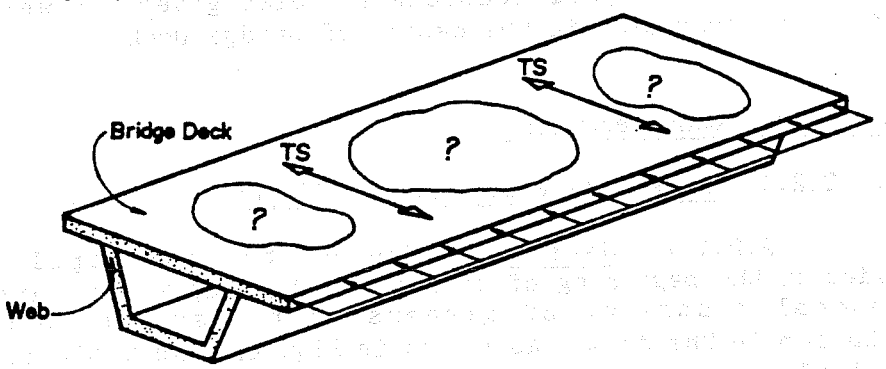
2.2.1 Transverse Prestressing Effects

2.2.1.1 General. One of the principal concerns identified at the beginning of this research study was the influence of the lateral restraint of girders on transverse prestress distribution in the deck. As shown in Fig. 2.1 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 Research Report 316-2 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 neoprone pads. In box girders and in slab-girder bridges with fixed support conditions for the girders there is a restraint problem which needs to be considered. However, for slab-girder bridges current Texas practice is to almost exclusively use flexible neoprene bearings, with occasional use of steel rocker bearings. 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.

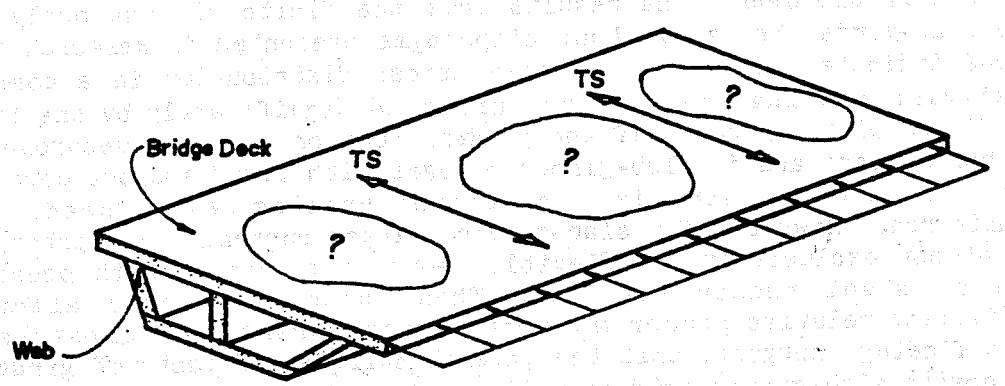
In contrast to girder restraint considerations in slab-girder bridges the analytical and experimental results presented in Research Report 316-2 clearly indicate that there are significant reductions in transverse slab prestress in both slab-girder bridges and box girder bridges because of the presence of diaphragms. Therefore, the effect of diaphragms on the prestress distribution in a transversely prestressed bridge deck must be considered in design. Finite element



(a) Girder-Slab Bridge



(b) 1-Cell Box Girder Bridge



(c) 2-Cell Box Girder Bridge

Fig. 2.1 Regions of Uncertainty of Transverse Prestressed (TS) Distribution

programs such as those used in the present research study would be both cumbersome and relatively expensive for use in design. In sections which follow, simplified procedures are proposed for determining the forces required to compensate for the restraining effects of diaphragms and thereby ensure adequate transverse prestressing throughout a bridge deck slab.

For practical design considerations there are two methods which can be used to compensate for diaphragm restraining effects. One method involves prestressing the diaphragms themselves. Prestressing the diaphragms with a supplementary force equal to the force attracted by them due to transverse prestressing of the deck would permit approximately equal shortening in the slab and diaphragms. Consequently, the deck transverse prestress distribution would be relatively unaffected by the diaphragms. The design procedure which is developed for determining the prestress force required in the diaphragms to compensate for restraining effects follows a rationale similar to that used for determining development length of reinforcing bars in the current ACI Building Code [7]. In that procedure a basic development length is modified by multiplicative factors to account for the influence of various conditions in order to arrive at a design embedment length. When this approach is used to develop a transverse prestress design procedure, the study of variables suggests the approach as shown by Eq. (2.1) for determining the prestress force required in the diaphragms to account for diaphragm restraint effects:

$$P_D = C_t\, C_K\, C_L\, C_{SK}\, 1.6\, F_S \qquad (2.1)$$

where P_D = prestress force required in a diaphragm to account for restraining effects,

C_t, C_K, C_L, C_{SK} = factors to account for various parameters such as slab thickness, diaphragm stiffness, bridge skew angle, and diaphragm spacing,

1.6 = constant to account for basic effect of diaphragms (unit of length is ft)

and F_S = transverse prestress force per unit edge length applied to deck slab to resist effects of imposed loading assuming no diaphragm restraining effects.

(Note: in all dimensional equations in this report the units of length are to be taken as feet or per foot)

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. This approach will be further developed in later sections.

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.

This translates into equation form as given by Eq. (2.2):

$$F_e = f\, F_S \tag{2.2}$$

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

f = amplification factor greater than 1 which is dependent on the bridge skew angle,

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

In Section 2.2.1.3.3 which follows, this design approach will be developed, and recommendations will be given as to what regions of a slab will require amplified transverse prestressing to compensate for diaphragm effects.

Although the primary focus of this research study was slab-girder bridges, design recommendations for transverse prestressing effects in box-girder bridges will also be addressed. The supporting basis for the design recommendations of box-girder bridges is found in Report 316-2 and in Ref. 8. The design approach for box-girder bridges is somewhat more complex than for slab-girder bridges. It requires in most cases that both suggested methods be used; i.e., amplified slab prestress force is required to compensate for restraining effects of the webs and a prestressing force producing transverse shortening consistent with the deck prestressing is also required to be applied to the diaphragms.

2.2.1.2 Box-Girder Bridges. Based on Report 316-2 it is concluded that if fully effective transverse prestressing is to be present in all regions of the deck of box-girder bridges, it is necessary to eliminate or overcome any interaction between the girder

webs and diaphragms and the transverse prestressing of the top slab. Since the diaphragm may provide substantial local support to the deck, such fully effective transverse prestressing may not be necessary in the diaphragm regions. In some forms of precast box girders the diaphragm effect can be limited to the segment with the diaphragm by transversely prestressing the other segments before attaching them to the segment with the diaphragm. However, in general, in box-girder bridges the results clearly show that it is impractical to account for the diaphragm restraining effects with amplified slab transverse prestressing alone. The only practical method to compensate for diaphragm restraining effects is to also prestress the diaphragms. Therefore, where it is necessary to counter diaphragm restraining effects in box-girder bridges, it will be necessary to apply substantial diaphragm prestress force to produce transverse shortening consistent with the level of transverse prestressing applied to the top slab.

The results also indicate that there are some other relatively smaller restraining effects in box girders which must be compensated for in design. In particular, the lateral restraining effects between interaction of the webs on the transverse shortening of the top slab must be accounted for. For a wide range of parameters including slab thickness, web inclination (including vertical as well as inclined webs), web thickness, and strand profile, simple approximate design recommendations for top slab transverse prestressing effects in one-, two- and three-cell box girder sections can be made in lieu of the more general analysis procedures of Report 316-2. The amplified transverse prestressing force per unit edge length, F_e, required to compensate for web restraining effects is given by the following equations for various box-girder sections:

One-cell section:

$$F_e = 1.1\ F_S \tag{2.3}$$

Two-cell section:

$$F_e = 1.15\ F_S \tag{2.4}$$

Three-cell section:

$$F_e = 1.4\ F_S \tag{2.5}$$

where F_e = amplified transverse slab prestress force per unit edge length in order to compensate for web restraining effects,

and F_S = transverse slab prestressing force per unit edge length required to resist effects of structural loads assuming no web restraining effects.

This amplified transverse prestressing force will be required along the entire length of the box-girder bridge. Since the box girder section behaves as a rigid frame in the transverse direction, the effect of such applied forces on transverse moments in the webs and soffits must be considered.

2.2.1.3 Slab-Girder Bridges. The 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.

2.2.1.3.1 Bridge with No Diaphragms. For a nonskew or skew bridge which will not include diaphragms, or in which the diaphragms will not be present at the time of transverse prestressing, for design purposes the transverse prestress distribution can be assumed to be equal to the applied edge prestress less appropriate friction losses and time effects.

2.2.1.3.2 Compensating for Diaphragm Restraining Effects by Prestressing Diaphragms. The basic diaphragm prestress force required to compensate for the diaphragm restraining effects is given by Eq. (2.6):

$$P_{Db} = 1.6\ F_S \qquad (2.6)$$

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.),

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

Equation (2.6) represents the basic diaphragm restraining effects in Eq. (2.1)

$$P_D = C_t C_K C_L C_{SK}\ 1.6\ F_S \qquad (2.1)$$

$$= C_t C_K C_L C_{SK}\ P_{Db}$$

before correction factors are applied to account for various parameters.

To illustrate Eq. (2.6), if the design transverse prestress is 200 psi in an 8 in. slab, then the transverse slab prestress force per unit edge length, F_S, would be 19,200 lbs per foot, and the diaphragm force required to compensate for restraining effects would be 1.6 times 19,200 lbs, which is 30,720 lbs. This basic equation (Eq. 2.6) is applicable for both end and interior diaphragms. For this basic equation, the bridge slab thickness is assumed to be 8 in., the bridge skew 0 degrees, the diaphragm spacing 25 ft, and the diaphragm stiffness corresponds to that of the standard concrete diaphragms shown in Fig. 2.2. Modifications to this basic equation which account for slab thickness, diaphragm stiffness, bridge length (i.e., spacing between diaphragms), and bridge skew will be proposed in sections which follow.

Table 2.1 presents comparisons between the diaphragm prestress force required to compensate for diaphragm effects determined by Eq. (2.6) and that determined by 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. 2.6) is modified for the effect of slab thickness, as:

$$P_D = C_t \; 1.6 \; F_S \qquad (2.7)$$

where C_t = correction factor for slab thickness.

The proposed slab thickness correction factor is:

$$C_t = 8/t \qquad (2.8)$$

where t = slab thickness, in.

Table 2.2 presents a comparison between the diaphragm prestress force required to overcome diaphragm restraining effects predicted by Eq. (2.7) 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

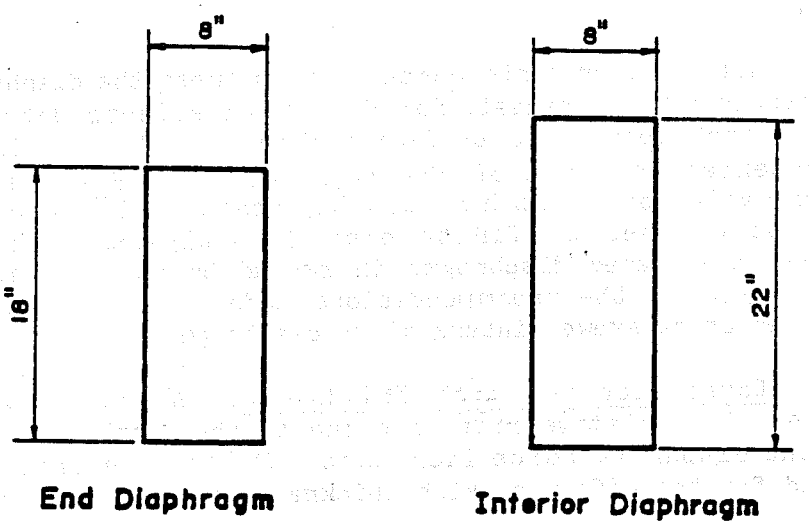


Fig. 2.2 Standard Concrete Diaphragms

TABLE 2.1 Comparison Between Diaphragm Prestress Force Required to Compensate for Diaphragm Restraining Effects Determined by Proposed Basic Equation and that Computed by Finite Element Analysis

Strand Profile	Diaphragm Case	P_D/F_S End Diaphragm		P_D/F_S Interior Diaphragm	
		Finite Element Analysis	Proposed (Eq. 2.6)	Finite Element Analysis	Proposed (Eq. 2.6)
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
Slab thickness = 8 in.
Standard concrete diaphragms (see Fig.2.1)
Skew angle = 0 degrees

TABLE 2.2 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 Thickness (in)	Diaphragm Case	P_D/F_S End Diaphragm		P_D/F_S Interior Diaphragm	
		Finite Element Analysis	Proposed (Eq. 2.7)	Finite Element Analysis	Proposed (Eq. 2.7)
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
Standard concrete diaphragms (see Fig. 2.1)
Skew angle = 0 degrees
Straight strand profile

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 similar to those shown in Fig. 2.2 for use on prestressed concrete girder bridges. For this case, the basic equation (Eq. (2.6)) does not need modification. However, if nonstandard concrete diaphragms or if steel diaphragms such as those used for steel girder bridges are called for in design, the following modification to the basic ratio is proposed:

$$P_D = C_K \, 1.6 \, F_S \qquad (2.9)$$

where C_K = correction factor for diaphragm stiffness.

The correction factor for diaphragm stiffness is defined as follows:

$$C_K = (EA)_D \, / \, 640{,}000 \qquad (2.10)$$

where E = modulus of elasticity of the diaphragm material, ksi,

and A = effective diaphragm cross-sectional area resisting axial deformations, $in.^2$

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

Table 2.3 presents a comparison between the diaphragm prestress force required to overcome diaphragm restraining effects predicted by Eq. (2.9) and that predicted by finite element analysis for varying diaphragm stiffness. The comparisons are 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, one line of interior diaphragm at midspan is used. From 55 ft to 95 ft, two lines of interior diaphragms at third points are used. For bridge lengths greater than 95 ft, three diaphragm lines at quarter points are used. Thus, the spacing between diaphragms in bridges which include interior diaphragms varies from about 18 ft to 32 ft. As the distance between interior diaphragms decreases, the restraining force in both end and

TABLE 2.3 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) ($k\text{-}in.^2/in.^2$)	Strand Profile	Diaphragm Case	P_D/F_S End Diaphragm		P_D/F_S Interior Diaphragm	
			Finite Element Analysis	Proposed (Eq. 2.9)	Finite Element Analysis	Proposed (Eq. 2.9)
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, see Fig. 2.2)	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
Slab thickness = 8 in.
Skew angle = 0 degrees
* Concrete modulus assumed = 4000 ksi

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:

$$P_D = C_L \ 1.6 \ F_S \tag{2.11}$$

where C_L = 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:

$$C_L = 25/S_D \tag{2.12}$$

where S_D = spacing between interior diaphragms or between end and interior diaphragms ft.

Table 2.4 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. (2.11) 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 0 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, to take advantage of the reduction in diaphragm prestress force required to overcome restraining effects, the following correction is proposed:

$$P_D = C_{SK} \ 1.6 \ F_S \tag{2.13}$$

where C_{SK} = correction factor for effect of bridge skew.

The C_{SK} factor is defined as:

$$C_{SK} = \cos \theta \geq 0.75 \tag{2.14}$$

TABLE 2.4 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. 2.11)	Finite Element Analysis	Proposed (Eq. 2.11)
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.
Standard concrete diaphragms (see Fig. 2.1)
Skew angle = 0 degrees
Straight strand profile

where 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.

The limit of 0.75 corresponds to a skew angle of about 40 degrees which the finite element analysis revealed was the angle at which no further reductions in diaphragm force resulted.

Table 2.5 presents a comparison between the basic diaphragm prestress force equation modified for skew angle and that computed by finite element analysis. The proposed correction for skew angle effects roughly approximates that obtained by finite element analysis. Again, the agreement for the interior diaphragm case is very good.

However, detailing considerations suggest that prestressing diaphragms on a skew bridge is probably not practical. Therefore, the factor C_{SK} will not be included in the actual design recommendation.

Multiple Corrections. As indicated in the introduction to Section 2.2.1.1, it is proposed that the correction factors be multiplied as illustrated by Eq. (2.15) for multiple corrections to the basic equation:

$$P_D = C_t C_K C_L C_{SK}\ 1.6\ F_S \qquad (2.15)$$

A parametric study with a mix of variables revealed that Eq. (2.15) could be as much as 20% unconservative if more than two of the correction factors used had a value less than 1. Therefore, in using Eq. (2.15) no more than two correction factors with values less than one can be used. However, the two lowest correction factors may be used. Table 2.6 compares the results obtained for Eq. (2.15) 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 assume 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 that of the superstructure. Regardless of the height of the diaphragm, the prestressing force should never be located such tha it may induce tension stress in the diaphragm.

Taking into consideration the above constraints, a reasonable allowable eccentricity of the diaphragm prestress force is 1/2 the

TABLE 2.5 Comparison Between Diaphragm Prestress Force Required to Compensate for Diaphragm Restraining Effects Determined by Proposed Basic Equation Modified for Bridge Skew Angle and that Computed by Finite Element Analysis

Skew Angle (deg)	Diaphragm Case	P_D/F_S End Diaphragm		P_D/F_S Interior Diaphragm	
		Finite Element Analysis	Proposed (Eq. 2.13)	Finite Element Analysis	Proposed (Eq. 2.13)
0	All	1.4	1.6	1.55	1.6
0	End Only	1.4	1.6	—	—
20	All	1.35	1.50	1.45	1.50
20	End Only	1.35	1.50	—	—
40*	All	1.19	1.2	1.26	1.2
40*	End Only	1.19	1.2	—	—
60*	All	1.32	1.2	1.39	1.2
60*	End Only	1.32	1.2	—	—

* From two-dimensional finite element analysis

Assumptions for Comparison

Slab thickness = 8 in.
Standard concrete diaphragms (see Fig. 2.1)
Diaphragm spacing = 25 ft
Straight strand profile

TABLE 2.6 Comparison Between Diaphragm Prestress Force Calculated by Eq.(2.15) and That Determined by Finite Element Analysis for a Mix of Variables

Case	Bridge Variables	Correction Factors	Proposed Eq. (2.15)	Finite Element Analysis
1	Bridge length = 76 ft Diaphragm spacing = 25 ft θ = 40 deg. Slab thickness = 10 in. $(EA)_D$ = 320,000 k-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 deg. Slab thickness = 6.5 in. $(EA)_D$ = 960,000 k-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 Diaphragm spacing = 25 ft θ = 20 deg. Slab thickness = 7 in. $(EA)_D$ = 640,000 k-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 deg. Slab thickness = 9 in. $(EA)_D$ = 640,000 k-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

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 how this affects the stresses in the bridge deck should be carried out.

2.2.1.3.3 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 region around the diaphragms was a viable method to overcome the restraining effects of the diaphragms. In the case of the model bridge, 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 be 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. (2.16) is proposed:

$$F_e = 1.6\ F_S \tag{2.16}$$

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,

and F_S = transverse 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 centered on the diaphragms. For bridges with greater than 10 degree skew, Eq. (2.17) is proposed:

$$F_e = 1.2\ F_S \tag{2.17}$$

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 edge strip used with nonskew bridges.

The slab edge length over which this amplified prestressing force must be applied is given by Eq. (2.18):

$$x = W \tan\theta + 4 \leq (L + W \tan\theta)/N \tag{2.18}$$

where x = slab edge length at diaphragms over which F_e will be required, ft,

W = width of bridge slab, ft,

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,

L = span length, ft,

and N = number of diaphragm lines per span (i.e., 4 for a span with 2 sets of interior diaphragms, and 2 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 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 wide, which was the equivalent distance used for the laboratory bridge model. Figures 2.3 and 2.4 show diaphragm amplified prestress regions for a bridge with no skew and with skew, respectively.

To examine the applicability of Eqs. (2.16) through (2.18) finite element analysis was used to examine the prestress distributions of the prototype bridge of Research Report 316-2 for skew angles varying from 0 to 60 degrees. Table 2.7 summarizes the various cases analyzed and the width of the diaphragm regions over which F_e was applied. Figures 2.5 through 2.9 present stress contours for the various cases analyzed in Table 2.7. 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 stresses equal to the stress induced by F_S. This is clearly not possible in practice. However, in all cases, the results presented in Figs. 2.5 through 2.9 indicate that a substantial portion of the deck area is between 95% and 120%, which suggests a reasonably uniform prestress distribution. There are a few "hot" spots up to 150% but this is not a problem with the low levels of slab prestress usually used. Thus, the use of Eqs. (2.16) through (2.19) resulted in reasonable, yet generally conservative slab prestress distributions for the wide range of skew angles examined for the study bridge.

Several other cases shown in Table 2.8 were also examined to study the applicability of Eqs. (2.16) through (2.18) for bridges with a mix of parameters. The cases in Table 2.8 were selected to represent the range of possible parameters which might affect the prestress distributions. The results from the finite element analyses are presented in Figs. 2.10 through 2.13. As before, the stress contours are all greater than 95% of the edge stress produced by F_S, and a

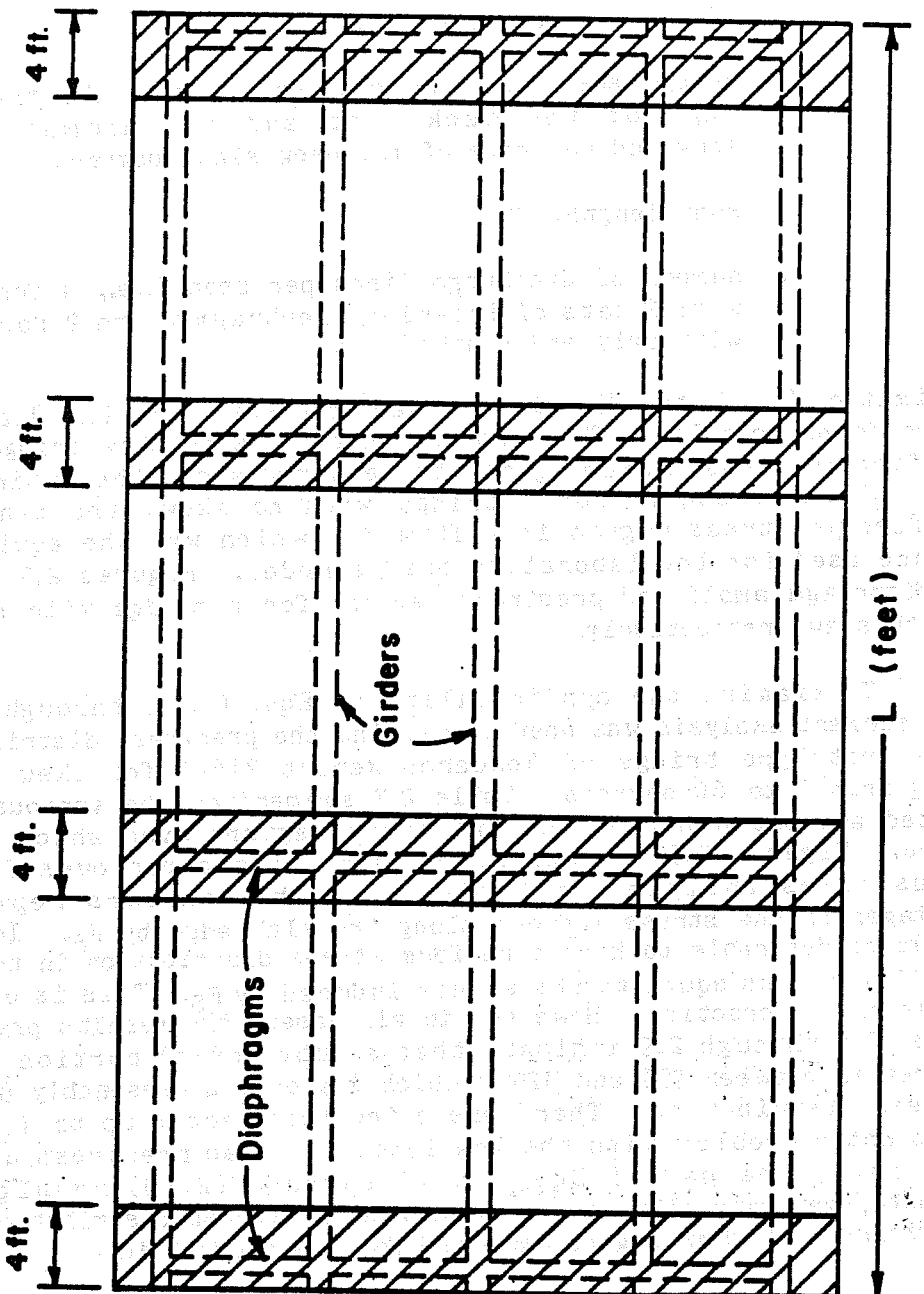


Fig. 2.3 Diaphragm amplified prestress regions for a nonskew bridge

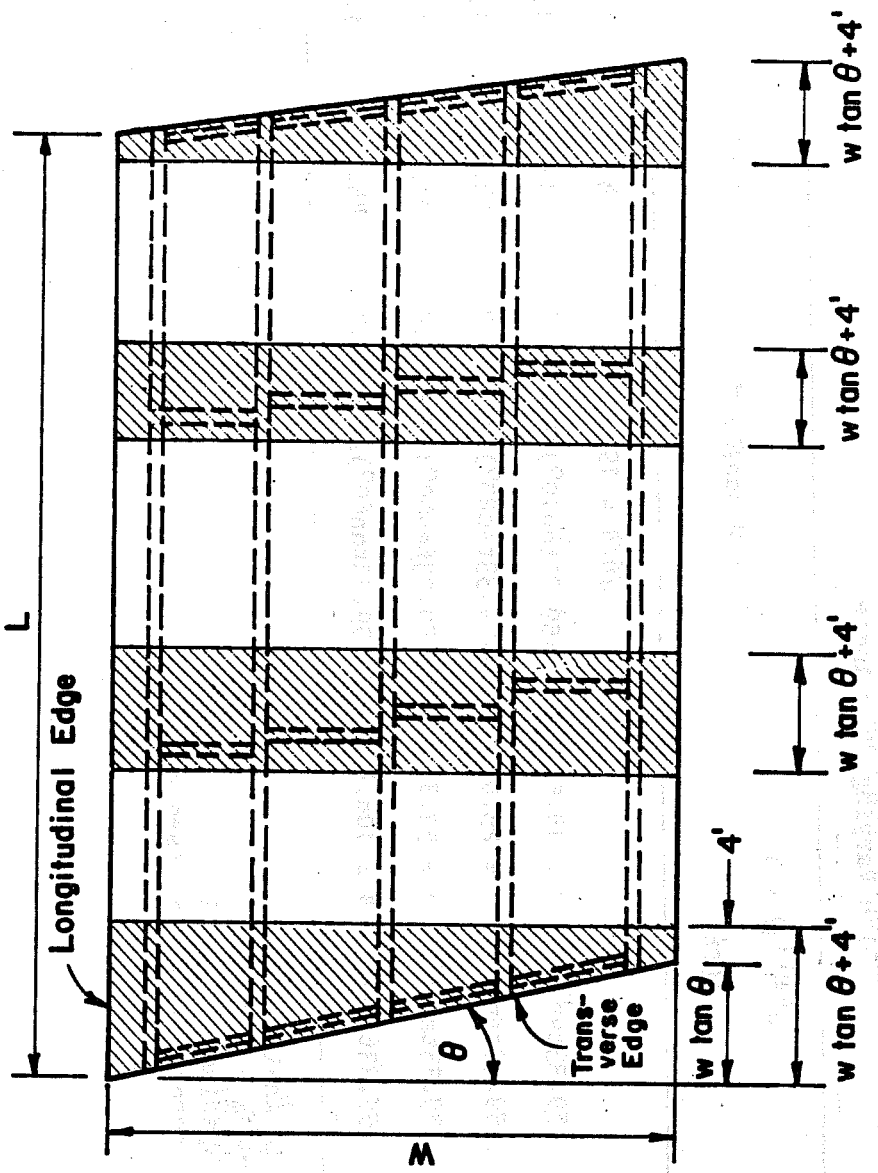


Fig. 2.4 Diaphragm amplified prestress regions for a skew bridge

TABLE 2.7 Cases Analyzed with Finite Element Analysis for Study Bridge with Varying Skew Angle

Skew Angle (deg.)	W tan θ + 4 (ft)	(L + W tan θ)/N	x (ft)	F_e
0	4	76/4 = 19	4	$1.6F_S$
10	59.33tan10° + 4 = 14.5	(76+59.33tan10°)/4 = 21.6	14.5	$1.6F_S$
20	59.33tan20° + 4 = 25.6	(76+59.33tan20°)/4 = 24.4	24.4	$1.2F_S$
40	59.33tan40° + 4 = 53.8	(76+59.33tan40°)/4 = 31.5	31.5	$1.2F_S$
60	59.33tan60° + 4 = 106.8	(76+59.33tan60°)/4 = 44.7	44.7	$1.2F_S$

Assumptions in Analysis:
- Slab thickness = 8.25 in.
- Diaphragm spacing = 25.3 ft
- Standard concrete diaphragms (see Fig. 2.2)

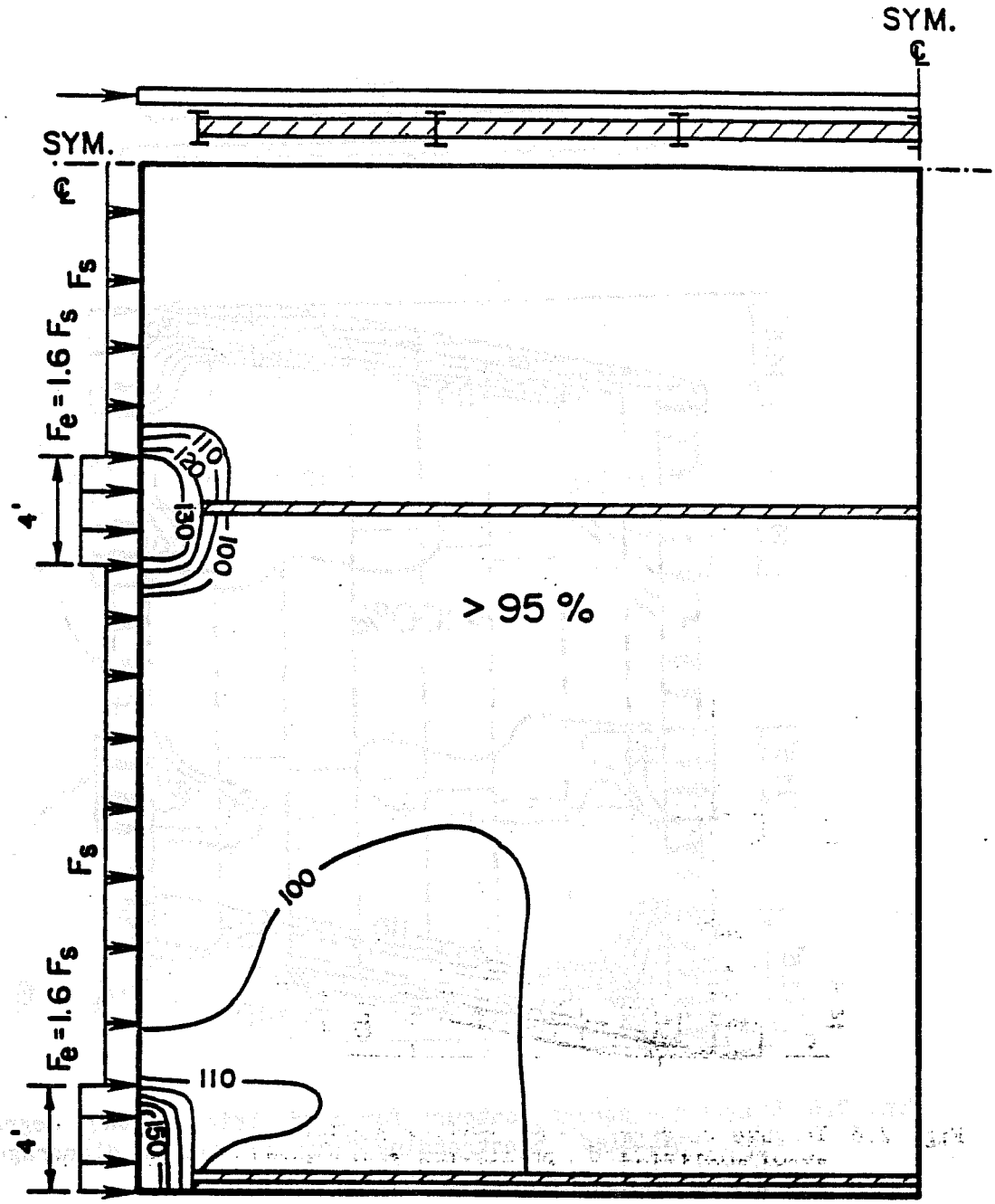


Fig. 2.5 Transverse stress contours for one-quarter symmetry model of study bridge with 0 degree skew; amplified prestressing force applied in diaphragm regions; F_s produces edge stress = 100%

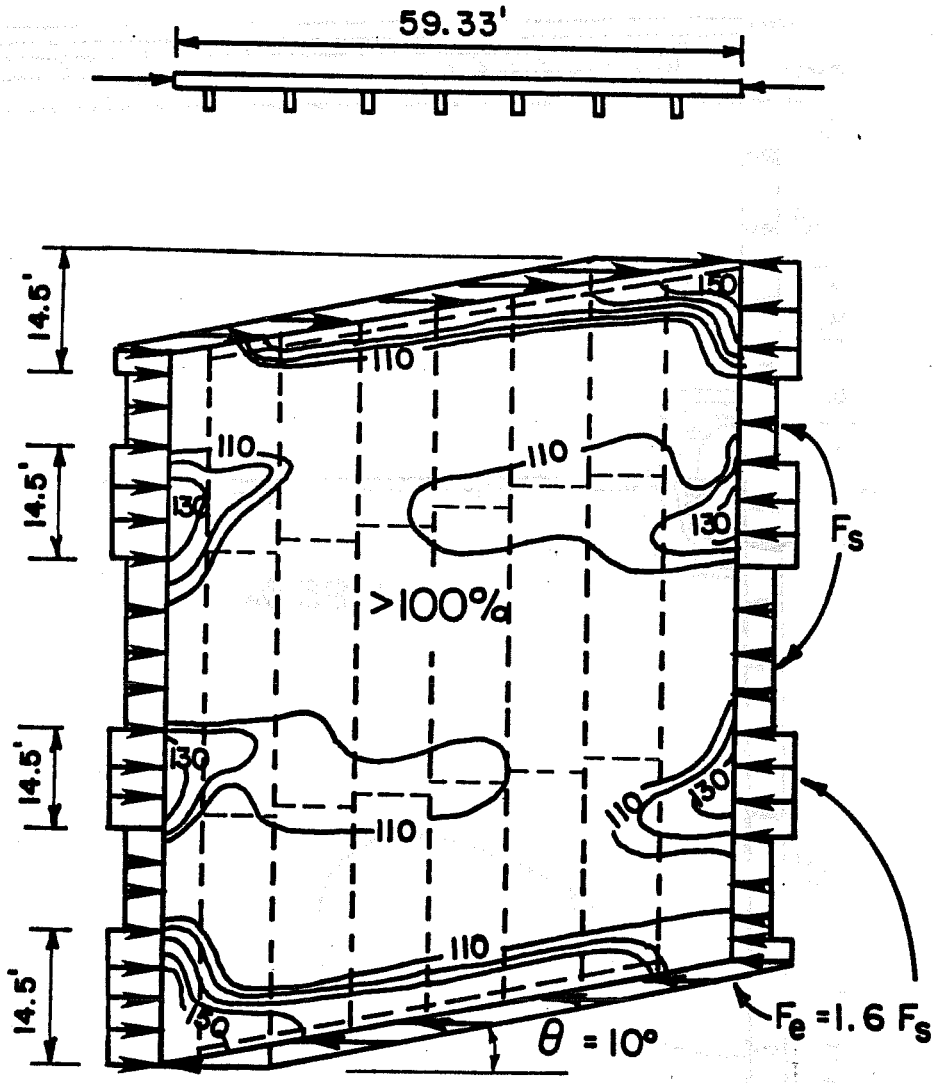


Fig. 2.6 Transverse stress contours for study bridge with 10 degree skew; amplified prestressing force applied in diaphragm regions; F_s produces edge stress = 100%

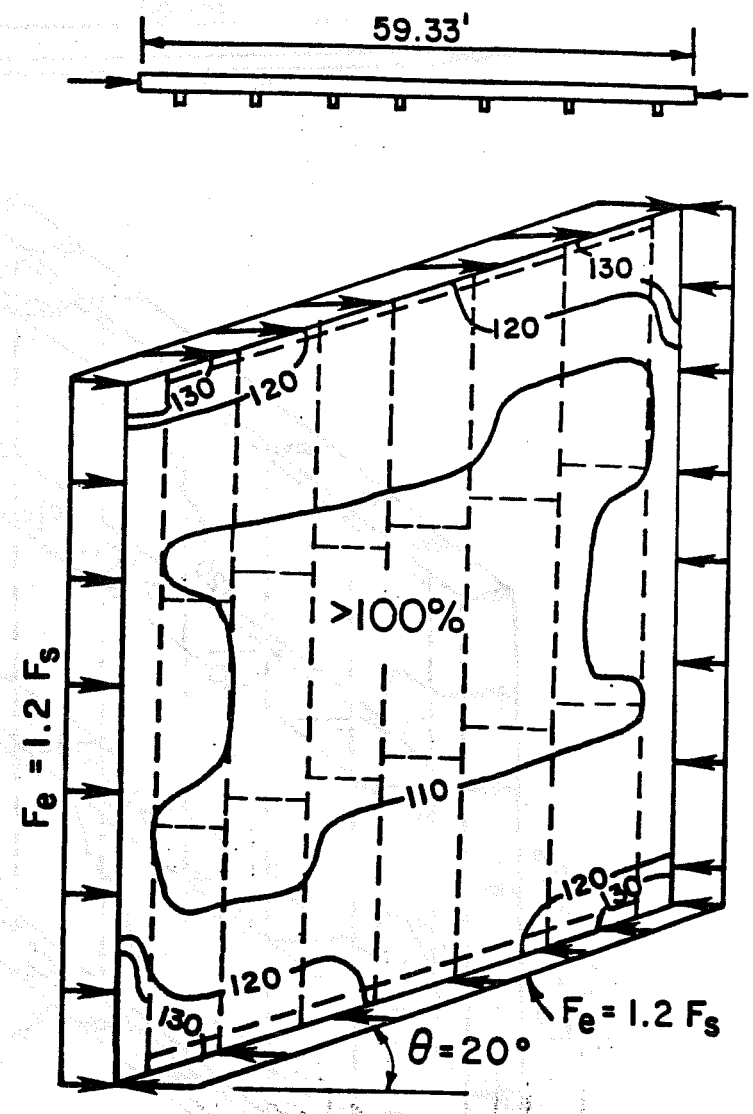


Fig. 2.7 Transverse stress contours for study bridge with 20 degree skew; amplified prestressing force applied along entire length of bridge; F_s produces edge stress = 100%

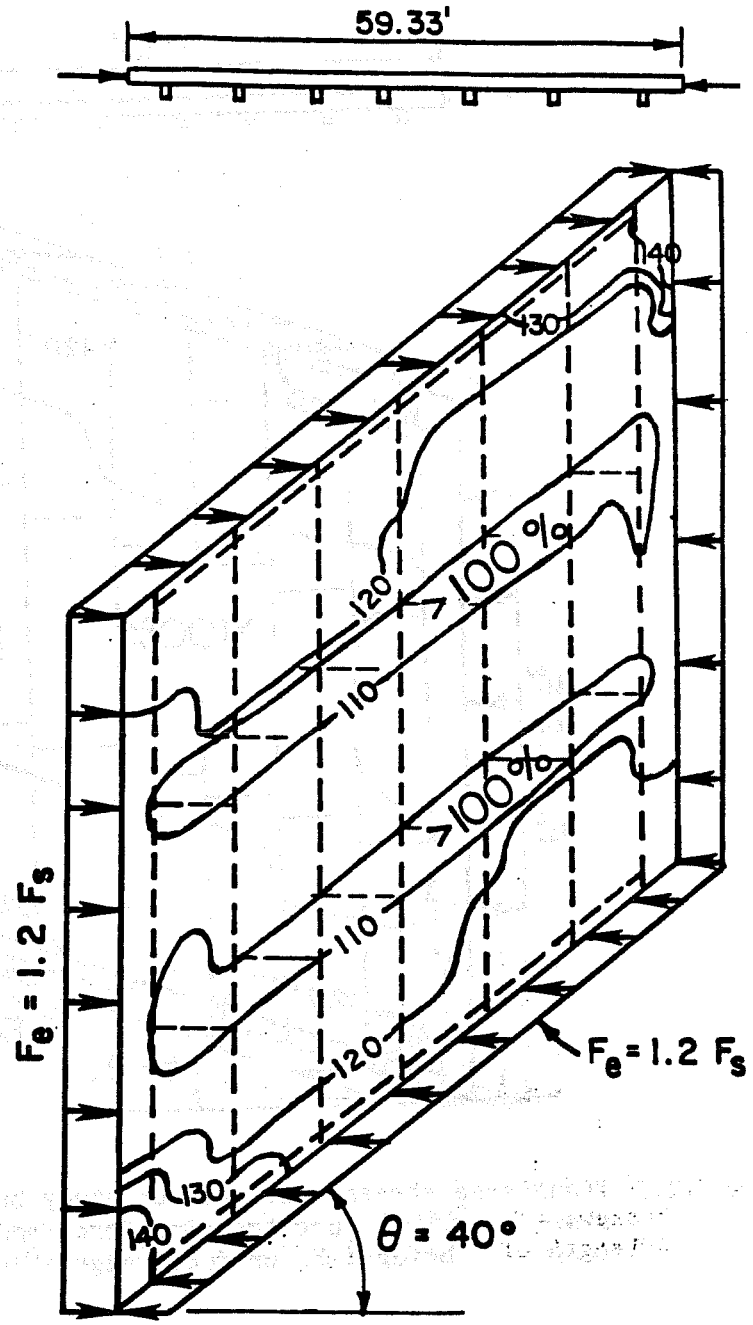


Fig. 2.8 Transverse stress contours for study bridge with 40 degree skew; amplified prestressing force applied along entire length of bridge; F_s produces edge stress = 100%

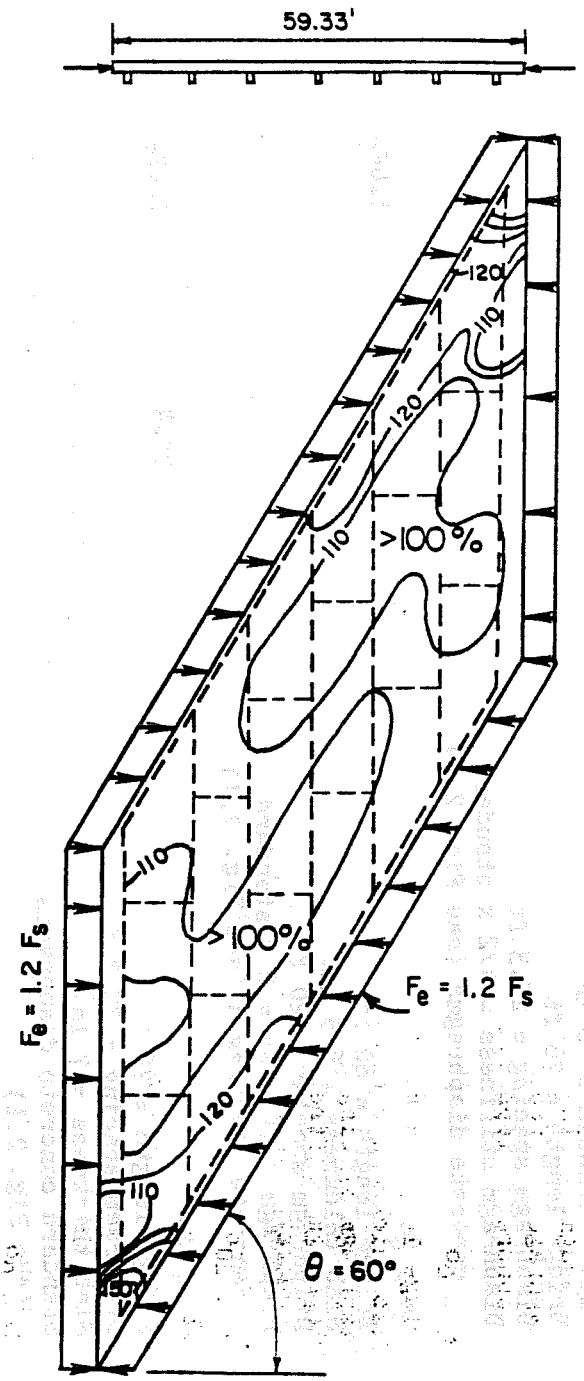


Fig. 2.9 Transverse stress contours for study bridge with 60 degree skew; amplified prestressing force applied along entire length of bridge; F_s produces edge stress = 100%

TABLE 2.8 Cases Analyzed with Finite Element Analysis for Mix of Bridge Parameters

Case	x (ft)	F_e
$\theta = 40^o$ Slab thickness = 10 in. Bridge length = 76 ft Diaphragm spacing = 25.3 ft Diaphragm stiffness = 1/2 x standard concrete diaphragms (see Fig. 2.1)	31.5	$1.2F_S$
$\theta = 0^o$ Bridge length = 60 ft Slab thickness = 6.5 in. Diaphragm spacing = 20 ft Diaphragm stiffness = 2 x standard concrete diaphragms (see Fig. 2.1)	4	$1.6F_S$
$\theta = 20^o$ Bridge length = 76 ft Diaphragm spacing = 25.3 ft Slab thickness = 7 in. Standard concrete diaphragms (see Fig. 2.1)	24.4	$1.2F_S$
$\theta = 0^o$ Bridge length = 76 ft Diaphragm spacing = 25.3 ft Slab thickness = 9 in. Standard concrete diaphragms (see Fig. 2.1)	4	$1.2F_S$

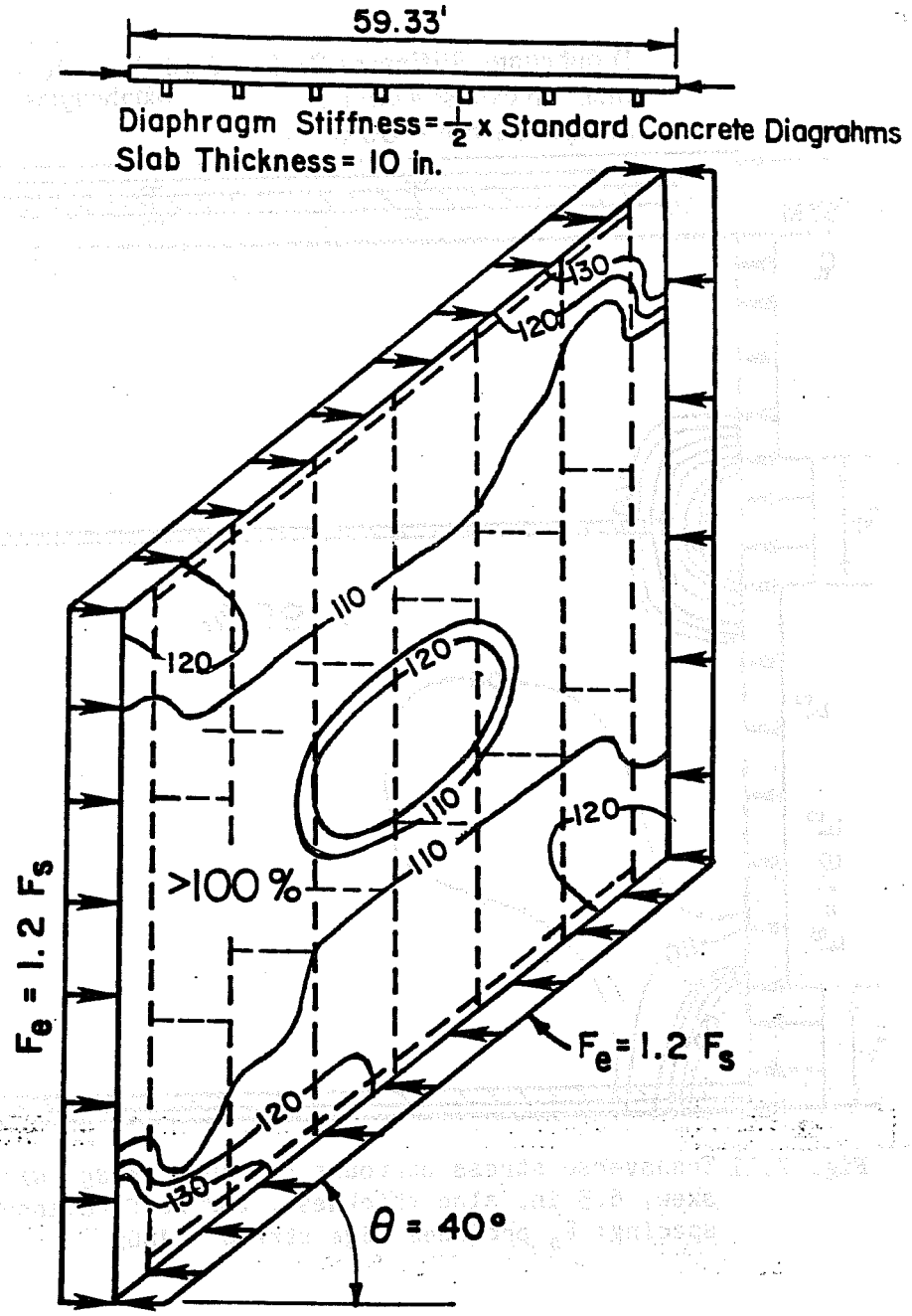


Fig. 2.10 Transverse stress contours of study bridge with 40 degree skew, 10 in. slab thickness, and diaphragm stiffness 1/2 of standard concrete diaphragms; F_s produces edge stress = 100%

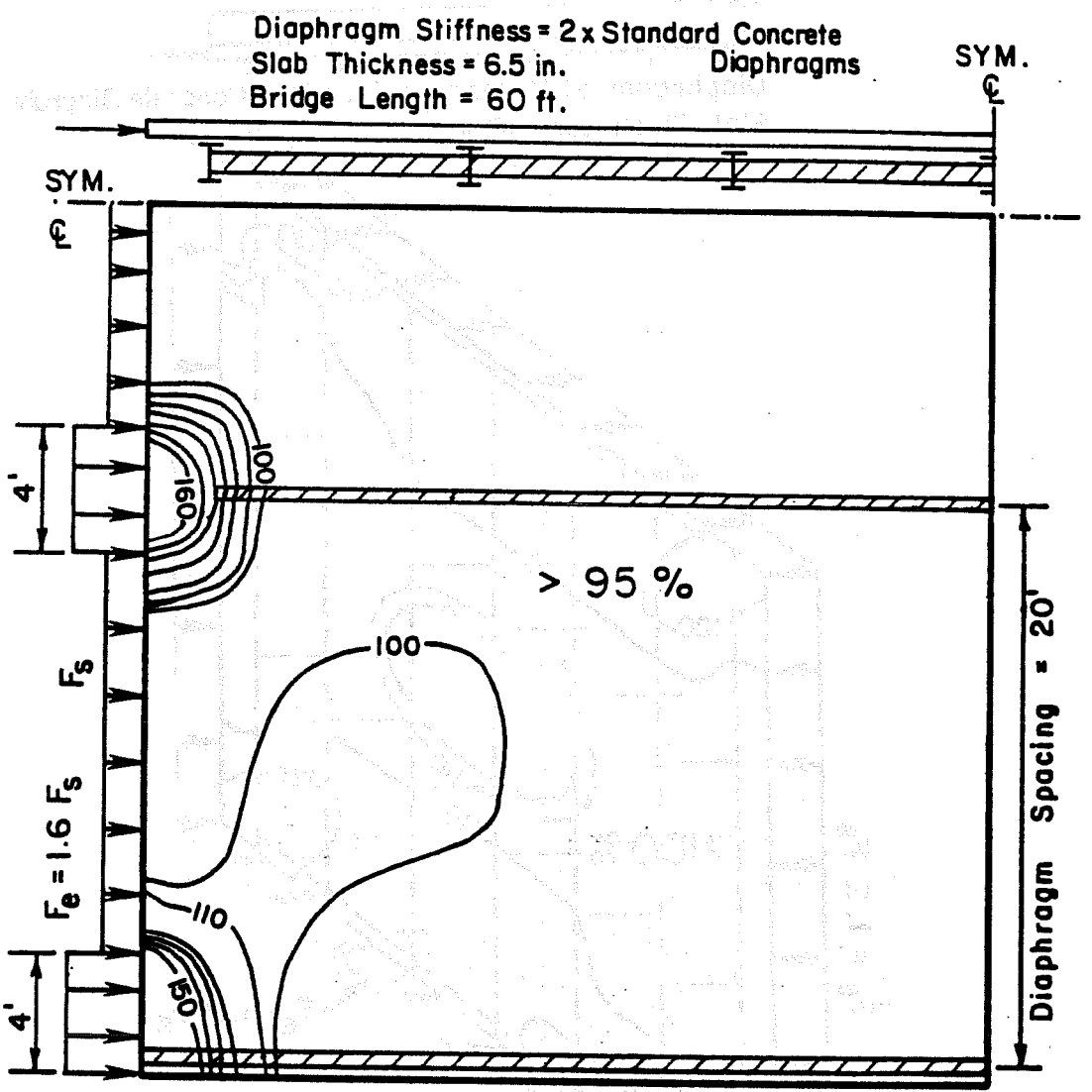


Fig. 2.11 Transverse stress contours of study bridge with 0 degree skew, 6.5 in. slab thickness, and 20 ft diaphragm spacing; F_s produces edge stress = 100%

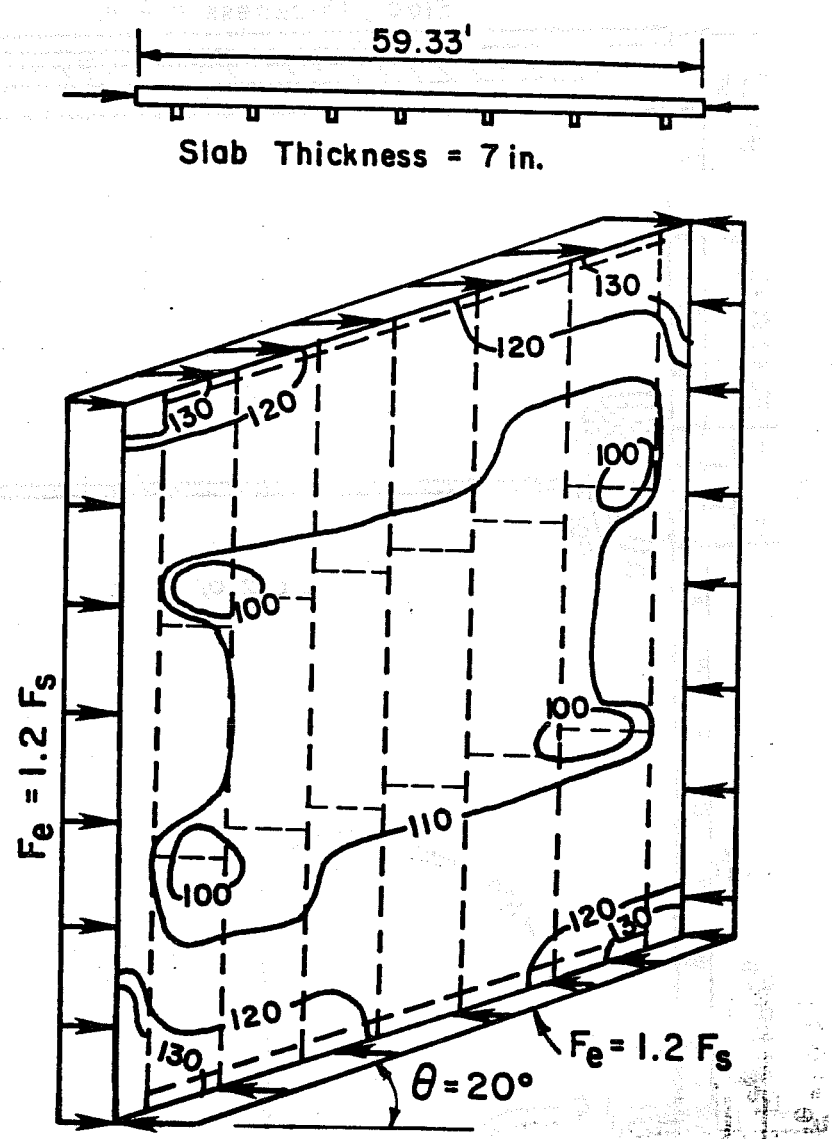


Fig. 2.12 Transverse stress contours for study bridge with 20 degree skew, 7 in. slab thickness, and standard concrete diaphragms; F_s produces edge stress = 100%

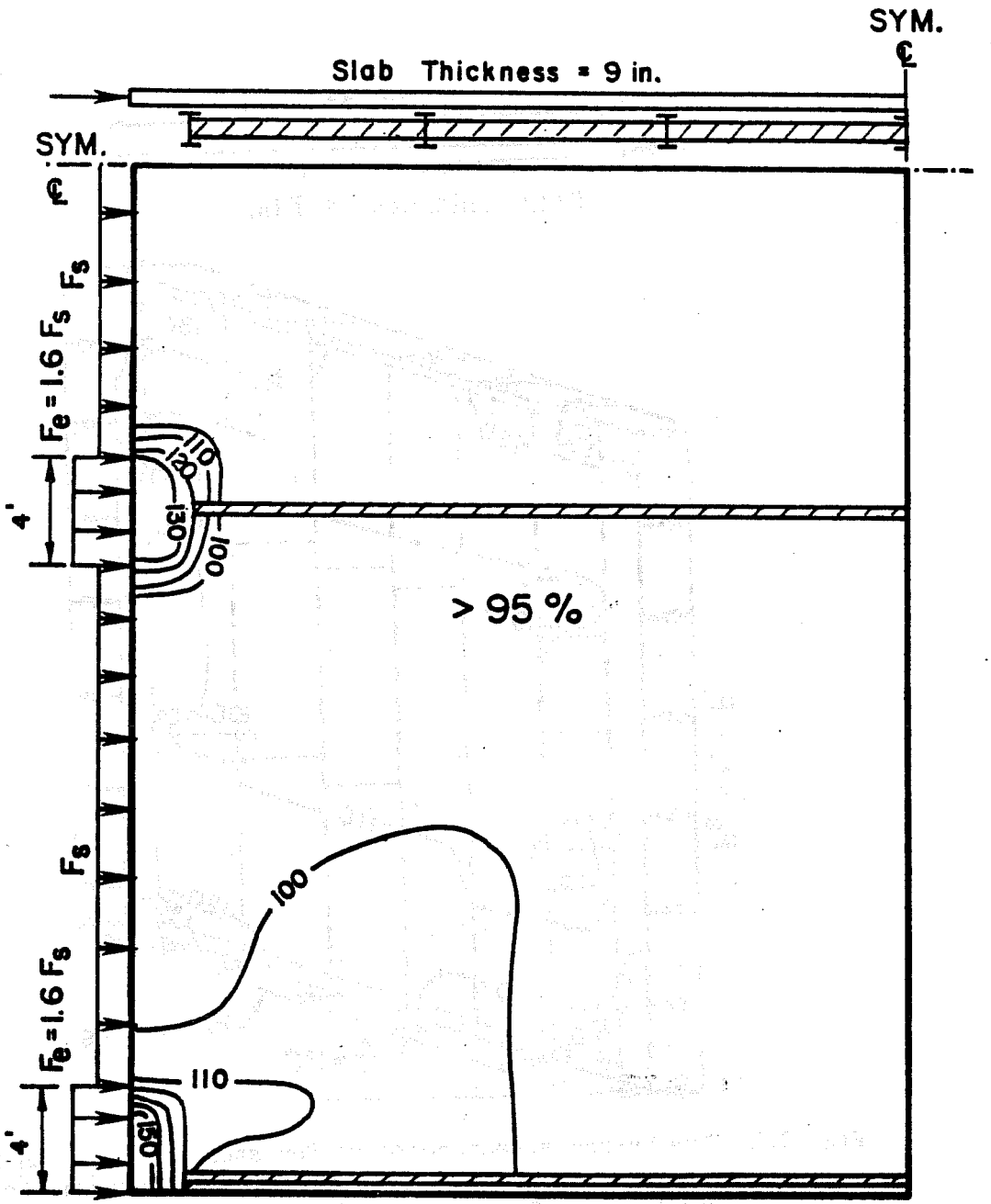


Fig. 2.13 Transverse stress contours for study bridge with 0 degree skew, 9 in. slab thickness, and standard concrete diaphragms; F_s produces edge stress = 100%

substantial portion of the deck slab is in the 95 to 120% range. This implies that a reasonably uniform prestress distribution resulted in each of these cases.

Overall, it is concluded that the use of the very simple Eqs. (2.16) through (2.18) should adequately compensate for the restraint effects of diaphragms. The prestress distributions which result from the use of these equations should be reasonably uniform and generally conservative.

2.2.1.4 Prestressing Losses. Prestressing losses such as friction losses in posttensioning result in less compression to resist imposed loads and must be considered in design. Ralls [9] reported a tendon force loss due to friction of 30% for a posttensioning 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 [6] Section 9.16.

2.2.1.5 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 with thin slabs. However, for those cases in which secondary moments can exist, the effects can be considered using conventional continuous elastic beam theory [10,11].

2.2.1.6 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 non-uniform 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 is addressed by ACI provisions for prestressed slab systems. The maximum allowable tendon spacing is the lesser of 8 times the slab thickness or 5 ft. This provision was set considering the load to be uniformly applied. However, it is believed that with adequate bonded distribution reinforcement, the ACI 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.

AASHTO Section 9.25.2.2 requires that prestressing strands in deck panels not be spaced farther apart than 1-1/2 times the composite slab thickness or more than 18 inches. This provision seems aimed at pretensioned deck panels and seems very restrictive for the type transverse slab envisioned herein.

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 illustrated in Fig. 2.14, there is an area between posttensioning 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 is taken here that it is preferable to prevent load application over these areas rather than providing passive reinforcement as local strengthening. This is because use of conventional reinforcing 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. As discussed later, only the vertical loads are considered in establishing the maximum transverse prestressing tendon spacing.

Figure 2.15(a) shows a section through a deck at the longitudinal edge. A concentrated wheel load is located one ft from the face of the guardrail, in accordance with AASHTO design specifications. 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. Figure 2.15(b) shows the moment capacity and moment due to loading across this section, taken at midpoint between two tendons.

Although the total transverse slab moment capacity is provided by the multiple couples in which the tension forces are provided by the prestressing, concrete tensile strength, and nonprestressed reinforcing, only the moment capacity due to prestressing will be considered here. This is because if the concrete cracks at some point in time, its tensile strength is no longer available. Also, relying on nonprestressed reinforcing to carry ordinary service loads in the slab could induce undesirable cracking. From the edge of slab stress tests described in Report 316-2, it is known that fully effective uniform post-tension slab stresses are obtained at a distance of 0.85 times the tendon spacing in from the anchorages. As shown in Fig. 2.15(b), the moment capacity of the deck due to prestressing decreases from the full amount at this point to zero at the tendon anchorages. The exact shape of the moment capacity curve in this region, however, is unknown, since

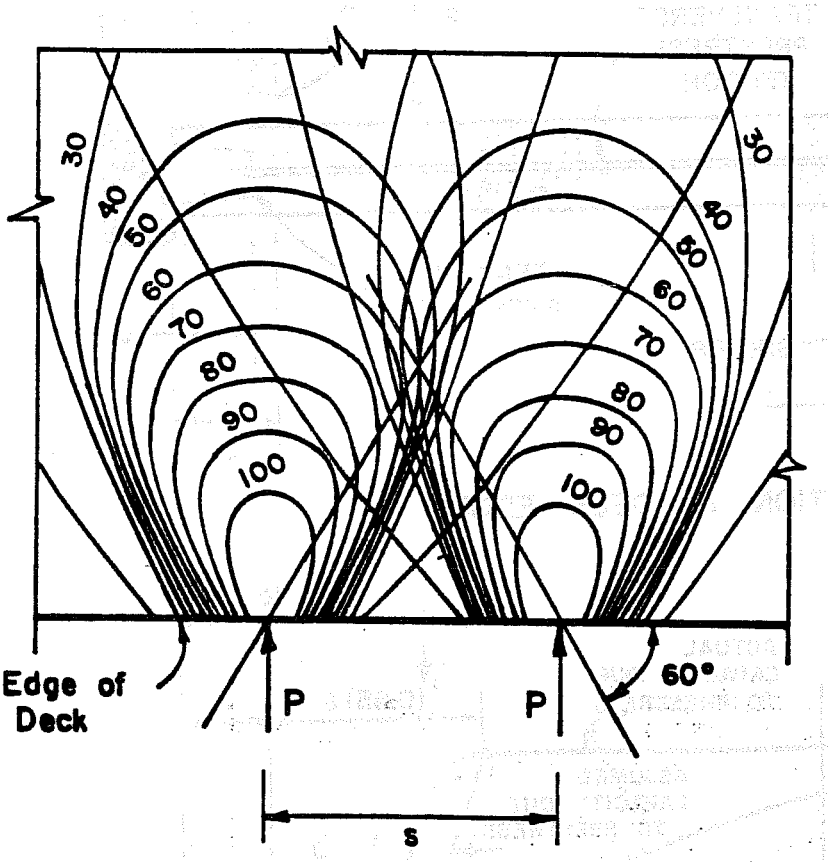
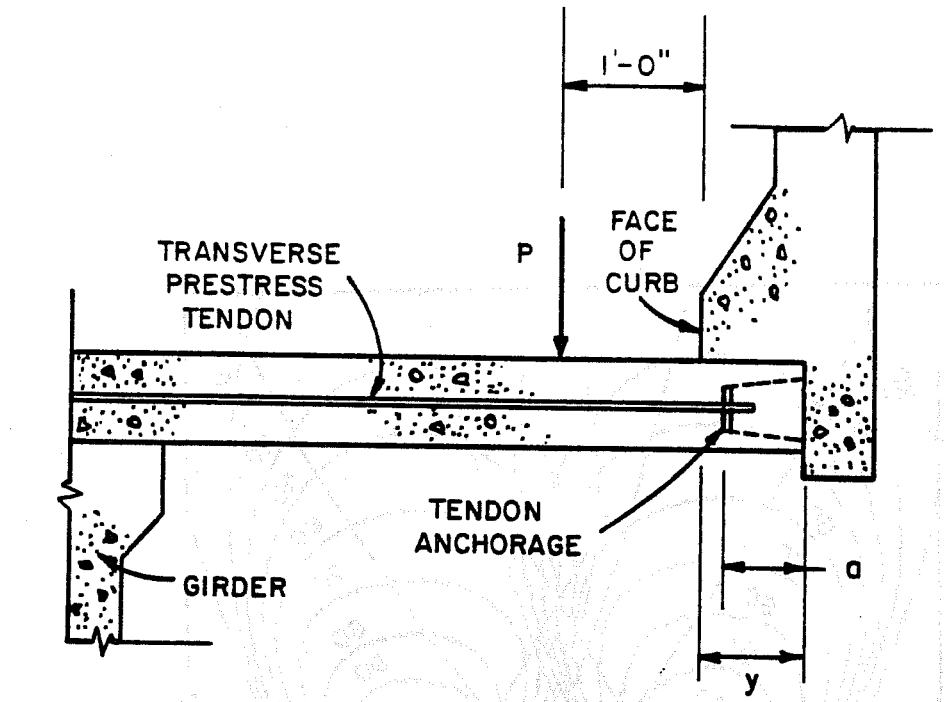
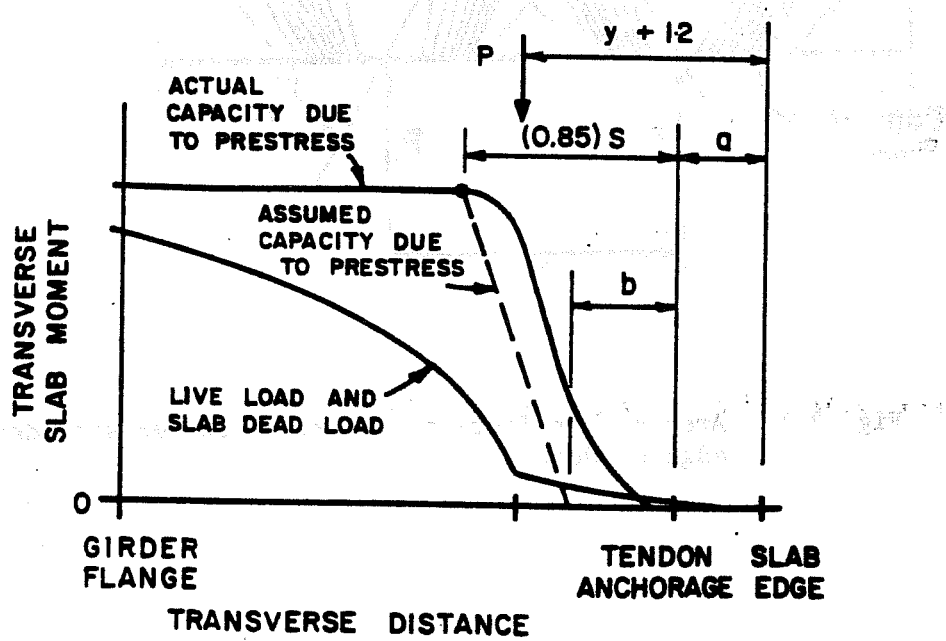


Fig. 2.14 Area of ineffective prestress between strands at edge of deck



(a) SECTION AT DECK EDGE



(b) TRANSVERSE MOMENT PROFILE BETWEEN TENDONS

Fig. 2.15 Development of maximum tendon spacing

the experimental data were not that refined. To simplify the problem, the moment capacity due to prestressing in this region is assumed to decrease linearly to a zero value at a distance "b" from the tendon anchorages. From Fig. 2.15, a reasonable value for the distance "b" is selected as 0.4 times the distance from the anchorage to the location of full effective stresses.

$$b = (0.4)\ (0.85)\ S \quad \text{(in.)}$$

$$b = (0.34)\ S \quad \text{(in.)}$$

where S = tendon spacing (in.).

The moment profile due to the applied concentrated live load and slab dead load is also shown in Fig. 2.15(b). The shape of the live load moment profile is curved concave downward due to the varying effective width of the supporting transverse slab strip. The effective width of transverse slab resisting the concentrated wheel load varies from a maximum value at the support to a small effective width at the point of loading. The slab dead load moment varies parabolically from a maximum at the support to zero at the slab edge.

Note that the dead load moment of the traffic rail has not been included in the moment diagram in Fig. 2.15(b). While it does produce a transverse moment and needs to be considered in the overall slab design, it does not need to be considered in determining the maximum tendon spacing. This is because it possesses sufficient stiffness and reinforcing to easily span the distance between the tendons. Consider a Texas Standard Type T201 traffic rail. This concrete rail is nominally 7 in. wide, 27 in. high, and weighs 212 lb per linear ft. If a bridge deck had prestressing tendons spaced at the maximum ACI value of 5 ft, and an expansion joint was so placed in the rail as to make the rail cantilever the full 5 ft distance between tendons, the reinforcing needed on the top of the rail would be less than 0.06 $in.^2$, or less than 1/3 of a #4 reinforcing bar. The standard design calls for two #4 bars in the top of the rail. The dead load of traffic rails is therefore easily transferred longitudinally to the strand locations where the prestressing is fully effective in the slab, and need not be considered in the maximum strand spacing determination.

Transverse slab moments due to lateral loads on the traffic rail are also not included in the diagram in Fig. 2.15(b). Moment capacity must be provided continuously along the deck edges for lateral impact loads on the traffic rail. But since major collision impact is not an ordinary service load, cracking of the concrete under this load is permissible. (Such cracking should be sealed in the bridge repair after the accident.) Therefore, instead of arranging the prestressing tendons to provide the necessary moment capacity, nonprestressed reinforcement may be used to provide moment capacity along the deck

edges for rail loads due to lateral impact of traffic. This aspect is discussed further in Sec. 2.2.2.1.

While the slab dead load moment may be significant towards the cantilever support, it is small enough at the location of the applied concentrated live load to be neglected. If the distance "y" in Fig. 2.15(a) is taken as 18 in. (approximately the width of a Jersey style concrete barrier) with a slab thickness of 8 in., the transverse slab dead load moment at the location of the concentrated load in Fig. 2.15(b) is 0.31 kip-ft per ft. This compares with normal design service load moments for an 8 in. slab on the order of 4 to 5 kip-ft per ft.

Referring again to Fig. 2.15(b), 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. This condition gives:

$$y + 12 = b + a \tag{2.19}$$

Substituting the value for "b" into Eq. 2.19 gives:

$$12 + y = (0.34)\, S_{max} + a$$

$$S_{max} = 2.94\,(y - a + 12) \quad \text{(in.)}$$

For design purposes, use

$$S \leq 3\,(y - a + 12) \quad \text{(in.)} \tag{2.20}$$

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 when 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 10-1/4 in., for a 1-3/8 in. 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.

Table 2.9 gives the maximum tendon spacings for various values of "y" and "a" as calculated using Eq. 2.20.

TABLE 2.9 Maximum Tendon Spacings from Eq. 2.20 (in.)

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.

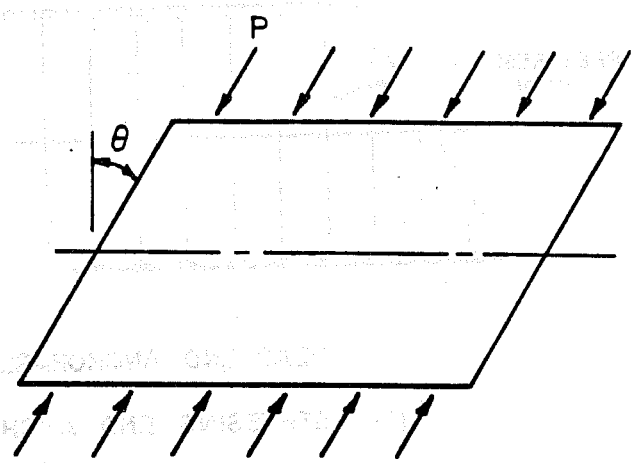
The maximum transverse prestressing tendon spacing allowed, then, should be the minimum given by the ACI limits of 8 times the slab thickness or 5 ft, or Eq. 2.20.

2.2.1.7 Tendon Layout for Skewed Bridges. On a nonskew bridge, the transverse tendons may be distributed at the 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.

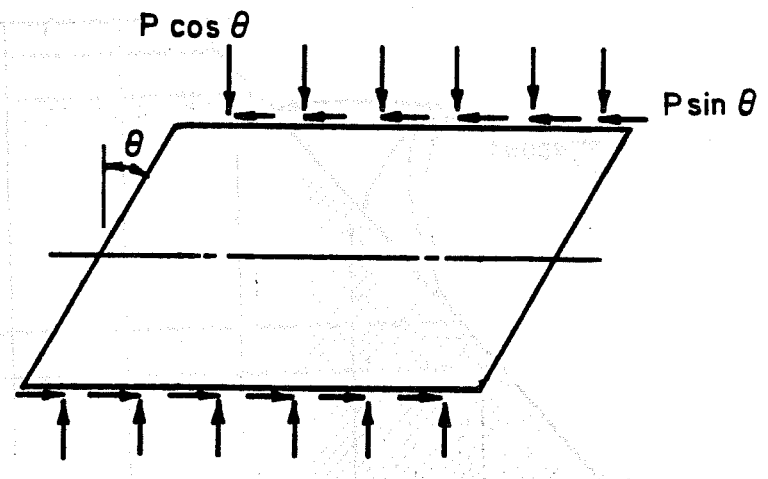
It is not recommended that tendons be placed on a skew in these instances. As illustrated in Fig. 2.16, the transverse prestressing force available to resist slab moments is reduced from the applied prestressing by the cosine of the skew angle. This amounts to nearly a 15% reduction for a bridge with 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 as shown in Fig. 2.16. An example bridge, 59 ft-4 in. wide and 60 ft long with a 25 deg. skew, was analyzed using the two-dimensional finite element model [12]. For the example deck, the transverse concrete compressive stress due to post-tensioning varied from 92 to 127% of that intended. This small shortfall in the required transverse stress could be easily compensated for by increasing the amount of post-tensioning strands. However, this increase combined with that needed because of component forces as discussed above would require an increase in post-tensioning force of nearly 25%. Because of this reduced efficiency, the use of skewed tendons should be avoided wherever possible.

For perpendicular tendons to be used on skewed bridges, the complications mentioned previously 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. 2.17(a). The two major difficulties in this situation are at the acute corners of the deck, as illustrated in Fig. 2.17(b). There the required tendon lengths become so short that losses due to anchorage seating are extreme. 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. 2.18. 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

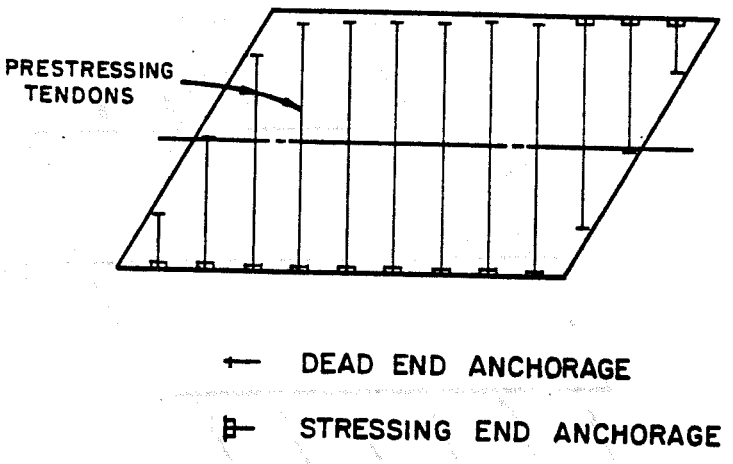


(a) SKEWED TRANSVERSE PRESTRESSING TENDONS

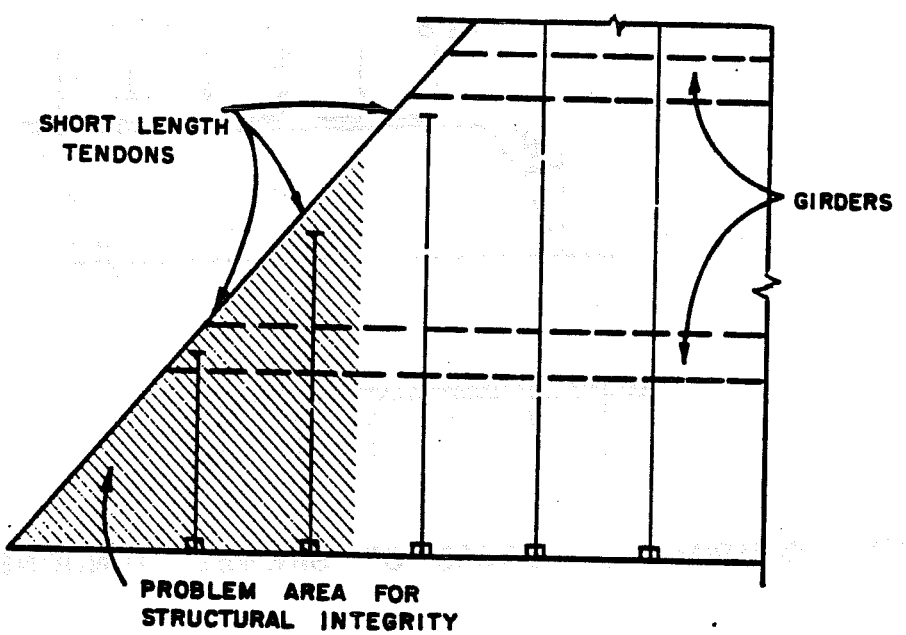


(b) COMPONENT FORCES OF SKEWED TENDONS

Fig. 2.16 Transverse prestressing tendons placed on a skew



(a) TENDON ANCHORAGE TYPES FOR SKEWED BRIDGES



(b) PROBLEMS AT ACUTE CORNERS OF DECK

Fig. 2.17 Details of skewed bridge with perpendicular prestressing tendons

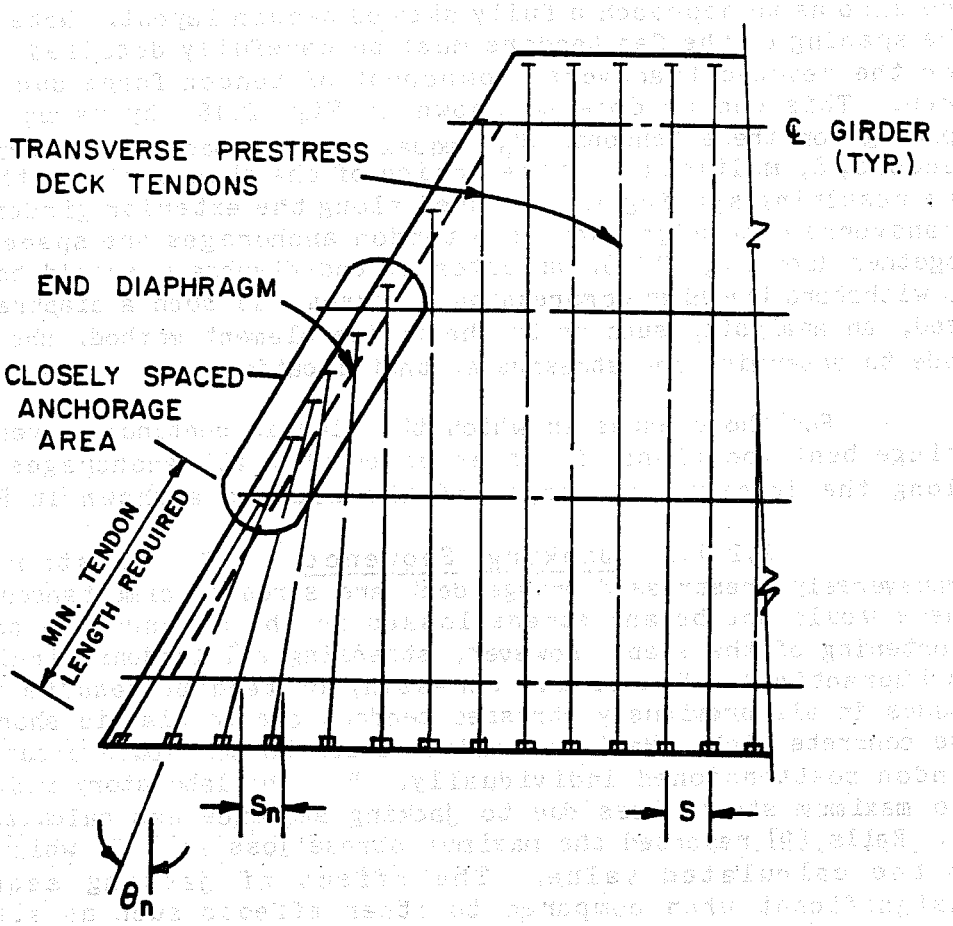


Fig. 2.18 Tendon layout for skewed decks

due to live load, allows the use of longer prestress 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. Note also that the spacing of the fan tendons must be carefully detailed to account for the reduced transverse component of tendon force due to tendon skew. This can be done as shown in Fig. 2.18, by using a 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. 2.18), 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 as a shown in Fig. 2.19.

2.2.1.8 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 posttensioned individually. For the laboratory model bridge, the maximum stress loss due to jacking sequence was calculated to be 3%. Ralls [9] reported the maximum stress loss as 3.8%, 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.

2.2.1.9 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.

2.2.1.10 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. This was approximately

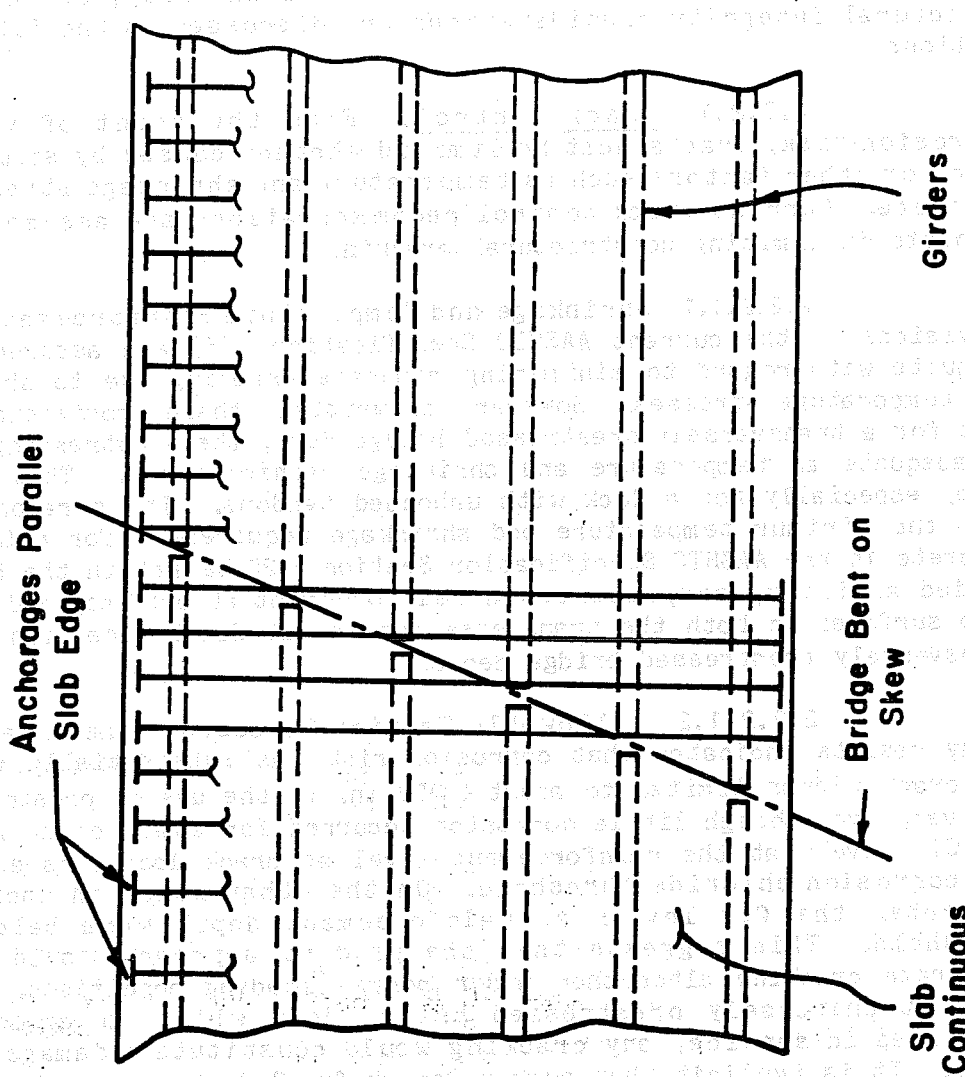


Fig. 2.19 Anchorages in skew bridge with continuous slab

the compression induced by the prestressing in the durability specimens reported in Report 316-1.

2.2.2 Serviceability, Strength and Structural Integrity. The structural design implications of serviceability, strength and structural integrity considerations are discussed in the following sections.

2.2.2.1 Crack Control. From the point of view of corrosion risk, cracks must be limited whether caused by structural loads or other factors such as temperature and shrinkage stresses in concrete. Current crack control recommendations are assumed to be adequate in limiting nonstructural cracking.

2.2.2.1.1 Shrinkage and Temperature Reinforcement. The provisions of the current AASHTO Specifications [6] 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.

2.2.2.1.2 Allowable Tension Stresses. The durability study results indicated that corrosion risk was substantially reduced for crack widths limited to about 0.002 in. 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 corrosion chloride 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.

A design philosophy which eliminates cracking under service loads is expressed in equation form as:

$$f_t < f_r \qquad (2.21)$$

where f_t = maximum tension stresses in the concrete deck due to loading.

and f_r = tensile strength of concrete.

This equation suggests that cracking will not occur if the flexural tension stresses due to induced moments for applied loads are kept below the tension stress which causes concrete to crack. For a transversely prestressed bridge deck, f_t would be the maximum flexural stress under service loads magnified by the impact factor for dynamic effects, and is calculated using classical elastic beam theory.

The most respected test for tensile strength of concrete, f_r, is the split cylinder test [13,14]. A value of 6.5 $\sqrt{f'_c}$ can be used to estimate the split cylinder tensile strength and probably best represents an average value. Since the compressive strength of concrete follows a normal distribution [15], and since the tensile strength has been correlated to the compressive strength, then it is reasonable to assume that the tensile strength also follows a normal distribution. The standard deviation for concrete compressive strengths greater than 4000 psi, which is probably the minimum f'_c for a prestressed concrete application, is reported to be a constant value of about 600 psi [15]. Applying statistical theory for a variable which follows a normal distribution, 3.46 $\sqrt{f'_c}$ is determined to be the value in which there is a 95% probability that the actual tensile strength is greater. Thus, Eq. (2.21) translates into:

$$f_t < 3.46 \sqrt{f'_c} \qquad (2.22)$$

However, in practice, there are material defects and construction errors which cannot be easily predicted. This was very evident in the laboratory bridge model where it was found that the transverse slab stresses were very sensitive to errors in tendon placement. To account for this variability, a strength reduction factor, ϕ, must be applied. Thus, Eq. (2.22) becomes:

$$f_t < \phi\, 3.46 \sqrt{f'_c} \qquad (2.23)$$

Normally, for flexural design of reinforced and prestressed concrete, under AASHTO Specifications, a ϕ = 0.9 to 1.0 is assumed, which indicates a fairly good quality control. AASHTO Section 9.14 sets ϕ = 0.95 for post-tensioned cast-in-place concrete members. However, for a thin slab application where errors in tendon placement are more pronounced, ϕ = 0.85 is probably more appropriate. The resulting equation is:

$$f_t < 3 \sqrt{f'_c} \qquad (2.24)$$

This tension stress index of 3 $\sqrt{f'_c}$ is exactly what is currently specified in AASHTO 9.15.1.1(a) for members with auxiliary bonded reinforcement which are subject to severe corrosive exposure conditions such as marine spray or deicing salts.

However, besides the corrosion and cracking problem in bridge decks, there is also concern for the fatigue and cracking problem. A bridge slab is subject to dynamic and fatigue loads from moving vehicular traffic. A recently completed study by Overman [16] revealed that once flexural cracks form in pretensioned concrete beams, prestressing tendons can suffer fatigue fracture. This situation is probably aggravated in posttensioned concrete because of fretting. The results from Overman's study [16] indicate that 3 $\sqrt{f'_c}$ is probably a reasonable limit on tension for fatigue considerations.

It is evident that cracking could seriously impair the corrosion resistance as well as the fatigue performance of a bridge deck. Based on both of these considerations as well as the likelihood of overloads on a bridge, it would be prudent to adopt a somewhat smaller tension limit than 3 $\sqrt{f'_c}$. The value 2 $\sqrt{f'_c}$ seems to be a reasonably conservative tension limit, and yet has significant economic advantages over a zero tensile stress limit. Therefore, the proposed design recommendation is to limit the extreme fiber slab tensile stresses under full service loads to 2 $\sqrt{f'_c}$.

2.2.2.1.3 Bonded Reinforcing Requirements. In addition to transverse prestressing in the deck, bonded reinforcement should be used near the top and bottom slab surfaces, in both the transverse and longitudinal directions, according to the following guidelines.

Transverse Reinforcing. 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 in the transverse direction recommended for each slab surface per foot width of deck follows from ACI 318 requirements, and is:

$$A_B = 0.024\ t\ (\text{in.}^2)$$

where t = overall thickness of deck (in.)

This amount of bonded transverse reinforcing 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 reinforcing need be provided only for temperature and shrinkage control as described in Sec. 2.2.2.1.1.

Longitudinal Distribution Reinforcement. Since the laboratory study done for this project pertained only to slab and girder bridges, no data are available for the longitudinal distribution reinforcing requirements of box girder bridges. It is therefore recommended that distribution reinforcement be provided in the bottom of the top slab of a concrete box girder bridge in accordance with AASHTO Sec. 3.24.10.

As discussed in companion Report 316-2, the longitudinal moment in a typical composite I-beam and slab bridge deck due to concentrated wheel loads is approximately 1/4 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, determination of the longitudinal distribution reinforcing in the bottom of the slab should be made by direct design. The design longitudinal moment should be 1/4 of the transverse live load plus impact moment, and the amount of reinforcing should conform to the minimum requirements of AASHTO Sec. 8.17.1. However, to expedite the design process, a design value of (0.03) t (sq. in. per ft. width of deck) for longitudinal reinforcement in slab-girder bridges appears adequate if a more exact determination is not desired.

For the reinforcing arrangement shown in Fig. 2.20, the maximum spacing of the longitudinal distribution bars allowed by AASHTO requirements for flexural reinforcement distribution (AASHTO Sec. 8.16.8.4) is 9.8 in. 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. is recommended for longitudinal distribution reinforcing, which provides nearly three bars in the cone of load influence beneath a 20-in. wide wheel.

Use of the above recommendations for slab and girder bridges results in the distribution reinforcing shown in Table 2.10.

2.2.2.1.4 Transverse Cracking. A transversely prestressed bridge deck designed in accordance with the recommendations for transverse prestressing presented in this report should be free of deck cracks running in the longitudinal direction. As pointed out earlier, the great advantage of the absence of these cracks is that one mechanism by which corrosion of the reinforcing and freeze-thaw deterioration of the concrete takes place is eliminated. However, if slab cracking should occur running in the transverse direction across

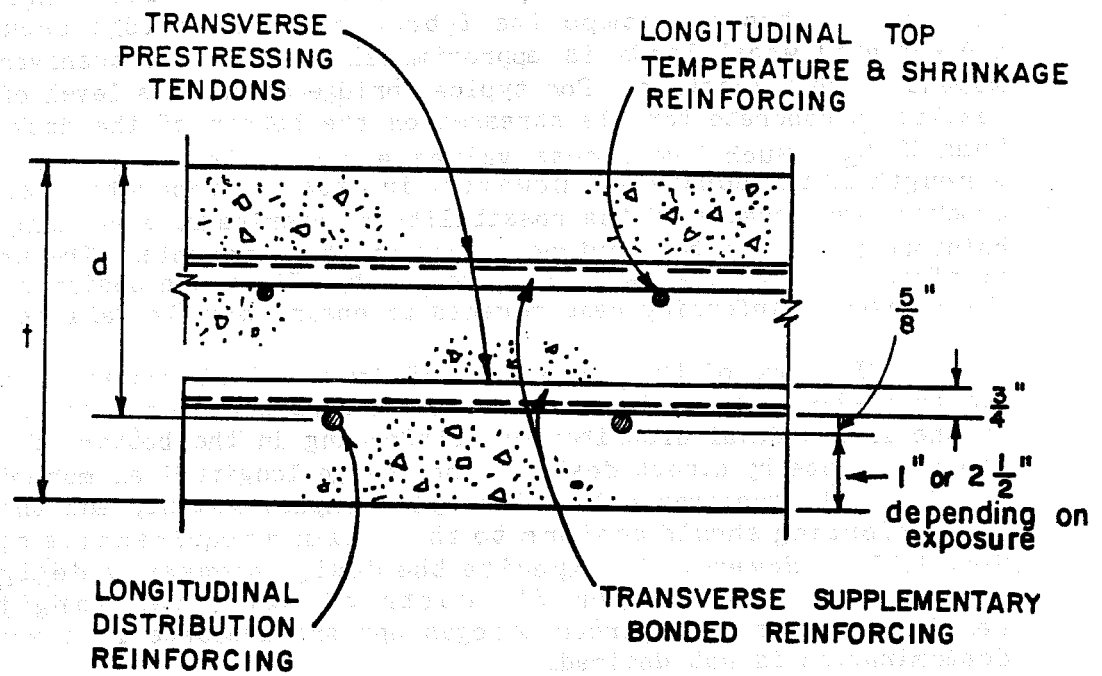


Fig.2.20 Transverse section of deck showing depth to longitudinal distribution reinforcing

TABLE 2.10 Recommended Longitudinal Distribution Reinforcing in the Bottom of the Deck for Slab and Girder Bridges

Deck Thickness (in.)	Spacing of Longitudinal Bars (in.)	
	#4	#5
6.5	11-1/2	12*
7.0	10-1/2	12*
7.5	10	12*
8.0	9	12*
8.5	8-1/2	12*
9.0	8	12*
10.0	7-1/2	11-1/2
11.0	6-1/2	10-1/2

* Maximum spacing controls

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.

Transverse cracks in a bridge deck are caused by several factors which may induce longitudinal tensile stresses in the slab, including: a) local effects of wheel loads, b) negative live load moments in continuous bridges, c) increasing camber of the superstructure due to creep in prestressed concrete girders, and d) temperature and shrinkage effects. Of the causes of transverse cracking in bridge decks, negative live load moments and temperature and shrinkage effects are the real design concerns.

For slab and girder bridges, the two most promising methods for dealing with transverse cracking in the deck are the use of epoxy-coated reinforcing and longitudinal posttensioning of the bridge deck. Because adequately thick, high quality, uncracked concrete protects the reinforcement against corrosion, resists freeze-thaw deterioration, and has a low susceptibility to fatigue, longitudinal posttensioning of the bridge deck is the preferred method. If a deck is not longitudinally posttensioned, epoxy-coated reinforcing 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. of concrete surfaces exposed to an eggressive environment should be coated.

Longitudinally posttensioning a bridge superstructure is a viable method of preventing transverse cracking in bridge decks. The ACI Building Code [7] recommends that if posttensioning is used to counteract temperature and shrinkage stresses, a minimum average compressive stress of 100 psi due to the effective prestress (after losses) on gross concrete area should be provided (ACI Code Sec. 7.12.3.1).

In precast prestressed concrete box girder bridges, where differential shrinkage strains in the superstructure are not a problem, the 100 psi minimum compressive stress recommended by ACI might seem overly conservative. However such bridges have no tensile capacity at the segment joints other than that furnished by the epoxy jointing material. Therefore, it is recommended that AASHTO Section 9.7.3.1.2 be followed which specifies that in segmental concrete box girder bridges no tension be allowed across any joint during any stage of erection or under service loading.

For slab and girder bridges and nonprestressed or cast-in-place box girder bridges where the effects of shrinkage are more significant, the 100 psi minimum compressive stress recommended by ACI seems appropriate. Although the following discussion on how this stress level may be achieved is oriented towards slab and girder

bridges, most of the content is also applicable to nonprestressed or cast-in-place box girder bridges.

Several possibilities exist for longitudinally posttensioning slab and girder bridge superstructures to eliminate transverse temperature and shrinkage cracking in the deck. These include: longitudinal posttensioning of only the slab for the full length of the bridge; longitudinal posttensioning 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 prestress 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 posttensioned. Also, girders for a bridge using this method of construction must be designed to accept the additional stress the longitudinal posttensioning imposes.

Longitudinal Posttensioning for Simple Span Bridges. Placing longitudinal posttensioning in the bridge deck is a simple and efficient means of providing the 100 psi minimum average compressive stress recommended by ACI to compensate for temperature and shrinkage stresses. Using the simple-span bridge shown in Fig. 2.21 as an example, the required longitudinal post-tensioning force in the slab for each girder is calculated as follows:

The distance from the centroid of the composite section to the top surface of the slab of 15.32 in. Assuming 2-1/2-in. cover and #4 top transverse bars, the longitudinal tendon eccentricity is then

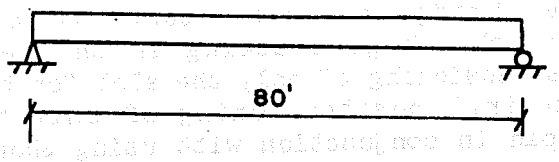
$$e = 15.32 - 2.5 - 0.5 - 0.38 = 11.94 \text{ in.}$$

The required force for a 100 psi stress at the centroid of the slab is found from

$$f = P/A + Pey_s/I$$

$$100 = P/1089 + (P(11.94)(11.95))/271,400)$$

$$P = 69,260 \text{ lb/girder}$$



Texas "Type C" Prestressed Girders
Girder Spacing = 7'-4"
Slab Thickness = 6 3/4"

Composite Girder Properties

$A = 1089 \text{ in.}^2$

$I = 271{,}400 \text{ in.}^4$

$S_T = 17{,}700 \text{ in.}^3$

$S_B = 8630 \text{ in.}^3$

Fig. 2.21 Example Simple-Span Bridge for determining longitudinal slab prestressing

Assuming 1/2-in. diameter strands and an effective stress after losses of 150 ksi, the required longitudinal post-tensioning in the deck is three strands per girder, or slab strands spaced transversely at about 2 ft 5 in. Note that for this example, approximately 36 strands are needed in each precast girder, assuming an allowable tensile stress of $3\sqrt{f'_c}$ and provision for a future asphalt overlay of 4 in. The deck posttensioning induces a tensile stress in the bottom of the precast girder of 32 psi which may be easily compensated for in the design of the precast member. Stressing and anchorage of the strands may be accomplished at a transverse edge of the deck, beneath the deck in end diaphragms or blockouts, or in blockouts in the deck itself, using center stressing posttensioning hardware as shown in Fig. 2.22.

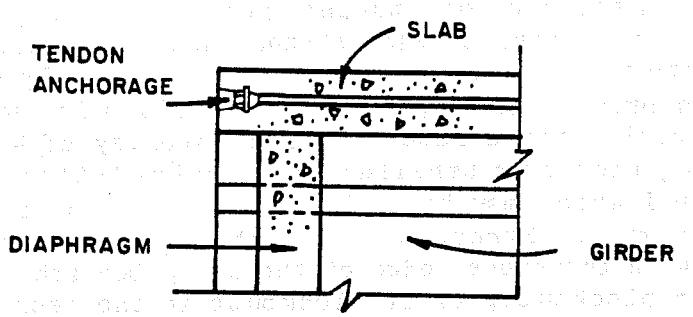
Obviously, for bridges with larger girders and thicker decks, more longitudinal posttensioning will be required than for the previous example. However, this increase does not alter the feasibility of longitudinal posttensioning of the deck on larger bridges.

Posttensioning the slab longitudinally to eliminate transverse shrinkage and temperature cracking is an attractive solution to the problem. The disadvantages to this method are the additional materials and labor required, the possible difficulties in locating convenient tendon anchorages, and the slightly modified precast girder design necessary.

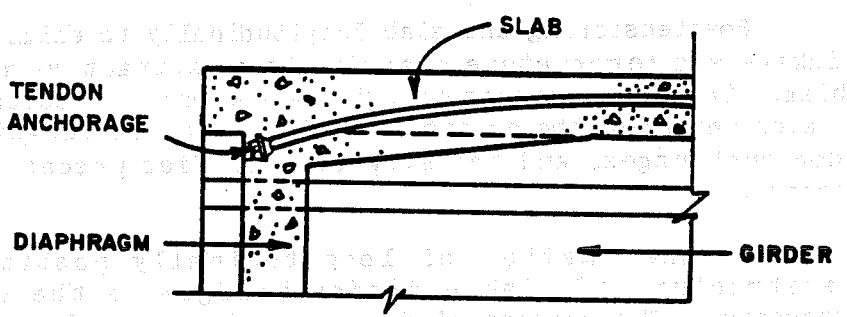
Another method of longitudinally posttensioning the superstructure of slab - girder bridges is the use of staged prestressing. The precast girder is prestressed only for its own dead load and that of the slab. At the time of casting, the girders are provided with posttensioning ducts which later are used for tendons carrying composite dead load and live loads. The location and size of these tendons are designed to ensure a 100 psi average compressive stress in the slab with no live load applied.

For the example bridge of Fig. 2.21, 16 strands would be needed in the precast girder for dead loads, and 33 additional strands would have to be installed later in posttensioning ducts centered 16 in. above the bottom of the girders at midspan. Thus, using the staged construction method in this case results in a total requirement of 49 1/2-in. prestressing strands per girder (allowing $3\sqrt{f'_c}$ tension). As stated previously, a conventionally designed precast prestressed girder for this span would require about 36 strands per girder. Adding three strands per girder for longitudinal posttensioning of the slab, a total of 39 strands would be needed in this case. It can therefore be seen that the method of stage prestressing is less efficient than post-tensioning the deck longitudinally to eliminate transverse cracking in simple spans.

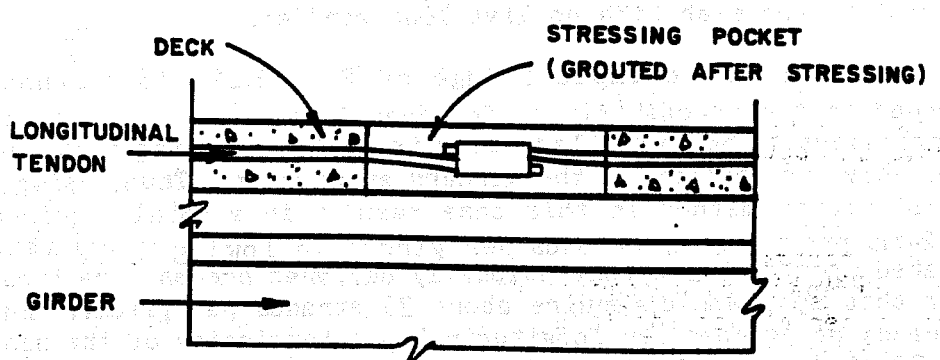
Other disadvantages of the stage prestressing method are that the tendon anchorages would be difficult to access in multiple span bridges, and that the tendons would have to be grouted since they are vital to the structural integrity of the bridge.



(a) IN TRANSVERSE DECK EDGE



(b) IN END DIAPHRAGM



(c) IN BLOCKOUT IN DECK

Fig. 2.22 Longitudinal sections through deck showing possible anchorages for longitudinal slab tendons

However, stage prestressing would help control increasing camber of the superstructure due to creep. The method also offers the ease of precast tendon ducts.

Longitudinal Posttensioning for Continuous Span Bridges. Continuous prestressed concrete box girder bridges must resist large negative dead and live load moments near the intermediate supports. If enough prestressing force has not been provided at appropriate eccentricities, significant transverse cracking will occur in the deck. As discussed in Sec. 2.2.2.1.4, a reasonable limit for longitudinal service load tensile stress in the top slab of a precast segmental box girder bridge to prevent this cracking is 0 psi. While cast-in-place box girder bridges might sustain small levels of tensile stress, they are subject to much higher shrinkage strains and creep losses. It is therefore recommended that all prestressed concrete box girder bridges be designed for a maximum longitudinal tensile stress in the top slab of 0 psi.

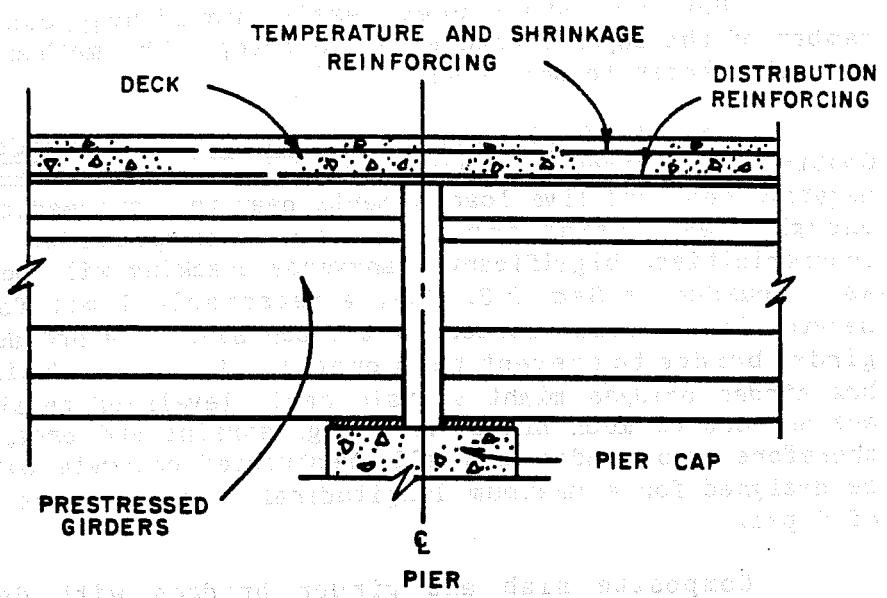
Composite slab and girder bridges with decks which are continuous longitudinally over several spans have been constructed in several different ways in Texas. In one method, the slab is often made continuous over the piers for the sole purpose of eliminating expansion joints, while the girders in adjacent spans are in no way connected. A second method is where continuity connections are made over the pier for both the slab and girders such that the structure behaves as a continuous bridge for dead load added after composite action is developed and for all live load.

a) Continuous Slab and Discontinuous Girders

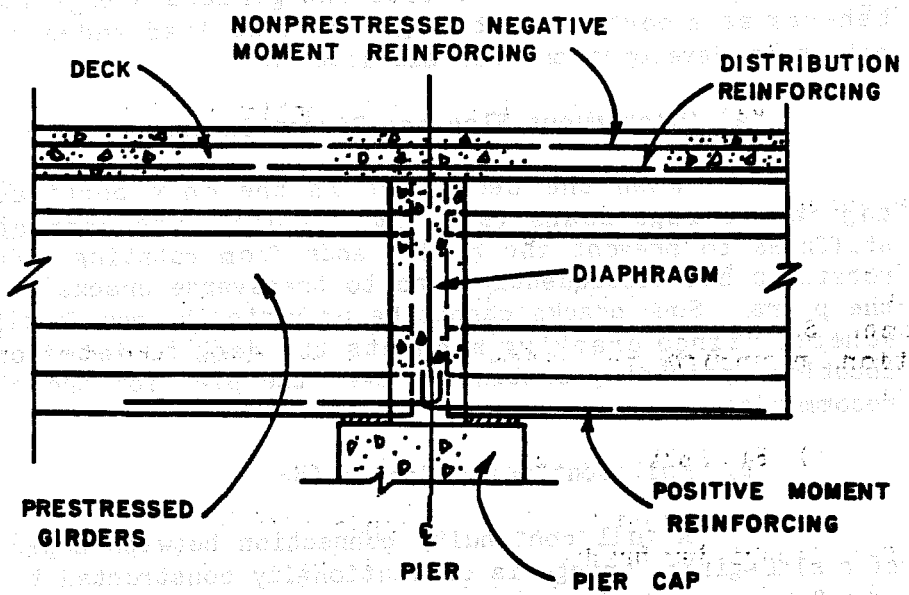
When the deck slab is the only continuous portion of adjacent bridge spans (see Fig. 2.23(a)), the connection lacks the stiffness to prevent the girder ends from rotating under load. These rotations can subsequently lead to transverse cracks in the deck above the piers. Such cracks cannot be prevented by any feasible reinforcing scheme. Since cracking subjects the deck to deterioration at that location, providing continuity over the pier for the slab only is not recommended.

b) Full Continuity Connection

A full continuity connection between superstructure spans of a slab-girder bridge is conventionally constructed by providing mild reinforcement in the concrete deck over the pier, and by joining the ends of the girders together by some means (see Fig. 2.23(b)). The use of nonprestressed reinforcement in a negative moment region to resist loads allows transverse cracking of the concrete to occur. To prevent subsequent corrosion of the reinforcing, one solution would be to use



(a) CONTINUOUS SLAB AND DISCONTINUOUS GIRDERS



(b) FULL CONTINUITY CONNECTION

Fig. 2.23 Continuity connections in slab-girder bridges

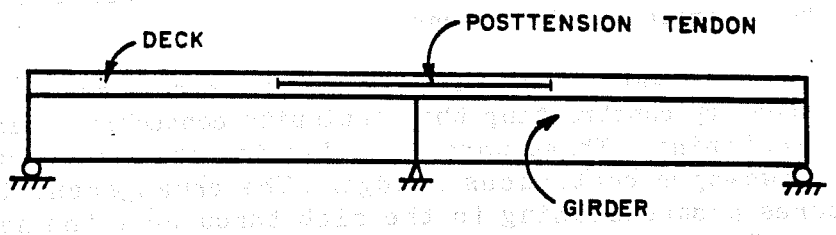
epoxy-coated reinforcing for the top layer in the bridge deck. Transverse cracking due to temperature and shrinkage stresses could also be tolerated in this manner.

Transverse cracking over the piers could be eliminated altogether by constructing the continuity connection using longitudinal posttensioning. Three possibilities for this are shown in Fig. 2.24 for a two-span continuous bridge. The arrangement in Fig. 2.24(a) features posttensioning in the slab throughout the negative moment region. For a typical two-span bridge with tendons in the slab running between quarter points of adjacent spans, two-thirds of the intended moment due to prestressing is lost to secondary moment. In addition, the longer the tendons are, the larger the adverse secondary moment becomes. For tendons extending the entire length of a two-span continuous bridge, the negative secondary moment at the pier is 50% larger than the positive primary moment. Thus, longitudinal tendons extending the full bridge length to control temperature and shrinkage stresses in continuous structures increase the required negative moment capacity at the piers.

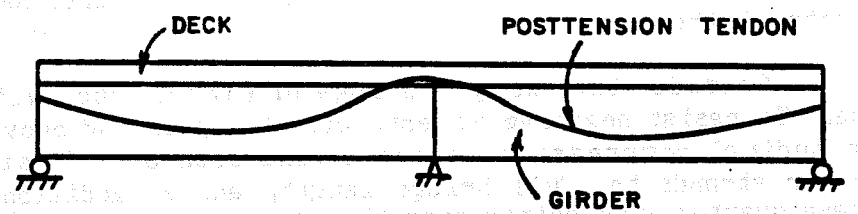
Consider the example bridge of Fig. 2.21 but with continuous spans. To resist negative moments over the pier and provide a minimum longitudinal compressive stress in the deck of 100 psi, six 1/2-in. diameter strands the full bridge length, and an additional 28 strands between quarter span points over the pier, would be required in the top of the slab above each girder. For this continuous bridge, the precast girders would each need 28 pretensioned strands. A total of 34 strands the full bridge length and 28 strands over the pier would then be used for each girder. Regardless of length, 34 strands per girder must be posttensioned and anchored. These figures compare with the total of 39 strands (36 in pretensioned member, 3 in deck) needed for each girder in the simple span design.

Besides the inefficiency due to secondary moments, this tendon layout suffers the disadvantages of a large number of tendon ducts to be field placed, a large number of strands to be posttensioned, and a possible thicker deck to accommodate placement of prestressing tendons in both directions.

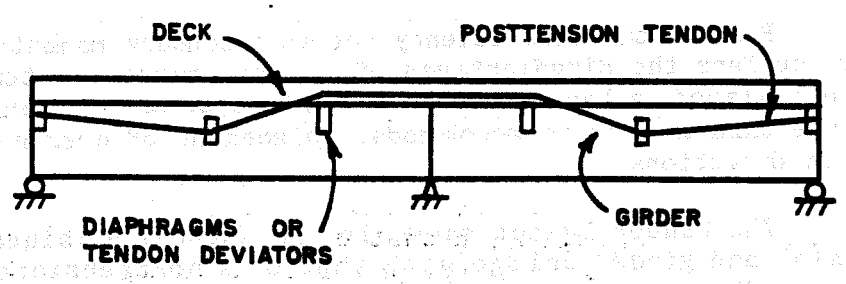
The tendon layout shown in Fig. 2.24(b) combines the concept of a slab and girder bridge with that of a posttensioned box girder bridge. While the draped tendon profile shown and the following discussion apply to bridges with precast prestressed concrete I girders, the method can also be applied to other bridge types by using external posttension tendons as shown in Fig. 2.24(c). This use of longitudinal posttensioning has several advantages: the draped tendon profile reduces secondary moment; negative moment over the pier is resisted by prestressing and thus does not cause cracking in the slab; the tendon path may be adjusted to provide compression in the slab to resist temperature and shrinkage stresses along the entire span; the



(a) POSTTENSIONING IN DECK ONLY



(b) DRAPED CONTINUOUS POSTTENSIONING



(c) EXTERNAL POSTTENSIONING TENDONS

Fig. 2.24 Continuity connections with longitudinal posttensioning

required amount of pretensioning in the precast girders may be reduced; and the posttension ducts are placed in the precast girders in the manufacturing plant, simplifying field construction.

The design of such a superstructure begins with determining the posttensioning requirements over the support, then adjusting the tendon path to obtain the desired compression in the deck, and finally, designing the precast girders to carry the loads not taken by the post-tensioned composite girder section.

Consider again the example bridge of Fig. 2.21 with continuous spans. Assume a draped posttensioned tendon with eccentricities as illustrated in Fig. 2.25. If 3 $\sqrt{f'_c}$ tension is allowed in the bottom fibers of the girders and a minimum of 100 psi compression is maintained in the deck for temperature and shrinkage stresses, 24 1/2-in. diameter strands are needed for post-tensioning, and 22 strands for pretensioning in each girder. A total of 46 strands is then needed per girder for the continuous design with draped tendons, compared to 39 strands per girder for the simple span bridge, and 62 strands per girder (including partial length strands) for the continuous design with slab tendons. Although the continuous bridge uses more prestressing than the simply supported structure, it has the great advantage of minimizing the number of transverse deck joints.

For structures longer than three or four spans where friction losses become large, stressing of the tendons may be done at temporary blockouts in the deck over the piers.

In summary, continuous bridges constructed with only the slab continuous over the piers are not recommended. Bridges provided with continuity connections for composite dead load and live load should preferably be constructed with longitudinal posttension tendons providing a minimum of 100 psi average compression in the deck. Continuous bridges which do not utilize longitudinal posttensioning should be provided with epoxy-coated reinforcement in the deck and grouted transverse prestressing as a minimum level of protection against corrosion.

2.2.2.2 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. Companion Report 316-2 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 [9] reported that the maximum upward camber and downward deflection was less than 0.01 in. due to prestressing the model bridge deck. This represents a camber or deflection to slab span ratio of about 0.02%. Thus, these small deflections are of no practical concern.

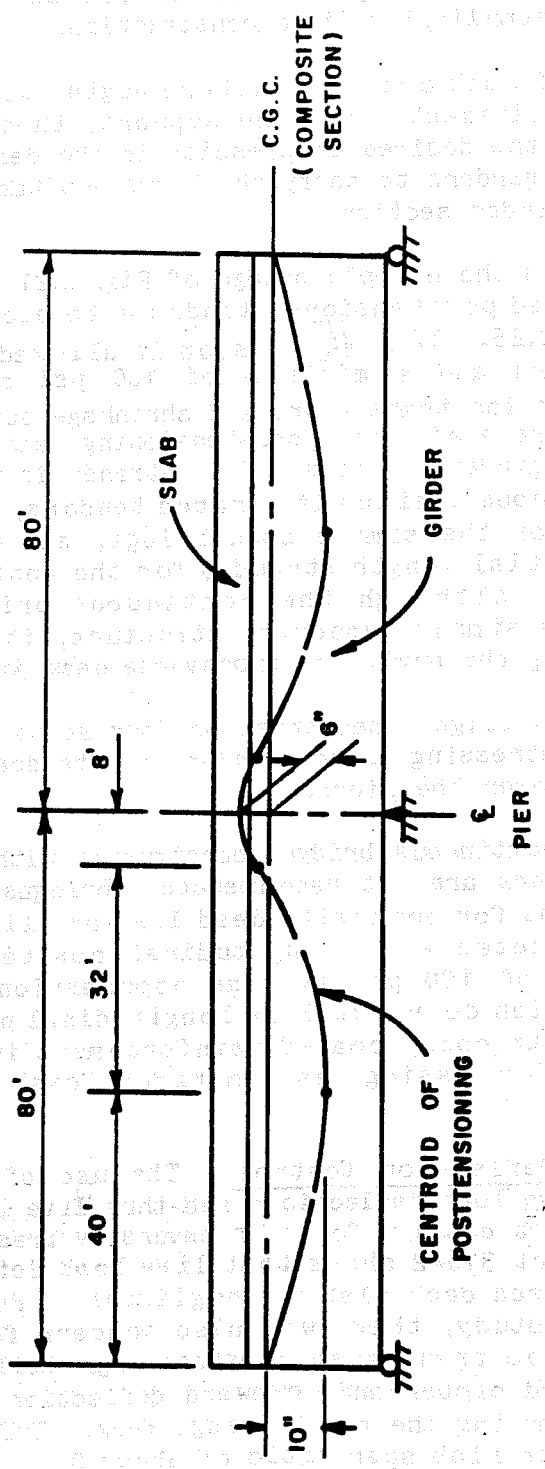


Fig. 2.25 Posttension tendon layout assumed for example bridge

2.2.2.3 Ultimate Strength. In Companion Report 316-2 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.

2.2.2.4 Bonded Versus Unbonded Tendons. There are both advantages and disadvantages in using either an unbonded or a bonded posttensioning system. The results from the durability study indicated that both an unbonded tendon completely surrounded by grease and an integral plastic duct, and a bonded tendon completely surrounded by grout and 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 fatigue behavior of unbonded and bonded prestressing tendons is quite different. Fretting action between prestressing steel, duct, or auxiliary reinforcement at a crack location can result in an amplified steel stress range. As stress range increases, fatigue life decreases. However, for an unbonded tendon, fatigue life is basically independent of cracking and actual tendon stress range levels are much lower than in bonded tendons since the strains are averaged over the full length of the tendon. The fracture critical location for an unbonded tendon is at the anchorages, and in particular at the jaws. The jaws biting into the prestressing tendon form notches which act as local stress risers. For the unbonded tendon case, fatigue fractures could initiate at these notches. Even though the bonded system may be more fatigue sensitive at crack locations, it is known that fractured wires in a strand can rebond in as little as 2 to 3 ft [16], and thus the tendon is not rendered completely useless. In contrast, if fatigue fracture of a tendon occurs at an anchorage in an unbonded system, there is no redistribution possible, and the load-carrying capability of the tendon is completely lost. AASHTO Specification Section 9.26.3 imposes specific fatigue test requirements on unbonded tendons.

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 [11]. Thus, for the same amount of prestressing steel in an unbonded and a bonded member, the ultimate strength of the bonded member will be 10-30% greater [11].

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 for grouting hardware and the cost for 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.

Overall, a bonded posttensioning system would be preferred because of its inherent ductility, structural integrity, and resistance to corrosive attack. When materials were received for the durability test specimens reported in 316-1, a considerable amount of the plastic duct was heavily damaged. In addition, it would be more difficult to protect and inspect the plastic ducting on the job site. Complete encapsulation of the sheath and anchorage system are required for corrosive protection. These considerations, as well as the need for increased bonded reinforcement for general structural integrity when unbonded tendons are used, indicate that the grouted, bonded system is preferable for bridge deck applications.

2.2.2.5 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 reinforcing must be provided. Currently, there are several methods available for the design of prestress tendon anchorage zones. These include that of Guyon [18] as further developed by Leonhardt [19], and that of Stone and Breen [20]. However, neither of these methods seem adequate for the analysis of multiple anchorages in thin slabs. This has been recognized by AASHTO and the National Cooperative Highway Research Program. An extended study of anchorage zone criteria has been proposed and proposals solicited in 1985. Until the results of such

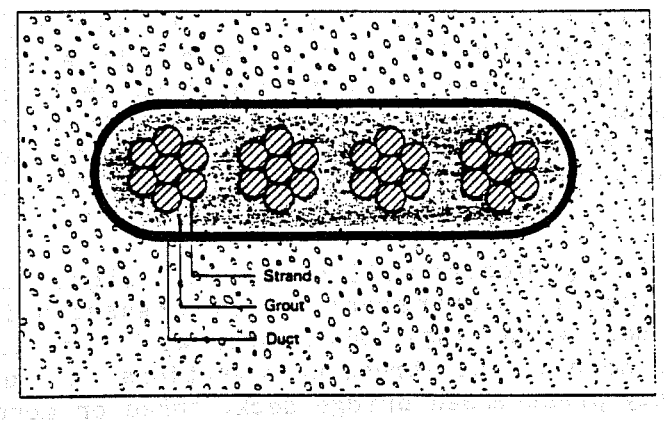
research are available, it is recommended that provisions be included in the project specifications requiring the contractor to show by some means that the proposed anchorage detail is adequate prior to its approval for use. Limited tests done in connection with a prototype bridge planned for La Grange, Texas indicate that the bonded slab reinforcement recommended in this report was adequate as anchorage zone reinforcement for multiple anchorages in an 8" slab. AASHTO Section 9.21 treats "beams" but is silent on "slabs".

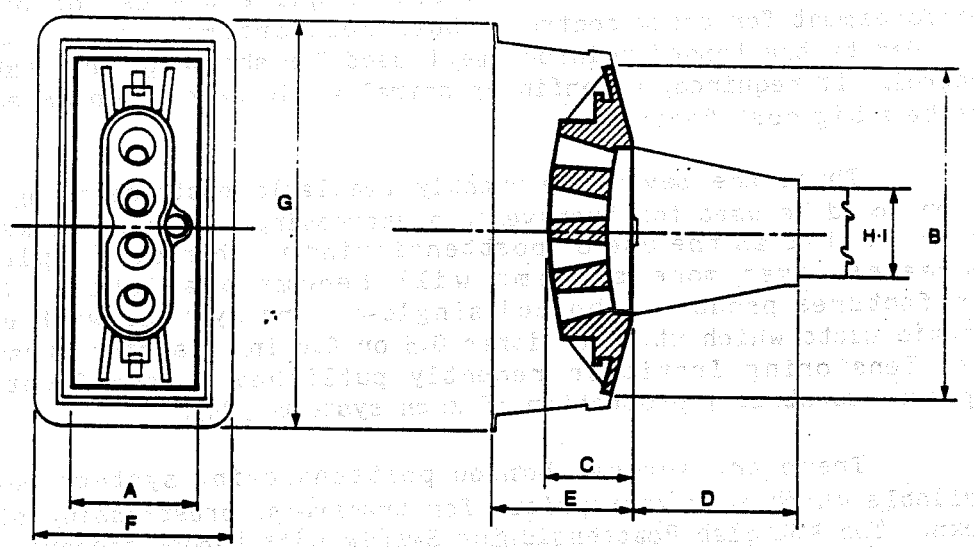
Anchorage design will also have a significant effect on the overall economy of a transversely prestressed bridge deck. Use of multiple tendons with single anchorages will decrease costs since the number of anchorages and stressing operations will also decrease. For a transversely prestressed bridge deck, three or more tendons anchored at a single location would probably require the use of auxiliary reinforcement for crack control. Such reinforcement can probably be provided by the bonded reinforcement used for shrinkage and temperature control. If required, a confining spiral would be no problem and would not be a big cost factor.

There are several currently available posttensioning systems which could be used for transverse prestressing of bridge decks. It is expected that as the use of posttensioning in thin slab applications increases, even more systems will become available. Several manufactures produce unbonded single-strand systems with extruded plastic ducts which utilize either 0.5 or 0.6 in. diameter strand. The Post Tensioning Institute recently published a specification for improved durability protection of such systems [24].

There are several bonded posttensioning systems currently available which seem appropriate for transverse prestressing of bridge decks. The VSL Slab Posttensioning System with bonded tendons shown in Figs. 2.26 and 2.27 appears quite suitable. The system can accommodate up to four 0.5 in. diameter tendons. Other units are available for four 0.6-in. diameter tendons. In this system, the anchorage body and the wedges are not placed in the block-out and on the strands until concreting is complete. In this way, the risk of these components becoming dirty or corroding from exposure before or during concreting is eliminated. The strands can be placed in a flat, smooth, or corrugated plastic duct for the specific purpose of added corrosion protection as an option. The duct is then injected with grout to protect the strands from corrosion.

Dywidag has two different bonded systems shown in Fig. 2.28 which might be used for transverse prestressing. One system utilizes a 0.6 in. diameter strand. The other system is similar to the first system except a Dywidag threaded bar is used as the prestressing steel.



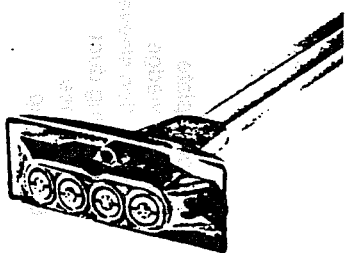


Anchorage Type	VSL SO5-4	SO6-4
A	3.50	3.50
B	11.00	11.00
C	2.87	2.87
D	6.25	6.25
E	5.00	5.00
F	5.62	5.62
G	13.00	13.00
H	3.00	3.00
I	1.00	1.00
J	24.00	24.00
K	4.00	4.00
L	2.50	2.50

Dimensions in inches.

Fig. 2.26 VSL slab posttensioning system with bonded tendons (from VSL Posttensioning Catalog)

This anchorage is used for four-strand grouted tendons placed in flat sheathing. The strands are stressed individually by a monostrand ram and locked off in the anchor head which bears on the embedded plastic form.



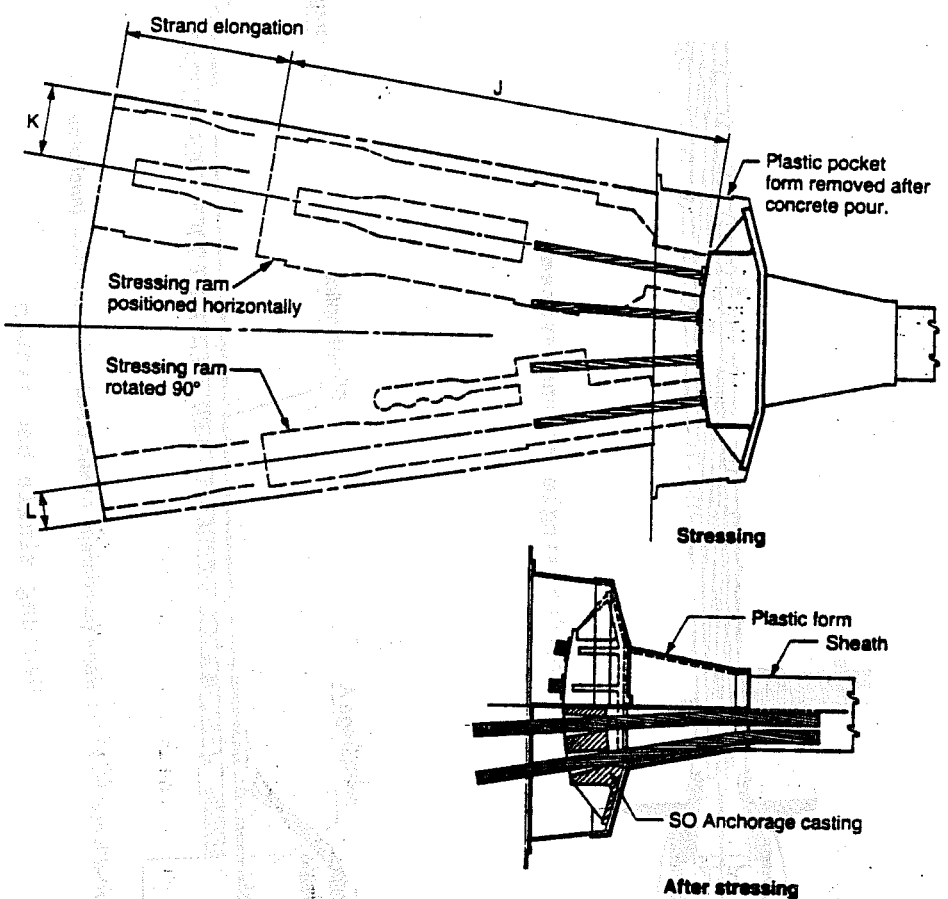
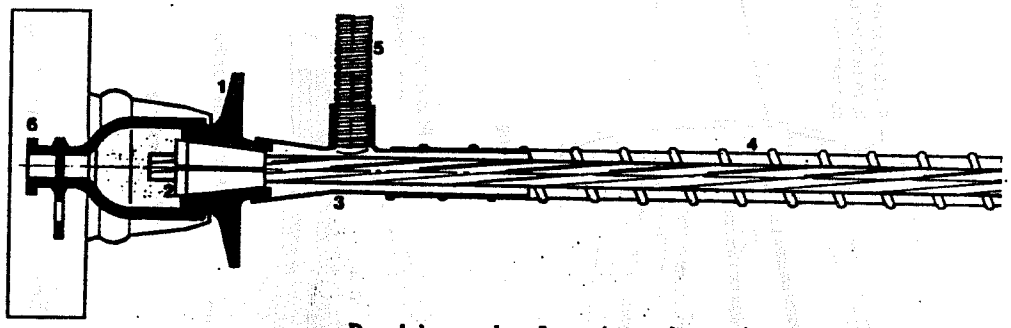
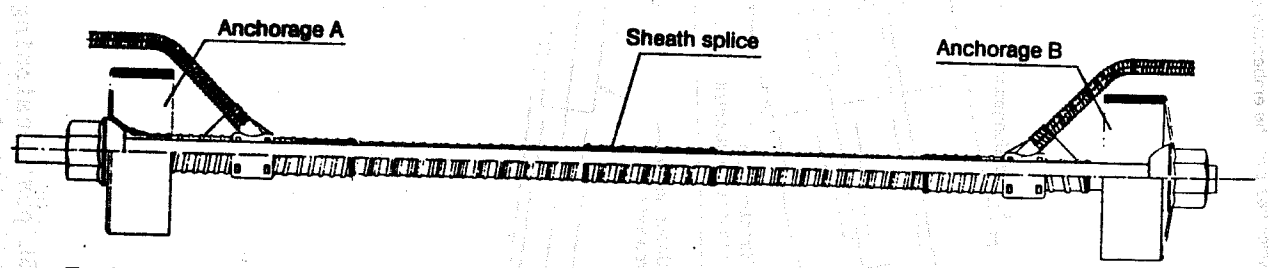


Fig. 2.27 VSL posttensioning system with bonded tendons (continued) (from VSL Posttensioning Catalog)



1 anchor plate
2 anchor wedge
3 connection sleeve – MV
4 sheathing duct
5 grout hose
6 grout cap

Dywidag single strand system



Tendon with live-end anchorage (anchorage A) and dead-end anchorage (anchorage B)

Dywidag single bar system

Fig. 2.28 Dywidag bonded posttensioning systems (from Dywidag Posttensioning Catalog)

Another product which might find wide acceptance in bridge decks is epoxy-coated prestressing strand. This product is commercially available from Florida Wire and Cable.

2.2.2.6 Railing Attachment. There are areas along the sides of a transversely prestressed bridge deck which are ineffectively stressed since the posttensioning 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 railings 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 reinforcing to resist lateral railing impact loads. This reinforcing should provide the full required moment capacity for a transverse distance from the deck edge equal to the tendon spacing.

2.2.2.7 Girder Live Load Distribution Factors. Since a transversely prestressed bridge deck is designed not to crack, it has greater flexural stiffness than a conventionally constructed reinforced concrete deck. The question arises as to whether the distribution of loads to the bridge girders is affected by this higher deck stiffness.

As shown in Report 316-2, the vertical load tests on the laboratory model showed that the transversely prestressed slab behaves essentially linearly elastic through factored loads. Therefore, elastic analysis of the system is appropriate. Sanders [21] developed recommended distribution factors based on elastic analysis studies of bridge systems. His results, summarized in Table 2.11, show that the current AASHTO values for live load distribution factors are applicable to deck systems which behave elastically, such as transversely prestressed bridge decks. Therefore, use of the current AASHTO live load distribution factors is recommended.

2.3 Durability Considerations

In the following sections, the design implications for improving the durability of bridge decks in terms of corrosion protection with the use of transverse prestressing will be discussed. These design implications are primarily based on the results from the durability study presented in companion Report 316-1.

2.3.1 Concrete Cover and Concrete Quality. Even though deck prestressing should eliminate 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

TABLE 2.11 Comparison of Live Load Distribution Factors for Multiple Traffic Lane Bridges by AASHTO and Sanders [21]

Bridge Type	Span Range (ft)	Bridge Width (ft)	Value of D	
			AASHTO	Sanders
Slab on composite steel I-beam	41- 90	29-37	5.5	5.3-5.7
Slab on noncomposite steel I-beam	50- 70	25-30	5.5	5.9
Concrete T-beams	40- 70	29-36	6.0	5.9-6.1
Slab on prestressed concrete I-beam	35-100	29-37	5.5	5.2-5.9

Live load distribution factor = S/D

where S = girder spacing (ft)

results indicate that the combination of a 2 in. cover and a water-cement ratio (W/C) of about 0.45 was adequate for corrosion protection to resist the aggressive exposure of the tests. However, Weed [25] 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 3/8 in. for a 2 in. cover. This implies that about 15% of the steel could be expected to have a cover less than 1-5/8 in. 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 value is 2 in. when deicers are used. This is very marginal as a practical all-inclusive requirement. Setting a minimum clear cover value of 2.5 in. would ensure that most steel would have at least a 2 in. cover, which would be at an acceptable corrosion risk. Therefore, it is proposed that a minimum 2.5 in. 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 [7]. The current AASHTO Section 9.25.1 recommendation of 1 in. 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 a threat of salt exposure at the bottom slab surfaces, such as in a marine environment, 2.5 in. bottom cover is also recommended.

2.3.2 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 posttensioning tendons is found in Refs. 22, 23, 24, and 26. 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.

2.3.3 Anchorage Protection. Maintaining a minimum 2.5 in. concrete cover around all surfaces of an anchorage would normally provide adequate corrosion protection. However, a minimum 2.5 in cover over the prestressing ducts will likely result in less concrete cover over some areas of the anchorage. For the durability specimens with a concrete cover of 2 in. over the prestressing, only 3/4 in. 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 posttensioning, 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 [26].

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 1-1/2 in. 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.

2.3.4 Cl^- Content. The durability study test results 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%. The limit on Cl^- content would be verified by trial mix on test samples.

2.4 Recommended Change in Current Design Procedure for Conventionally Reinforced Concrete Decks

The durability study results indicate that the current AASHTO provisions [6] (Sec. 8.16.8.4) for the design of conventionally reinforced concrete decks are inadequate for protection from chloride-induced corrosion. These design provisions indirectly permit crack widths in the concrete of about 0.011 in. which could lead to corrosion. Without changing any of the provisions directly, it is recommended that epoxy-coated reinforcing bars be used at least for the top mat of reinforcement in conventionally reinforced bridge decks exposed to chlorides in service.

2.5 Proposed AASHTO Provisions for Transversely Prestressed Bridge Decks

Because the AASHTO Specifications are minimum requirements for bridge design, some of the ideas included in the previous sections are

not fully represented in the suggested provisions. Specifically, neither longitudinal posttensioning of the deck, reduced allowable longitudinal concrete tensile stresses in box girder bridge decks, nor epoxy-coated reinforcing are expressly required. In addition, design details for continuous bridges and tendon placement in skewed slabs have been omitted.

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 [15]. There are basically three limit state 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 bridge deck.

The proposed AASHTO provisions assume that all other portions of the AASHTO Specifications 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.

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 ft width of slab, $in.^2$

C_K = correction factor for 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 cross-sectional area of diaphragm resisting axial deformation ($in.^2$)

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

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 lines 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 Sec. 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

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

Two-Cell Box Section

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

Three-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 Sec. 1.3.2.3 or Sec. 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 $C_t = 8/t$ (1.3.2.3-2)

$C_K = (EA)_D/640{,}000$ (1.3.2.3-3)

and $C_L = 25/S_D$ (1.3.2.3-4)

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 one foot length of slab.

Unless an analysis is carried out in accordance with Sec. 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 one foot edge length of slab in the diaphragm regions required to overcome diaphragm restraining effects shall be not less than:

For bridges with $\theta \leq 10^o$

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

For bridges with $\theta > 10^{\circ}$

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

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

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

where W and L are in units of feet. For end diaphragm regions, x shall be measured from the transverse slab edge on non-skew 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). When more precise information is not available, 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

$$A_B = 0.024t \qquad (1.6\text{-}1)$$

1.7 Distribution Reinforcement for Slab-Girder Bridges

1.7.1 For slab and girder bridges, longitudinal distribution reinforcement in the bottom of a transversely prestressed bridge deck shall be provided to resist at least 1/4 the maximum design transverse live load plus impact slab moment.

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

$$A_L \geq (0.03)t \qquad (1.7.2\text{-}1)$$

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 Sec. 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 stressed 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% 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 2-1/2 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) [24].

1.11.5 After placement of prestressing 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 1-1/2 in. of concrete or mortar 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.

CHAPTER 3

DESIGN EXAMPLES

3.1 Introduction

The design of prestressed bridge decks using the proposed recommendations of Chapter 2 is illustrated in this chapter with three design examples. In the first example, the prototype structure of the laboratory bridge model described in Report 316-2 is designed with transverse prestressing according to the proposed recommendations. For this case, both prestressing of the diaphragm and amplified slab prestressing in the diaphragm regions will be illustrated to provide a comparison of the two main ways to overcome restraining effects of the diaphragms. The other two bridges selected for examples have been constructed in Texas with conventionally reinforced decks. Redesign of these actual cases allows a direct and convenient comparison between a prestressed deck (with both longitudinal and transverse tendons) and a conventionally reinforced slab.

Prestressed bridge decks follow a design procedure much the same as that for a reinforced concrete deck. Aside from the additional calculations associated with prestressed concrete design, the only departures from conventional design procedure are the determination of the prestressing required to compensate for diaphragm restraint, and the calculations for longitudinal deck prestressing when it is used. The calculations for longitudinal deck prestressing in continuous bridges are somewhat more involved, but are very similar to those for posttensioning concrete box girder bridges. A summary of the major design steps for a transversely prestressed bridge deck is shown in Fig. 3.1.

The deck overhangs on the example bridges are small compared to the interior slab spans. Stresses in the slab overhangs were checked and, as might be expected, did not govern. The overhang calculations are omitted here for brevity. Also, in the calculations for longitudinal deck prestressing, stresses would usually be checked at each tenth point of the span. In these examples they are checked only at midspan and pier locations for brevity.

Like the bridge it was modeled after, the study bridge of example 1 is assumed to be in an aggressive marine environment. Therefore, 2-1/2 in. of concrete cover are required on both top and bottom of the deck. The bridges of the second and third examples are assumed to be exposed to deicing chemicals. Then, 2-1/2 in. concrete cover is required only for the top reinforcing steel of those decks.

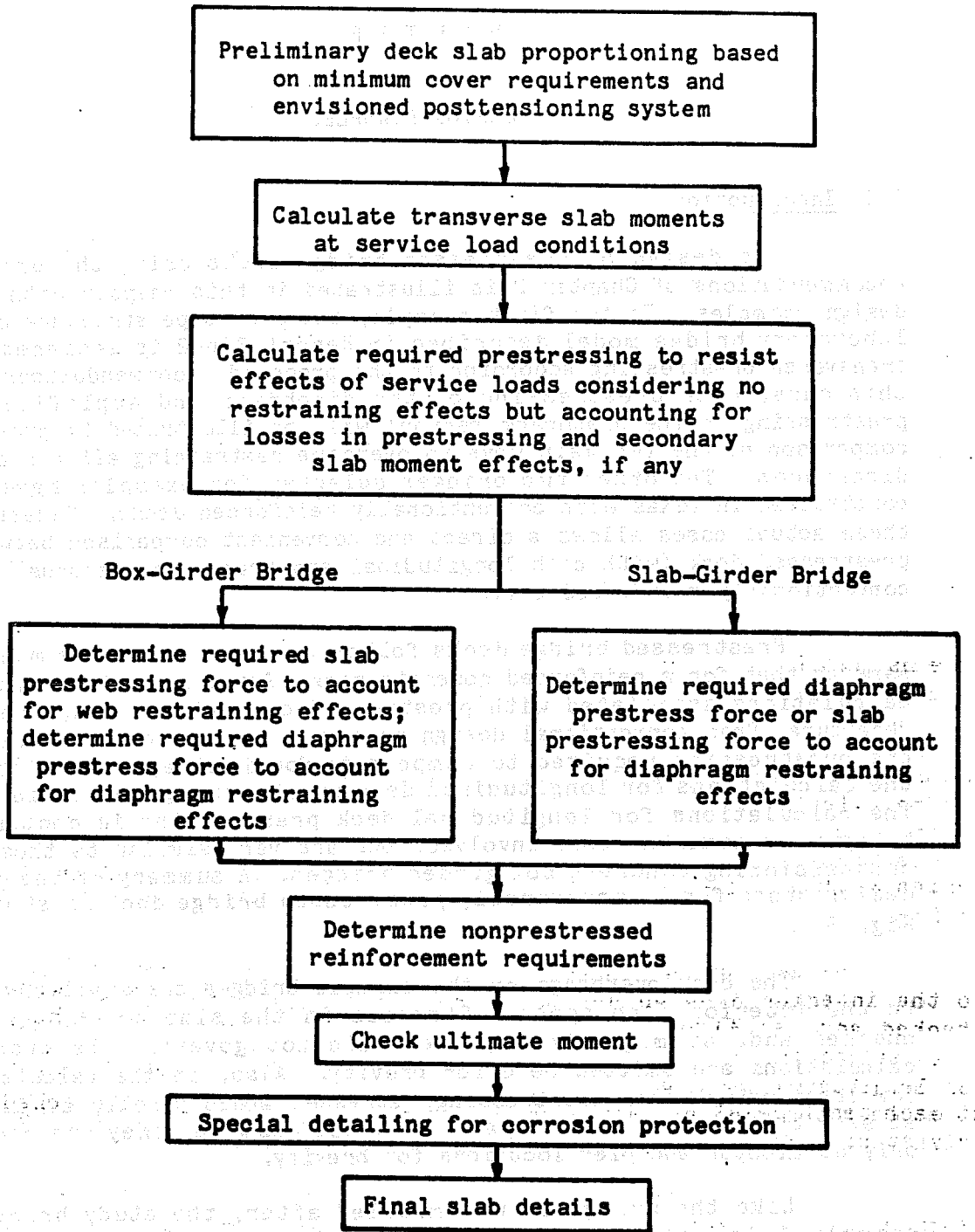


Fig. 3.1 Summary of major design steps of a transversely prestressed bridge deck

Throughout the examples, references to the proposed design provisions from Sec. 2.5 are prefixed by a "P" for "Proposed Specification Provisions", while those cited from the current AASHTO Specifications are so designated.

3.2 Design Example 1: Simple Span Nonskew Bridge

Figure 3.2 shows an overall view of the transversely prestressed bridge of example 1. The VSL Slab Posttensioning System with bonded tendons previously shown in Fig. 2.23 is assumed for the transverse prestressing. The other basic material properties assumed for the design are shown in Table 3.1. Since the overall length of this span is 76 ft, interior diaphragms will be furnished at the third points of the span so that the interior diaphragm spacing, S_D, may be taken as 25 ft.

3.2.1 Preliminary Proportioning. To meet the minimum cover requirements specified in P-1.11.3 for the reinforcement and in P-1.11.6 for the anchorages, an absolute minimum slab thickness of 6.5 in. is required. This would virtually correspond to a solid steel sandwich with cover. To provide some distance between the top and bottom mats of reinforcement, a 7.5 in. slab is assumed. A preliminary reinforcement placement which meets the cover requirements is shown in Fig. 3.3. A minimum cover of 2.5 in. is required for both top and bottom reinforcement since chlorides could penetrate either direction in a marine environment. Draping the tendons would have little benefit in this thin slab; therefore, straight middepth tendons are assumed.

3.2.2 Interior Deck Span. According to AASHTO 3.24.1.2(a), the effective interior slab span is calculated to be:

$$S = \text{clear span} = \text{girder spacing} - \text{girder flange width}$$

$$= 8.83 - (14/12)$$

$$= 7.66 \text{ ft}$$

3.2.3 Service Loads

3.2.3.1 Dead Load Moment. Assuming the density of prestressed concrete to be 0.15 kips/ft^3, the dead load moment, M_{DL}, is computed as follows:

$$M_{DL} = 0.15\,(7.5/12)((7.66)^2/10)$$

$$= 0.55 \text{ k-ft/ft}$$

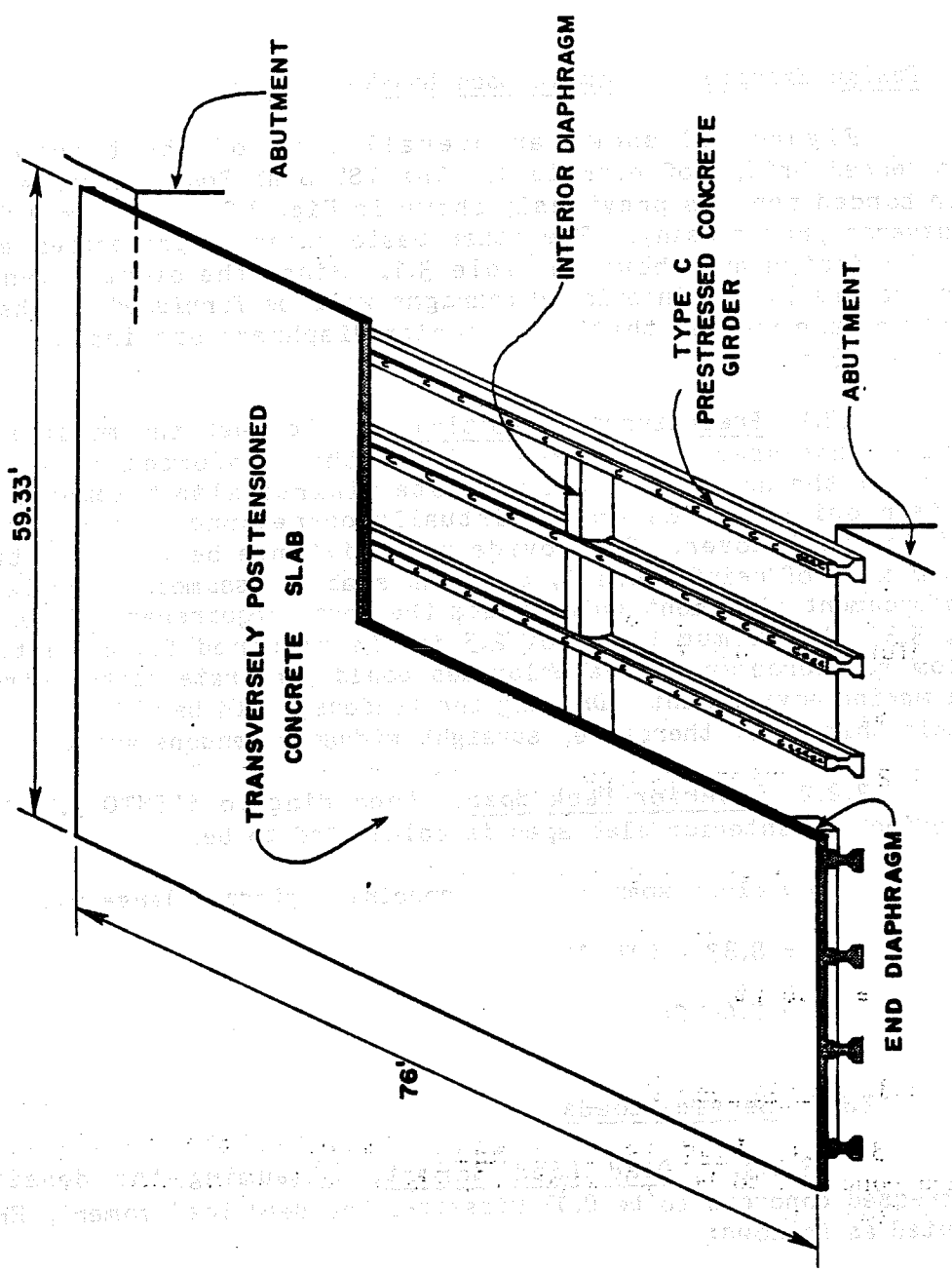


Fig. 3.2 View of bridge of Example 1

TABLE 3.1 Basic Material Properties Assumed for Transversely Prestressed Bridge Deck Design Example 1

Concrete	Compressive Strength f'_c = 5 ksi	
	Water/Cement Ratio W/C = 0.45	See P- 1.11.1
Bonded Nonprestressed Reinforcement	Yield Strength f_y = 60 ksi	
Prestressing	Ultimate Strength f_{pu} = 270 ksi	
	1/2 in. Diameter Strands--Strand Area A_{ps} = 0.153 in.^{2}	
Posttensioning Ducts	Impenetrable by Moisture	See P- 1.11.4
	Wobble Friction Coefficient	
	Grouted Multistrand Posttensioning System, K = 0.002/ft	

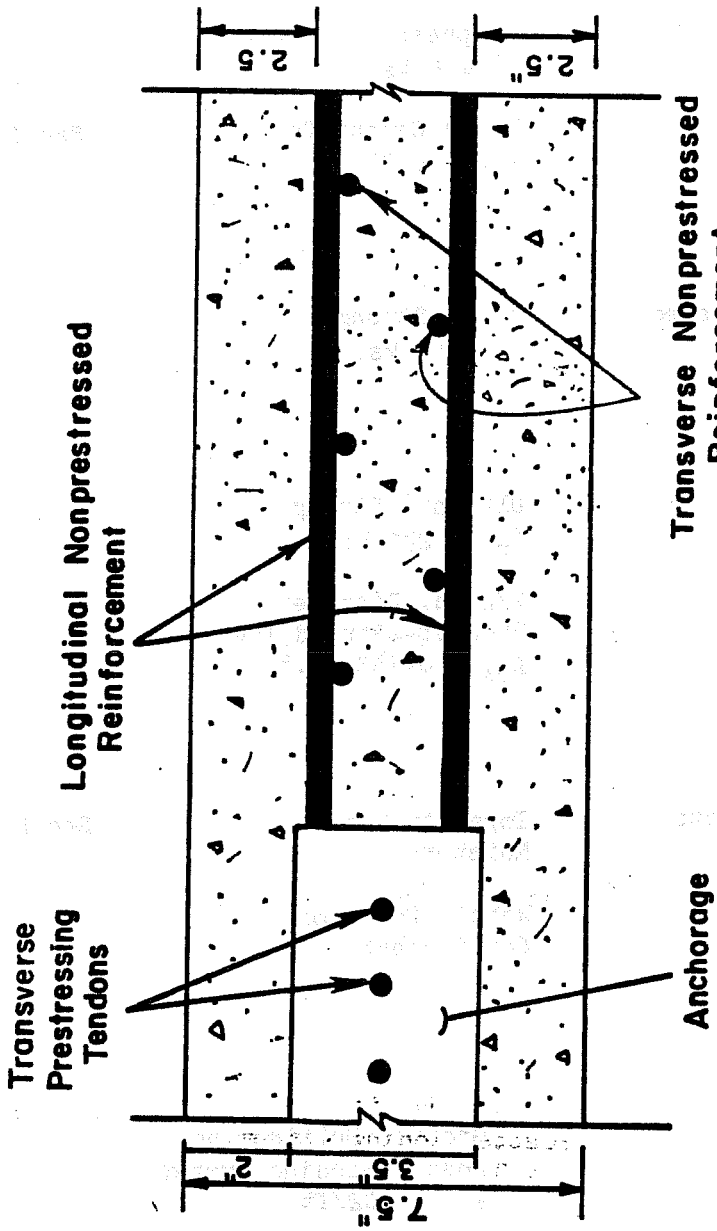


Fig. 3.3 Preliminary reinforcement placement

Consistent with current practice, partial continuity is assumed since the deck slab is supported by flexible girders. Therefore, a value of 10 is used in the moment calculation.

3.2.3.2 Impact Factor. AASHTO Sec. 3.8.2.1 requires a magnification of live load moments due to dynamic effects equal to the following:

$$I = 50/(125 + S) \leq 0.30$$

For S = 7.66 ft, I equals 0.30.

3.2.3.3 Live Load Moment. Assuming AASHTO HS20-44 loading (see AASHTO 3.7.7), the wheel group load, P, is 16 kips. Then, according to AASHTO 3.24.3, the live load moment, M_{LL}, is calculated as follows:

$$M_{LL} = (1 + I)(0.8)(S + 2)(16)/32$$

$$= (1 + 0.3)(0.8)(7.66 + 2)(16)/32$$

$$= 5.02 \text{ k-ft/ft}$$

3.2.3.4 Total Load Moment. Total load moment, M_{TL}, is the summation of dead and live load moments. Thus,

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

$$= 0.55 + 5.02$$

$$= 5.6 \text{ k-ft/ft}$$

3.2.4 Prestressing

3.2.4.1 Losses. To comply with P-1.3.2.1, prestressing losses must be taken into account. The VSL anchorage can accommodate four 1/2 in. diameter tendons. Assuming a maximum jacking stress of 0.8 f_{pu}, the posttensioning force, T_x, before friction losses is:

$$T_x = 4(0.153)(0.80)(270)$$

$$= 132 \text{ kips}$$

The prestressing force for the tendon group considering friction losses and jacking from one end only is computed by AASHTO 9.16.1:

$$T_o = T_x\, e^{-(KL + \mu\alpha)}$$

$$= 132\, e^{-(0.002(59.3))}$$

$$= 117 \text{ kips}$$

However, the steel stress must be no greater than 0.7 f_{pu} (AASHTO 9.15.1):

$$0.7(270)4(0.153) = 116 \text{ kips}$$

$$116 \text{ kips} < 117 \text{ kips}$$

$$T_o = 116 \text{ kips}$$

Assuming a detailed analysis under AASHTO Sec. 9.16.2.1 with Lo-lax strand shows that other prestressing losses due to creep, shrinkage, relaxation, and elastic shortening total 20%, then the effective prestressing force per tendon group is 93 kips.

3.2.4.2 Secondary Moment Effects. According to P-1.3.2.1, the influence of secondary moments on transverse stress distribution must be examined. However, since the design calls for middepth tendons, there are no secondary moment effects.

3.2.4.3 Required Prestressing for Service Loads. The moments and stresses for an interior slab panel are shown in Fig. 3.4. According to P-1.5, the tensile stresses under service loads cannot exceed $2\sqrt{f'_c}$ which is 0.141 ksi for 5000 psi concrete. Therefore, the required prestressing is calculated as follows:

$$-F_S/A + 0.597 = 0.141$$

$$-F_S/((7.5)(12)) = -0.46$$

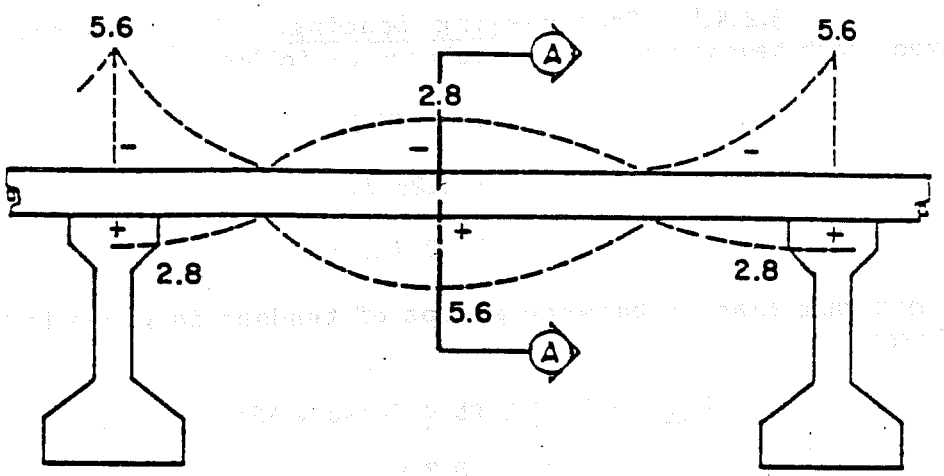
$$F_S = 41.3 \text{ kips/ft}$$

The compression stresses in the slab under service load conditions need to be checked and are limited by AASHTO 9.15.2.2.

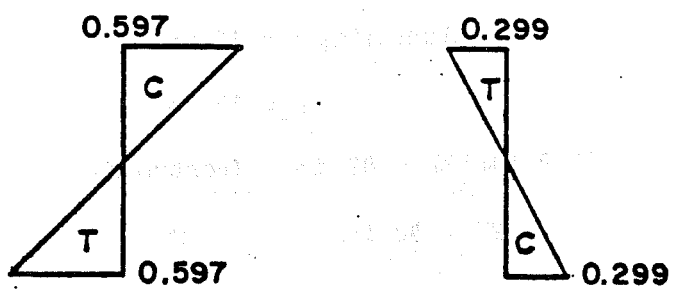
$$f_c = -P/A - M/S \leq 0.4\, f'_c$$

$$= -41.3/((7.5)(12)) - 5.6(12)/112$$

$$= -1.06 \text{ ksi}$$



a) "Moment Envelope" (k-ft/ft)



due to positive moment = 5.6 k-ft/ft

due to negative moment = 2.8 k-ft/ft

b) Stresses (ksi) at Section A-A

Stresses Based on Gross Concrete Section

Fig.3.4 Transverse moments and stresses for an interior slab panel

$$0.4\ f'_c = 2\ \text{ksi}$$

$$1.06 < 2 \qquad \text{OK}$$

3.2.4.4 Prestressing Spacing. The required spacing between each tendon group is calculated as follows:

$$S = 93/41.3$$

$$= 2.25\ \text{ft}$$

$$\cong 27\ \text{in.}$$

The maximum spacing between groups of tendons is given by P-1.4 as follows:

$$S_{max} = 8t \leq 5\ \text{ft} \leq 3(y-a+12)$$

$$8t = 8(7.5)$$

$$= 5\ \text{ft}$$

Assuming $y = 12$ in.

$$a = 10\ \text{in.}$$

$$3(12-10+12) = 42\ \text{in.} \quad \text{(controls)}$$

$$27 < 42\ \text{in.} \qquad \text{OK}$$

3.2.4.5 Compensating for Diaphragm Restraining Effects

3.2.4.5.1 By Using Amplified Prestressing in the Slab. According to P-1.3.2.1, the restraining effects of diaphragms must be included in the design. The approximate procedure of P-1.3.2.4 is first illustrated.

According to P-1.3.2.4, the required amplified prestress force per foot required in diaphragm regions for a bridge with no skew is given by the following:

$$F_e = 1.6\ F_S$$

F_S was previously calculated as 41.3 kips/ft. Therefore:

$$F_e = 1.6(41.3)$$

$$= 66\ \text{kips/ft}$$

This amplified prestressing force is required in the slab throughout the diaphragm regions. This amplified prestressing must be applied for a slab width in the diaphragm regions as given by:

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

In this case $\theta = 90°$, L = 75 ft, and N = 4 so that x = 4 ft. The tendon group spacing in these diaphragms regions is calculated as follows:

$$S = 93/66$$

$$= 1.41 \text{ ft}$$

$$= 17 \text{ in.}$$

In the diaphragm region, the compressive slab stresses for an interior slab span would still be 1.06 ksi as calculated previously in Sec. 3.2.4.3 since the slab would feel only the unamplified portion of the increased force. This is less than the AASHTO 9.15.2.2 limit of 0.4 f'_c. The compressive stresses in the cantilever portion of the slab in the diaphragm region would be greater than in the interior slab regions since the slab is unrestrained there, and therefore would need to be checked. However, for this example, the design is limited to the interior span.

3.2.4.5.2 By Prestressing Diaphragms. The alternate procedure for counteracting diaphragm restraints is illustrated in this subsection. Only one of the procedures needs to be used in slab-girder bridges. According to P-1.3.2.3, the prestress force required in the diaphragms is given by the following:

$$P_D = C_t C_K C_L \, 1.6 \, F_S$$

In this case:

$$C_t = 8/7.5$$

$$= 1.07$$

$C_K = 1$ (standard concrete diaphragms assumed)

$$C_L = 25/25$$

$$= 1$$

Therefore:

$$P_D = 1.07\ (1)\ (1)\ 1.6\ F_S$$

$$= 1.71\ F_S$$

F_S was previously calculated as 41.3 kips/ft.

According to P-1.3.2.3, F_S is calculated for a 1 ft width of slab. Therefore:

$$F_S = (41.3 \text{ kips/ft})\ (1 \text{ ft})$$

$$= 41.3 \text{ kips}$$

This yields P_D as follows:

$$P_D = 1.71\ (41.3)$$

$$= 71 \text{ kips}$$

Therefore, the prestressing force required in each diaphragm line to overcome diaphragm restraining effects is approximately 71 kips. Unless an analysis is carried out according to P-1.3.2.2, this prestress force must be applied at a distance not exceeding 1/12 the height of the diaphragm from the centroid of the diaphragm.

3.2.5 Bonded Nonprestressed Reinforcement

3.2.5.1 Shrinkage and Temperature Reinforcement AASHTO 8.20 requires a minimum area of shrinkage and temperature reinforcement at reinforced concrete slab surfaces not otherwise reinforced. But P-1.8.1 extends this to imply that this minimum is required near both top and bottom deck surfaces in both longitudinal and transverse directions with the use of bonded nonprestressed reinforcement. According to AASHTO 8.20.2:

$$A_t = 1/8 \text{ in.}^2/\text{ft}$$

This area of reinforcement translates into #3 reinforcing bars spaced at 10-1/2 in.

3.2.5.2 Longitudinal Distribution Steel P-1.7 requires uniformly-distributed, bonded nonprestressed reinforcement in the bottom of the slab in the longitudinal direction. The minimum area of bonded reinforcement in this bottom layer is given by P-1.7.2:

$$A_L = (0.03)\ t$$

$$= 0.03\ (7.5)$$

$$= 0.23 \text{ in.}^2/\text{ft}$$

This area of reinforcement translates into #4 reinforcing bars spaced at 10 in. This amount of distribution reinforcement will satisfy the shrinkage and temperature reinforcement requirement at the bottom of the slab in the longitudinal direction.

3.2.6 Check Ultimate Moment. The ultimate moment capacity of the slab per foot, M_u, is computed according to AASHTO 9.14 and 9.17.2.

$$M_u = \phi A_{ps} f_{su} d (1-(0.6 \rho f_{su})/f'_c)$$

where ϕ = phi factor, 0.95 for cast-in-place posttensioned concrete,

A_{ps} = prestressing steel area,

d = distance from extreme compression fiber to centroid of prestressing force,

$\rho = A_{ps}/bd$

$f_{su} = f_{pu} (1-(0.5 \rho f_{pu}/f'_c))$ (for bonded tendons)

In this case:

$$A_{ps} = 4 (0.153)(12)/27$$
$$= 0.27 \text{ in.}^2/\text{ft}$$
$$d = 7.5/2$$
$$= 3.75 \text{ in.}$$
$$\rho = 0.27/((12)(3.75))$$
$$= 0.0060$$
$$f_{su} = 270 (1-(0.5(0.0060)(270)/5))$$
$$= 226 \text{ ksi}$$

Thus:

$$M_u = 0.95(0.27)(226)(3.75)(1-(0.6(0.0060)226)/5)$$
$$= 182 \text{ in.-kips/ft}$$
$$= 15.2 \text{ k-ft/ft}$$

The factored moment per foot, M_{uf}, is calculated according to AASHTO 3.22:

$$M_{uf} = 1.3\ M_{DL} + 2.17\ M_{LL}$$

$$= 1.3(0.55) + 2.17(5.02)$$

$$= 11.6 \text{ kip-ft/ft}$$

Therefore,

$$M_u > M_{uf}$$

which implies ultimate strength conditions are satisfied.

3.2.7 Check Minimum and Maximum Steel Percentages. The minimum steel percentage of AASHTO 9.18.2 is satisfied by inspection.

The maximum steel percentage is checked according to AASHTO 9.18.1:

$$\rho\ f_{su}/f'_c \leq 0.30$$

$$(0.0060)(226)/5 = 0.27$$

$$0.27 < 0.30 \qquad \text{OK}$$

Thus, the steel percentage is below the maximum allowed for the nondiaphragm regions of the slab. For the diaphragm regions where more closely spaced tendons are used, the maximum steel percentage is exceeded. In this case, M_u is limited by the compression couple as described in AASHTO 9.18.1.

$$M_u = \phi\ 0.25\ f'_c\ bd^2$$

$$= 0.95(0.25)(5)(12)(3.75)^2$$

$$= 200 \text{ in.-kips/ft}$$

$$= 16.7 \text{ k-ft/ft}$$

This M_u exceeds the previously calculated value for M_{uf}; thus the design is acceptable.

3.2.8 Special Detailing for Corrosion Protection. For corrosion protection, special detailing requirements as described in P-1.11 must be met. Figure 3.5 illustrates these special detailing

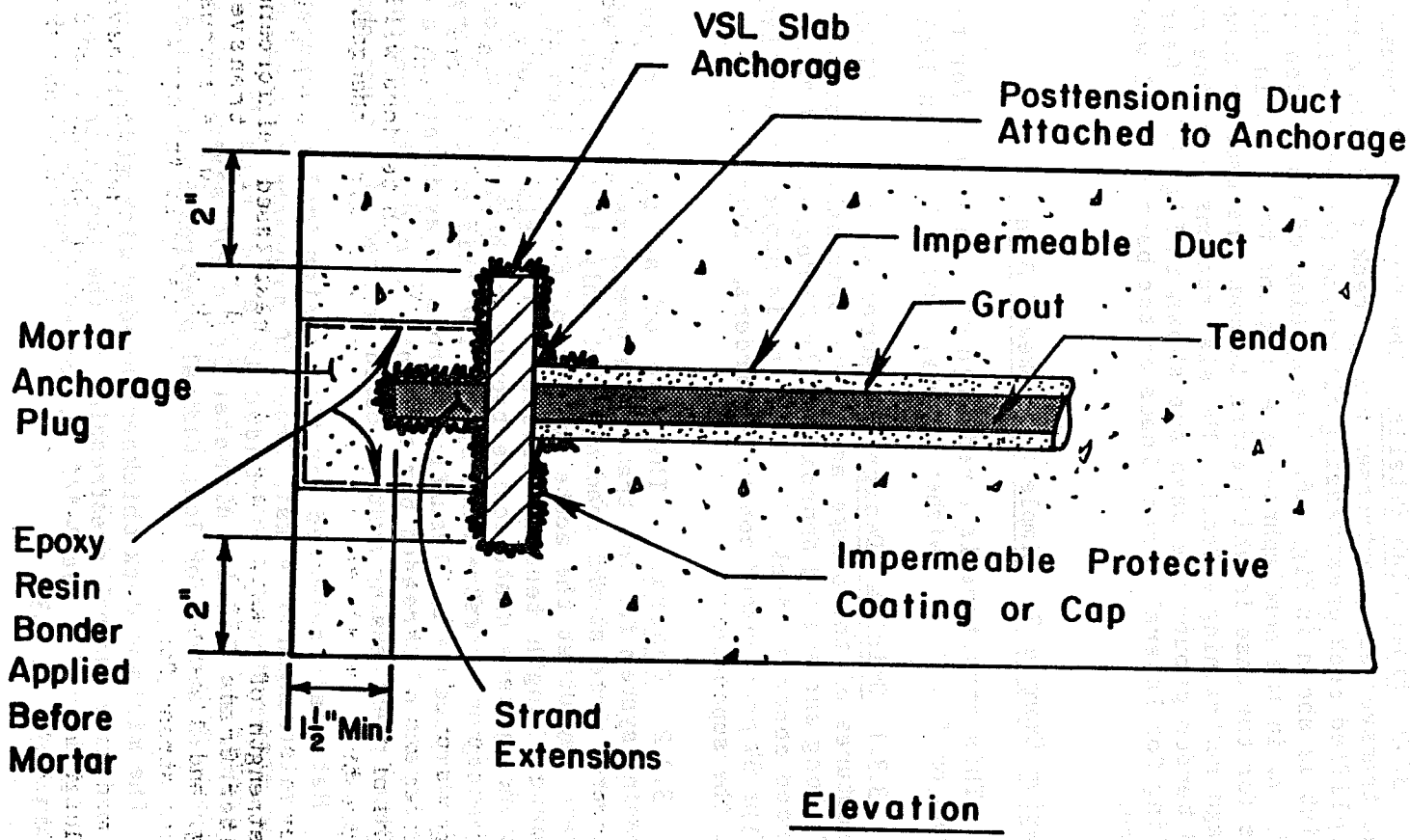


Fig. 3.5 Special detailing requirements

requirements which will ensure good performance for the anchorages and posttensioning system.

3.2.9 Final Slab Details. The final reinforcement detailing for the transversely prestressed bridge deck of design example 1 with the amplified deck prestressing alternate for overcoming diaphragm restraint is shown in Fig. 3.6. The spacing of the tendon group is cut from 27 in. in the nondiaphragm slab regions to 17 in. in the diaphragm regions for the case in which extra tendons are used to compensate for diaphragm restraining effects. The slab regions where the more closely spaced tendon groups are used are shown in Fig. 3.7. Reinforcement detailing for lateral impact loads on the barrier wall would also be required.

3.3 Design Example 2: Simple Span Skewed Bridge

This structure is a redesign of an end span for the F.M. 214 underpass on I.H. 27 in Swisher County, Texas, built in 1983.

3.3.1 Original Design. Figures 3.8 and 3.9 illustrate the major features of the bridge as it was originally designed. The span is 55 ft long and skewed approximately 21 degrees. A 7-1/2 in. thick reinforced concrete deck, 36 ft wide, is supported on five lines of Texas Type 54 prestressed concrete girders. Diaphragms are provided only at the supports.

3.3.2 General. The use of an unbonded monostrand prestressing system for 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 1/2-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. 3.10 The 2-1/2 in. concrete cover on the top surface is required by P 1.11.3.

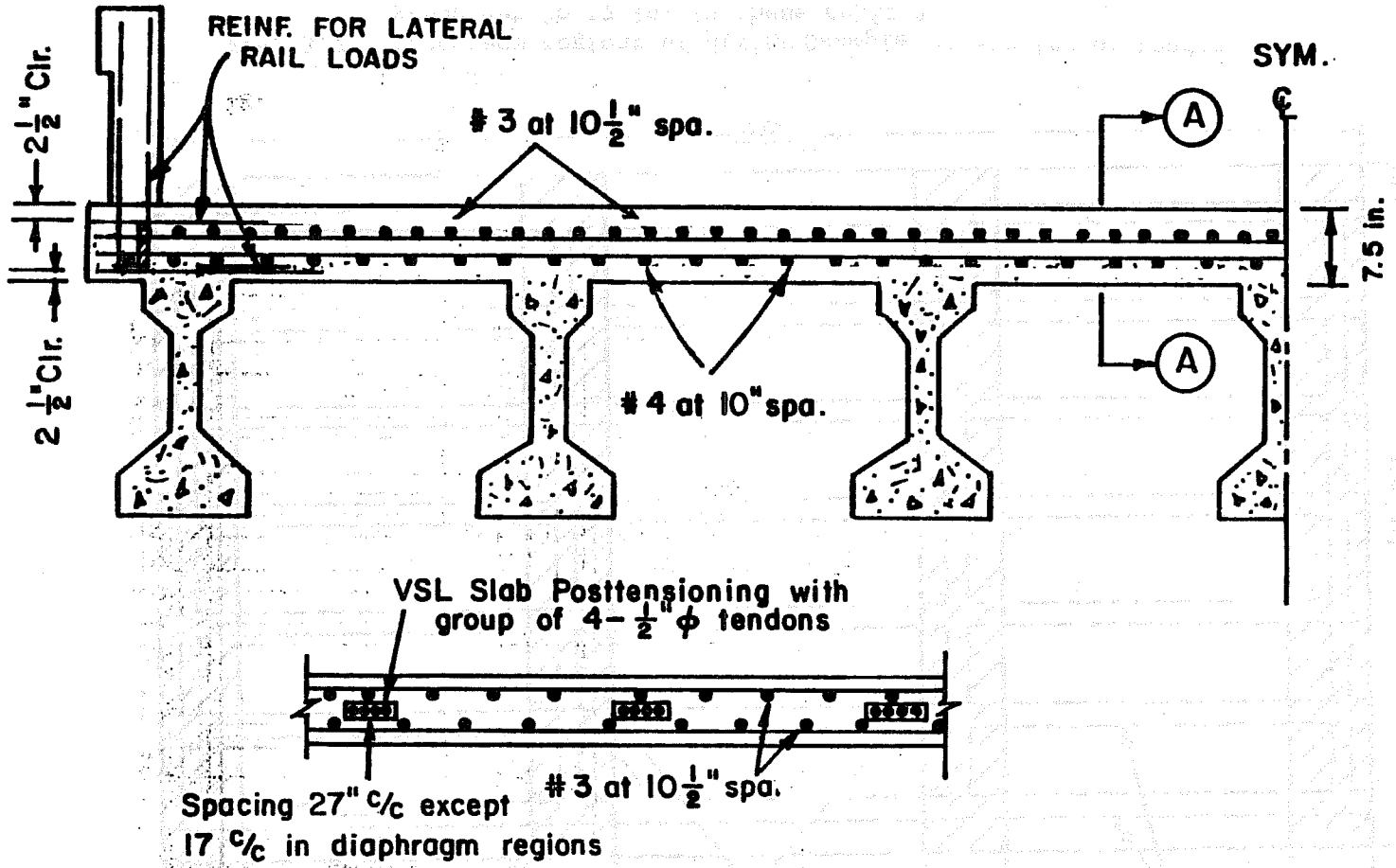


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Fig. 3.6 Reinforcement detailing of transversely prestressed bridge deck of design Example 1

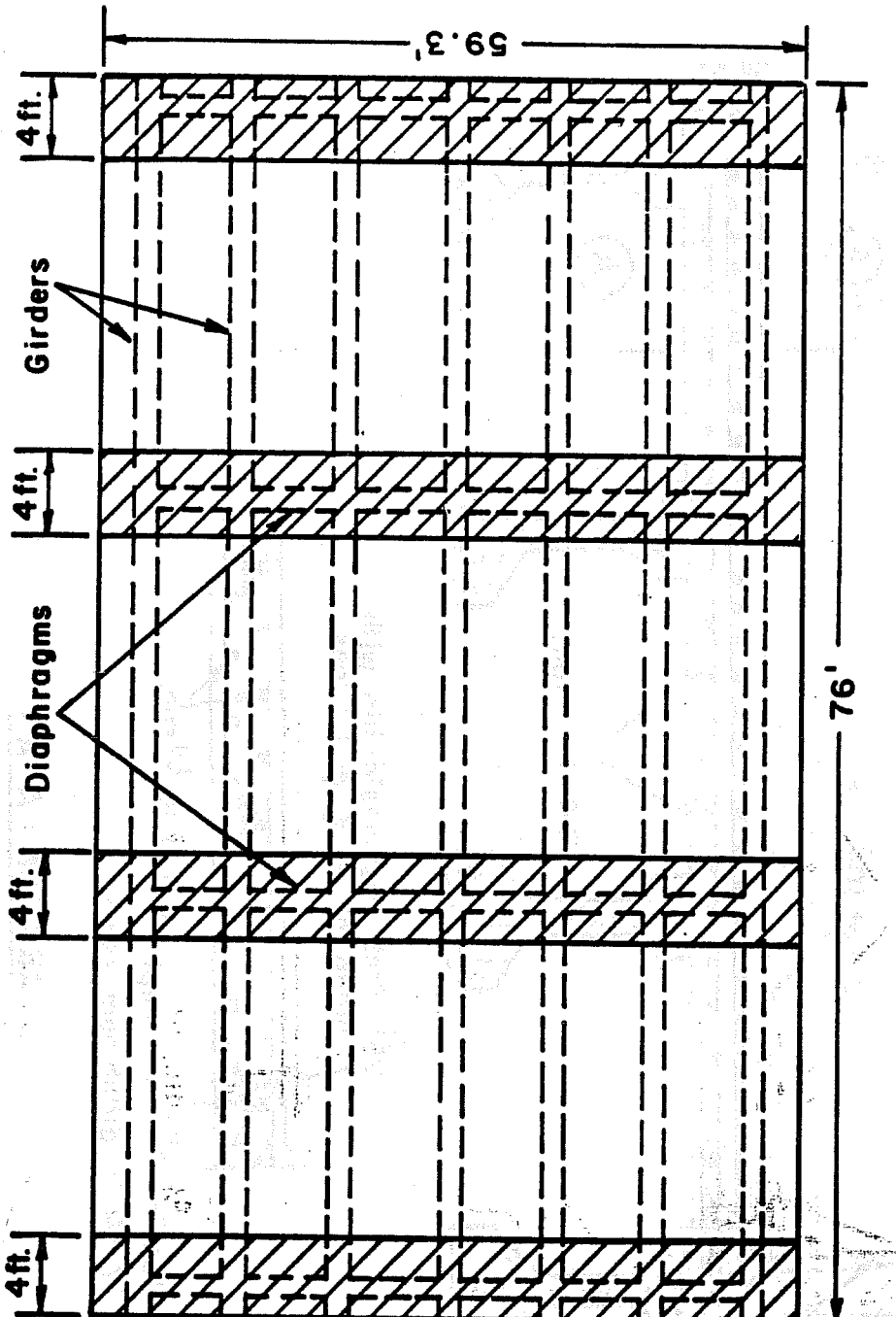


Fig. 3.7 Diaphragm regions of design Example 1; spacing of tendon group cut to 17 in. in these regions

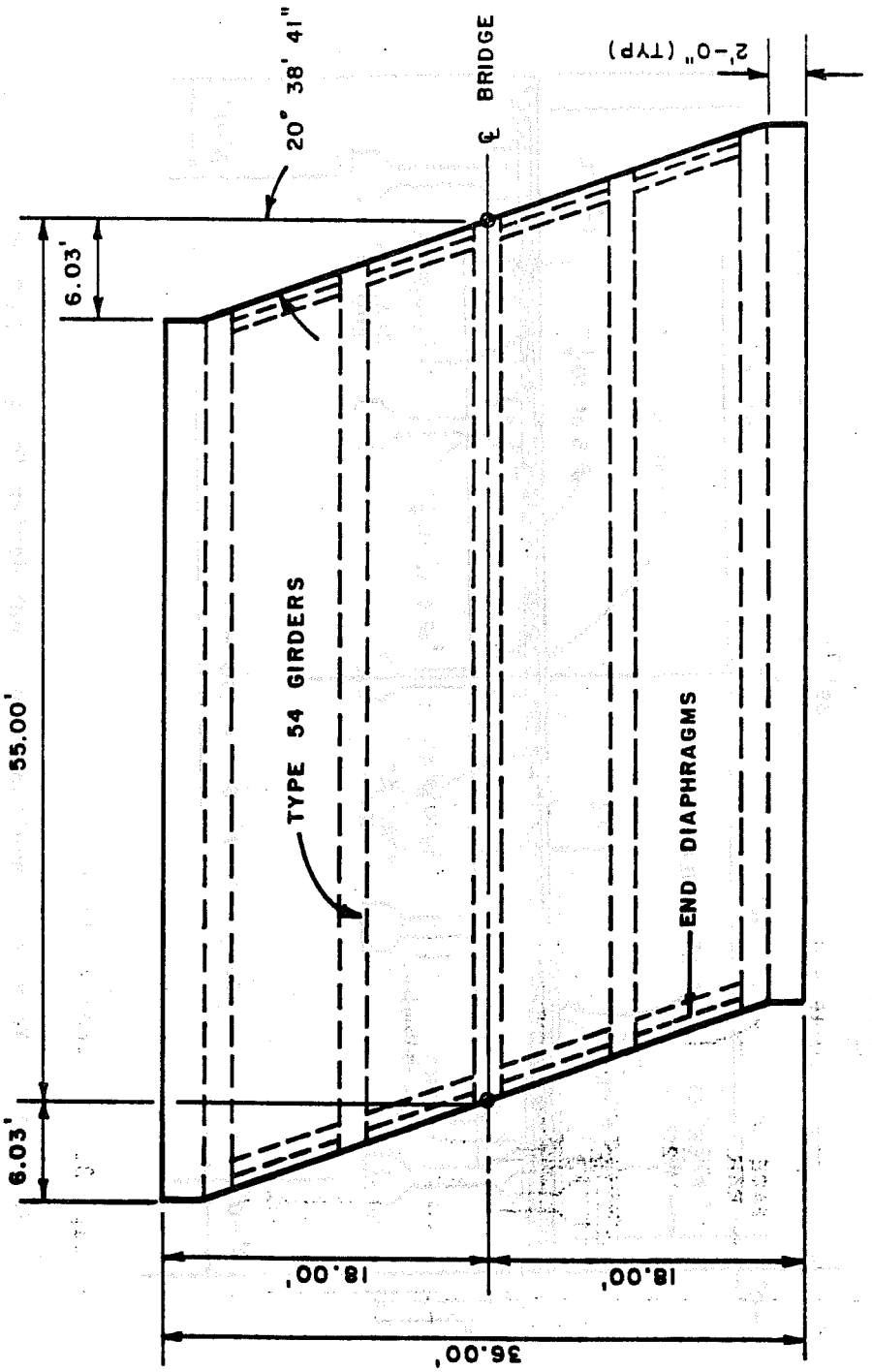


Fig. 3.8 Plan of original design for Example 2 bridge

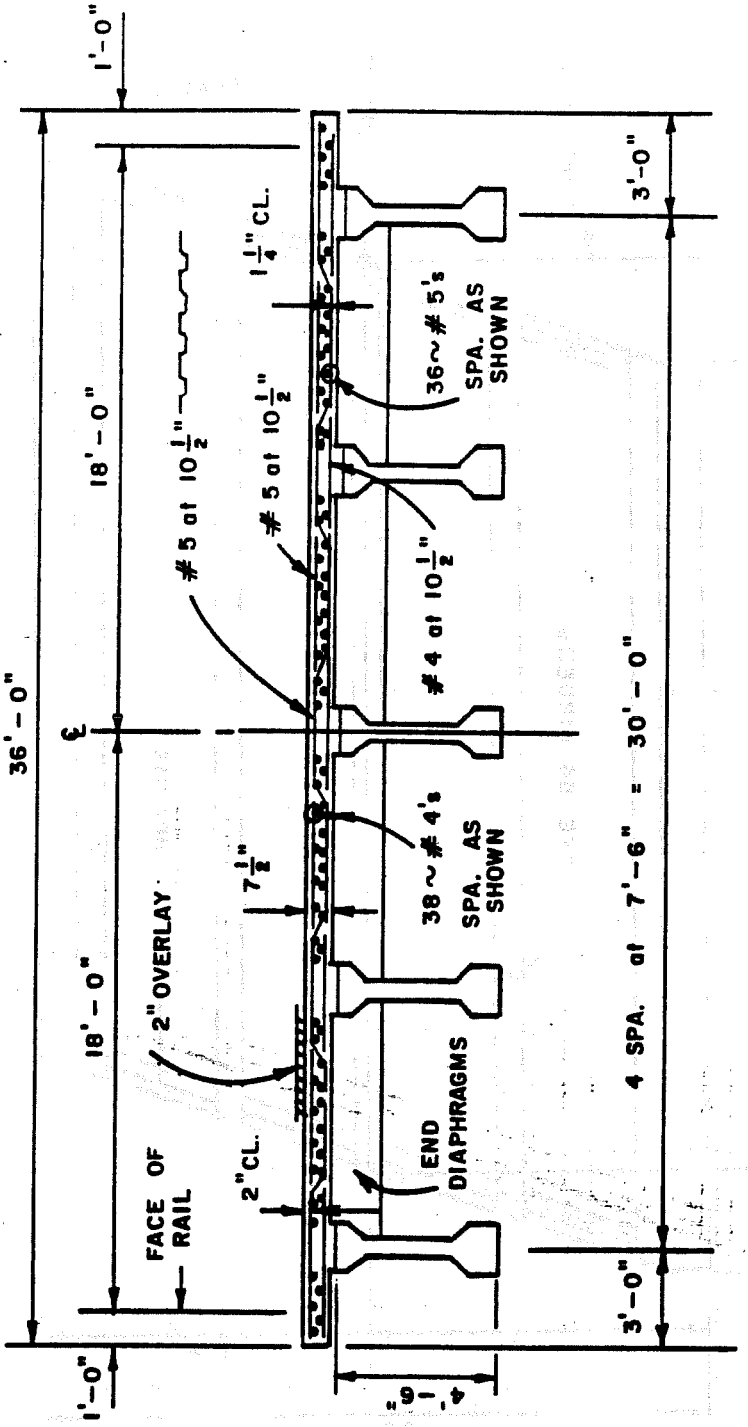
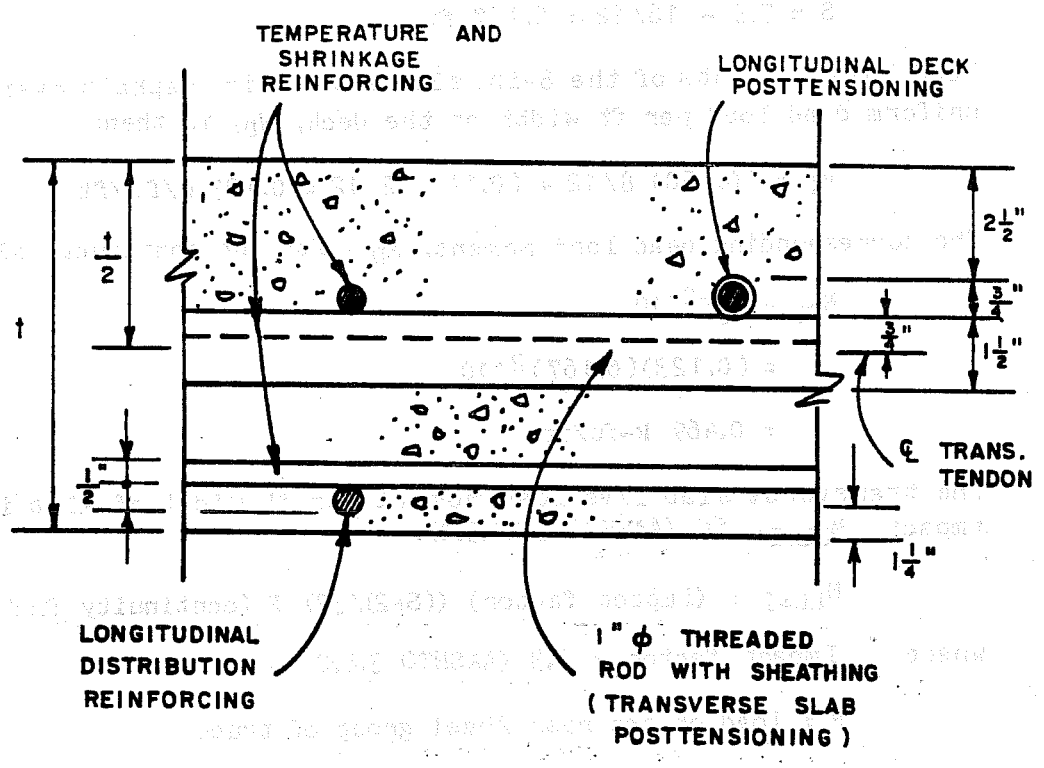


Fig. 3.9 Transverse section of original design for Example 2 bridge



$$\frac{t}{2} = 2\frac{1}{2} + \frac{3}{4} + \frac{3}{4} = 4"$$

$$t = 2(4) = 8"$$

Fig. 3.10 Transverse section of deck showing determination of deck thickness for design Example2

3.3.3 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.

$S = 7.5 - 16/12 = 6.167$ 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:

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

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

$$M_{DL} = W_D S^2/10$$

$$= (0.123)(6.167)^2/10$$

$$= 0.469 \text{ k-ft/ft}$$

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} = (impact factor) ((S+2)/32) P (continuity factor)

where: Impact factor = 1.3 (AASHTO 3.8.2)

P = load on one rear wheel group of truck

= 16 k for HS20 design loading (AASHTO 3.24.3)

Continuity factor = 0.8 (AASHTO 3.24.3.1)

$$M_{LL+I} = (1.3)\ ((6.167+2)/32)\ 16\ (0.8)$$

$$= 4.247 \text{ k-ft/ft}$$

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

$$M_S = M_{DL} + M_{LL+I}$$

$$= 0.469 + 4.247 = 4.716 \text{ k-ft/ft}$$

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

- Compression

$$f_c = (0.4)\, f'_c \qquad \text{(AASHTO 9.15.2.2)}$$

$$= (0.4)\ 4 = 1.60 \text{ ksi}$$

- Tension

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

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

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

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

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

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

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

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

$$F_S = 30.35 \text{ k/ft}$$

Assuming tensile stress controls, check the compression stress:

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

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

$$= 0.758 \text{ ksi} < 1.6 \text{ ksi} = f_c \qquad \text{OK}$$

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:

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

$$= 1.2\ (30.35) = 36.42\ \text{k/ft}$$

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

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

$$W \tan \theta + 4 = 36 \tan (21^o) + 4 = 17.8\ \text{ft}$$

$$(L+W \tan \theta)/N = (55 + 36 \tan (21^o))/2 = 34.4\ \text{ft}$$

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:

$$T_o = 0.8\ (150) = 120\ \text{ksi}$$

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

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

where: K = wobble coefficient = 0.0003 (AASHTO 9.16.1)

L' = length of tendon, ft

$$T_x = (120)\ e^{-(0.0003(36))} = 118.7\ \text{ksi}$$

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

$$(0.7)f'_s = (0.7)\ 150 = 105.0\ \text{ksi} < 118.7\ \text{ksi}$$

Therefore, $T_x = 105.0$ ksi.

Prestressing 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:

$$T_e = T_x - 22 = 105 - 22 = 83.0 \text{ ksi}$$

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:

$$F_T = T_e A_t = (83.0)\ 0.85 = 70.55 \text{ k}$$

where: A_t = area of prestressing steel for one tendon, in.^2

Spacing of the transverse tendons may now be calculated.

- Non-diaphragm regions:

$$\text{Spacing} = F_T/F_S = 70.55/30.35$$

$$= 2.32 \text{ ft} = 27.9 \text{ in.}$$

Use 28 in.

- Diaphragm regions:

$$\text{Spacing} = F_T/F_e = 70.55/36.42$$

$$= 1.94 \text{ ft} = 23.2 \text{ in.}$$

Use 23 in.

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

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

$$\leq 5 \text{ ft} = 60 \text{ in.}$$

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

From Fig. 3.9, it can be seen that the distance from the slab edge to the inside face of the rail, y, is 12 in. From manufacturers' literature, the distance a for a 1-in. diameter threaded rod with a plate anchorage is 8.25 in. Thus:

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

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.

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

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

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

$$M_u = 1.3\,(1.0\,(0.469) + 1.67\,(4.25))$$

$$M_u = 9.83 \text{ k-ft/ft}$$

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 the 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:

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

$$d = 4\ \text{in.}$$

$$b = 12\ \text{in.}$$

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

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

$$M_n = (0.364)\ 128.7\ (4)\ (1 - 0.6\ ((0.00759)128.7)/4)$$

$$= 160.0\ \text{k-in./ft}$$

$$= 13.33\ \text{k-ft/ft}$$

$$M_n \geq M_u/\ \emptyset$$

For posttensioned cast-in-place members, $\emptyset = 0.95$ (AASHTO 9.14).

$13.33 \geq 9.83/0.95 = 10.35$ k-ft/ft OK

3.3.7 Reinforcing Limits. The maximum steel allowed is such that:

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

$(0.00759)\ 128.7/4 = 0.24 < 0.3$ 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).

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

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

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

$$\emptyset\ M_n \geq 1.2\ M_{CR}$$

$$\emptyset\ M_n = (0.95)\ 13.33 = 12.7\ \text{k-ft/ft}$$

$1.2\ M_{CR} = 1.2\ (5.06) = 6.07$ k-ft/ft

$12.7 > 6.07$ OK

3.3.8 Distribution Reinforcement. Longitudinal distribution reinforcing in the bottom of the slab is taken as:

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

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

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

Use #4's spaced at 10 in. ($A_L = 0.24$ in.^{2}/ft).

3.3.9 Shrinkage and Temperature Reinforcing.

3.3.9.1 Nonprestressed Reinforcing. 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's spaced at 18 in. (0.133 in.^{2}/ft) in all these locations.

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

$A = 1213 \text{ in.}^2$

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

$y_t = 17.19$ 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 the following calculations:

From Fig. 3.10, 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:

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

$$f = P_L/A + P_L ec/I$$

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

$$0.10 = P_L/1213 + (P_L(14.31)(17.19-4)/476,500)$$

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

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

As in Sec. 3.3.4, the effective force per strand must be found to determine the tendon spacing. The maximum tendon stress at the jacking end during stressing is:

$$T_o = (0.8)f'_s = (0.8)\ 270 = 216 \text{ ksi}$$

and from AASHTO Sec. 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

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

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:

$$T_e = T_x - 32 = 189 - 32 = 157 \text{ ksi}$$

The effective prestress force per tendon for a 1/2-in. seven-wire strand with an area of 0.153 sq.in. is then:

$$F_T = T_e A_t = 157\ (0.153) = 24.0\ k$$

Spacing of the longitudinal tendons may now be calculated.

$$\text{Spacing} = F_T/P_L = 24.0/10.92 = 2.20\ ft = 26.4\ in.$$

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 be accommodated by slightly lowering the pretensioned strand eccentricity, increasing the concrete strength, or adding two more pretensioned strands. For this example the precast girders would also have to be designed for the 8-in. thick deck instead of the 7-1/2 in. deck in the original design.

3.3.10 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. As discussed in Sec. 2.2.1.7, 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:

$$\text{fan tendon spacing} = (\text{diaphragm region spacing}) \cos \theta$$

$$= 23 \cos (21^\circ) = 21.5\ in.$$

Use 21 in. as shown in Fig. 3.11.

By P-1.10, nonprestressed reinforcing 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

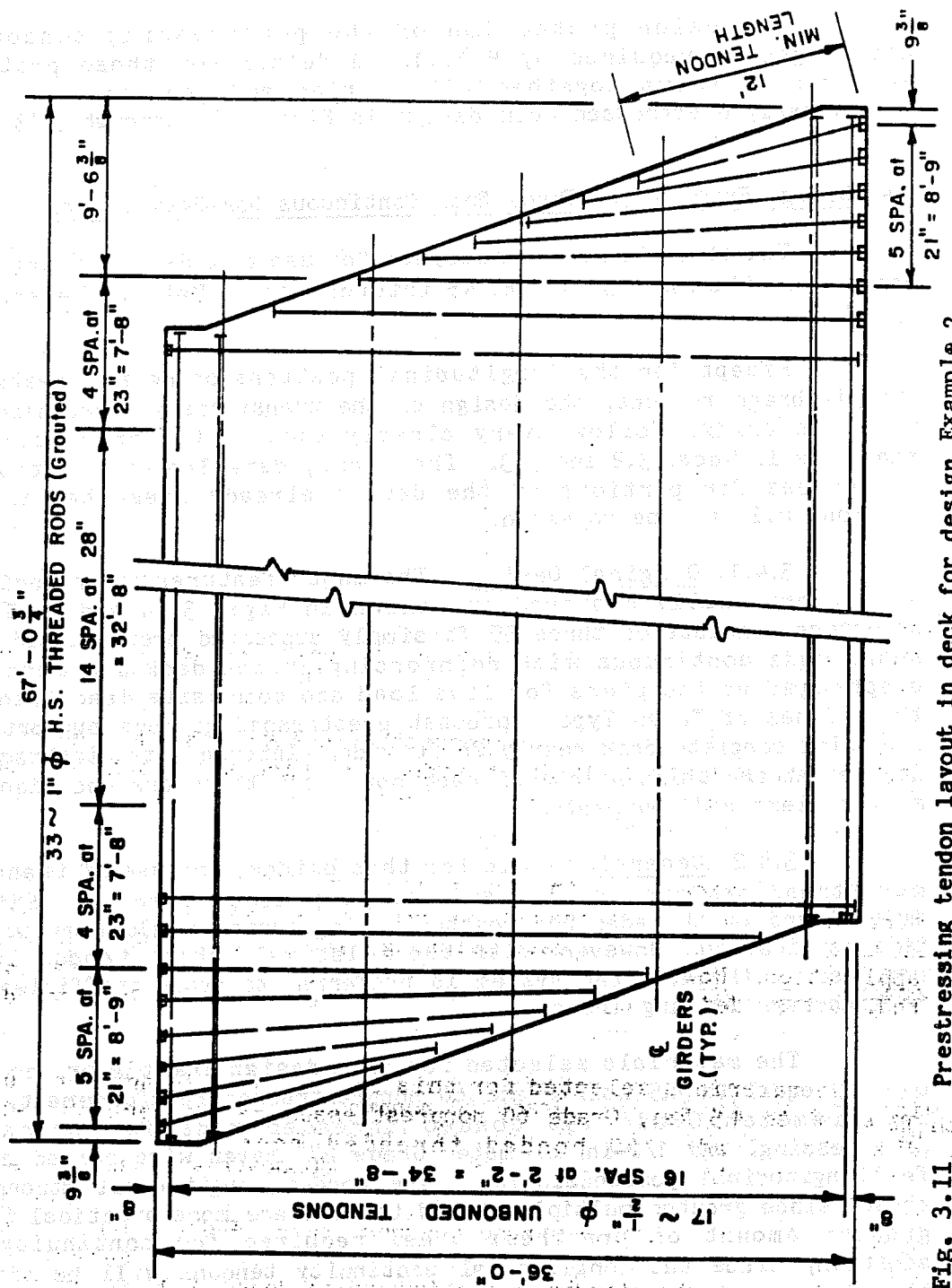


Fig. 3.11 Prestressing tendon layout in deck for design Example 2

reinforcing 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. 3.11 through 3.13.

3.4 Design Example 3: Three Span Continuous Non-Skew Bridge

The third bridge redesigned for use as a design example is a portion of the I.H. 635 - I.H. 45 interchange in Dallas, Texas, built in 1969.

Except for the longitudinal posttensioning and treatment of the diaphragm regions, the design of the transversely prestressed deck for this bridge follows very closely that of the previous design examples in Secs. 3.2 and 3.3. Therefore, detailed explanations and references for portions of the design already presented in those sections will not be repeated.

3.4.1 Original Design. The major features of the bridge as it was originally designed are shown in Figs. 3.14 and 3.15. The structure consists of three 55 ft simply supported pretensioned girder spans made continuous with reinforcing in the deck and monolithic diaphragms at the piers for live load and composite dead load. The three lines of Texas Type C precast prestressed girders support an 8-in. thick concrete deck nearly 26 ft wide. Intermediate diaphragms are located at the third points of each span, and there are end diaphragms at the piers and abutments.

3.4.2 General. Again for this bridge, the use of transverse monostrand tendons would result in very close spacing. Either a multistrand or threaded bar posttensioning system would work well for this application. However, with the relatively short tendon lengths required, a threaded bar system is preferred to avoid inefficiency due to anchorage seating losses.

The materials selected for this design example are concrete with f'_c equal to 4.5 ksi, Grade 60 nonprestressed reinforcement, 1-1/4 in. diameter Grade 150 bonded threaded rods for transverse prestressing, and 1/2-in. diameter Grade 270 seven wire bonded strands for longitudinal posttensioning. The bonded longitudinal strands are chosen since grouted multiple strand tendons are more practical for the greater amount of prestress steel required for continuity. In addition, since the longitudinal continuity tendons will be vital to the structural integrity of the bridge, the greater safety against anchorage failure and superior performance under overload of bonded prestressing are desirable.

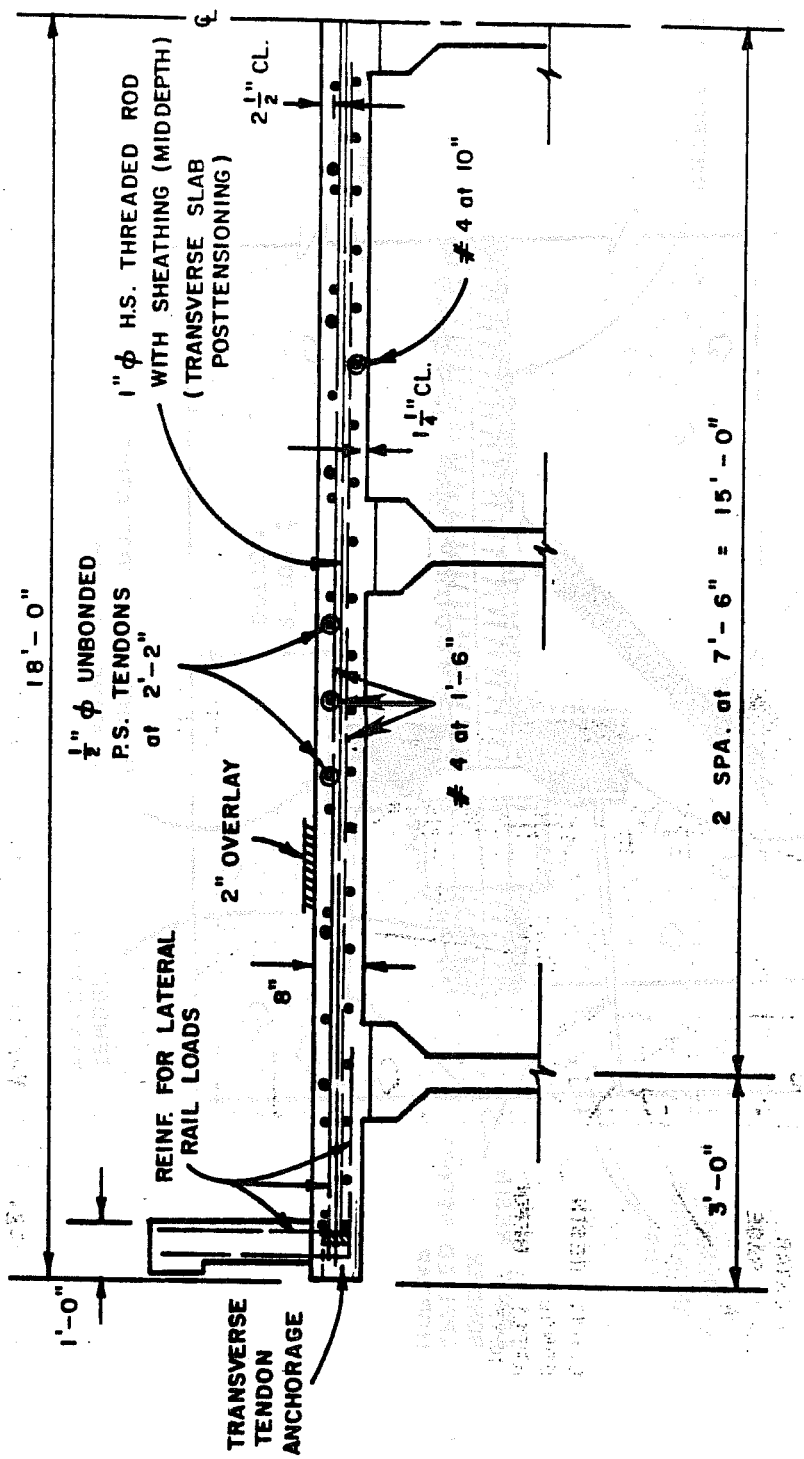


Fig. 3.12 Transverse section of deck for design Example 2

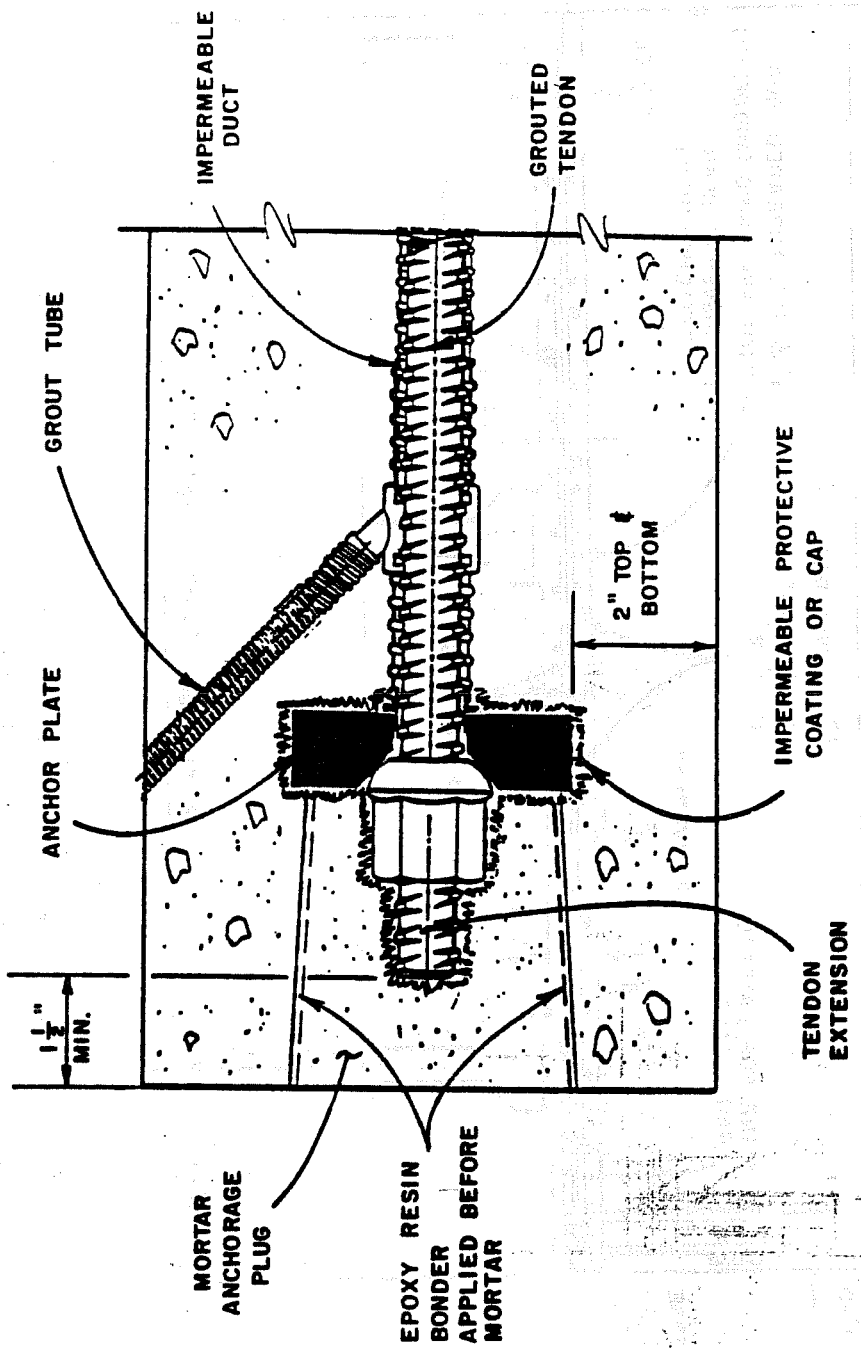


Fig. 3.13 Vertical section through slab showing special detailing requirements at tendon anchorages

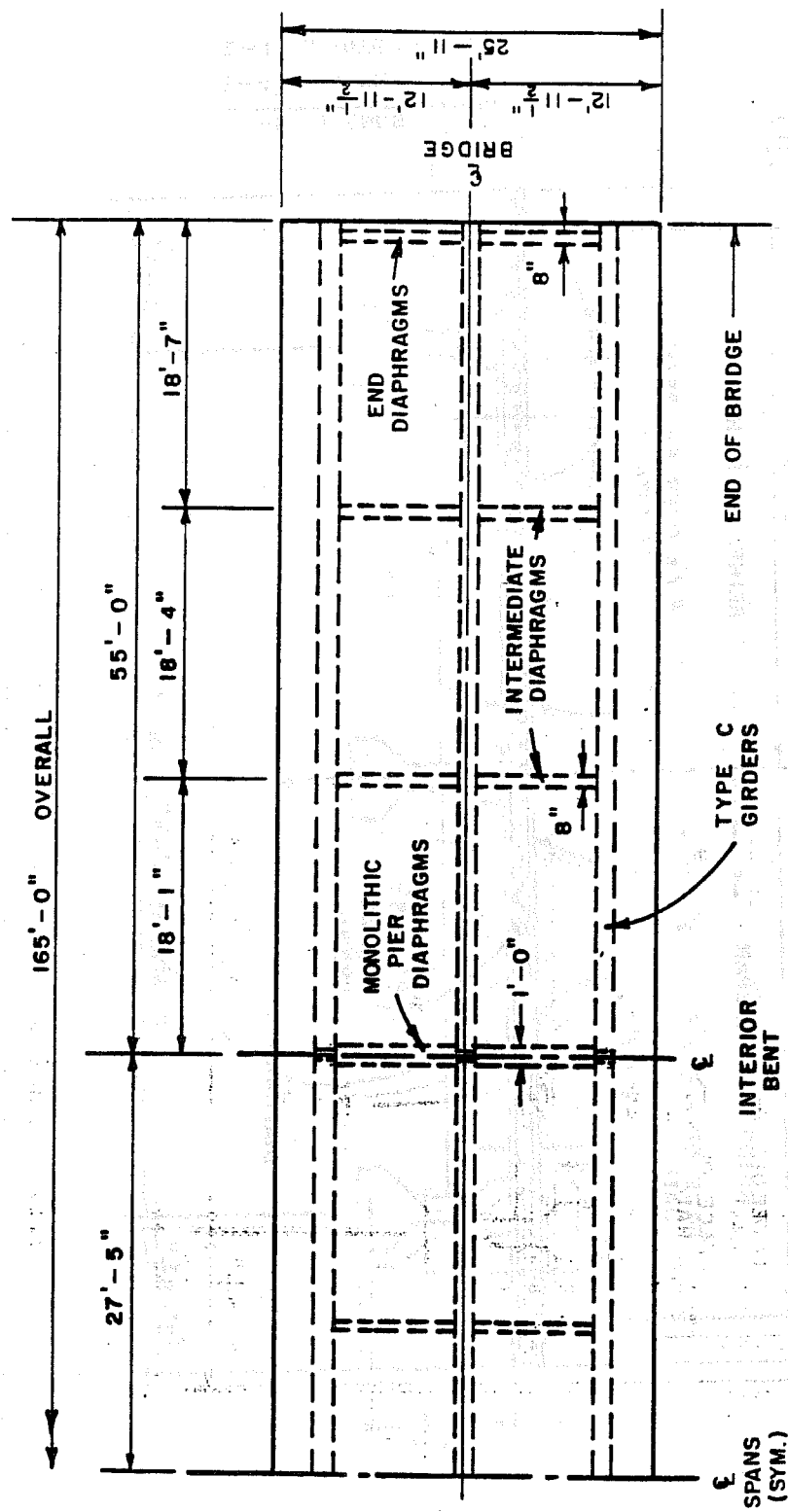


Fig. 3.14 Plan of original design for Example 3 bridge

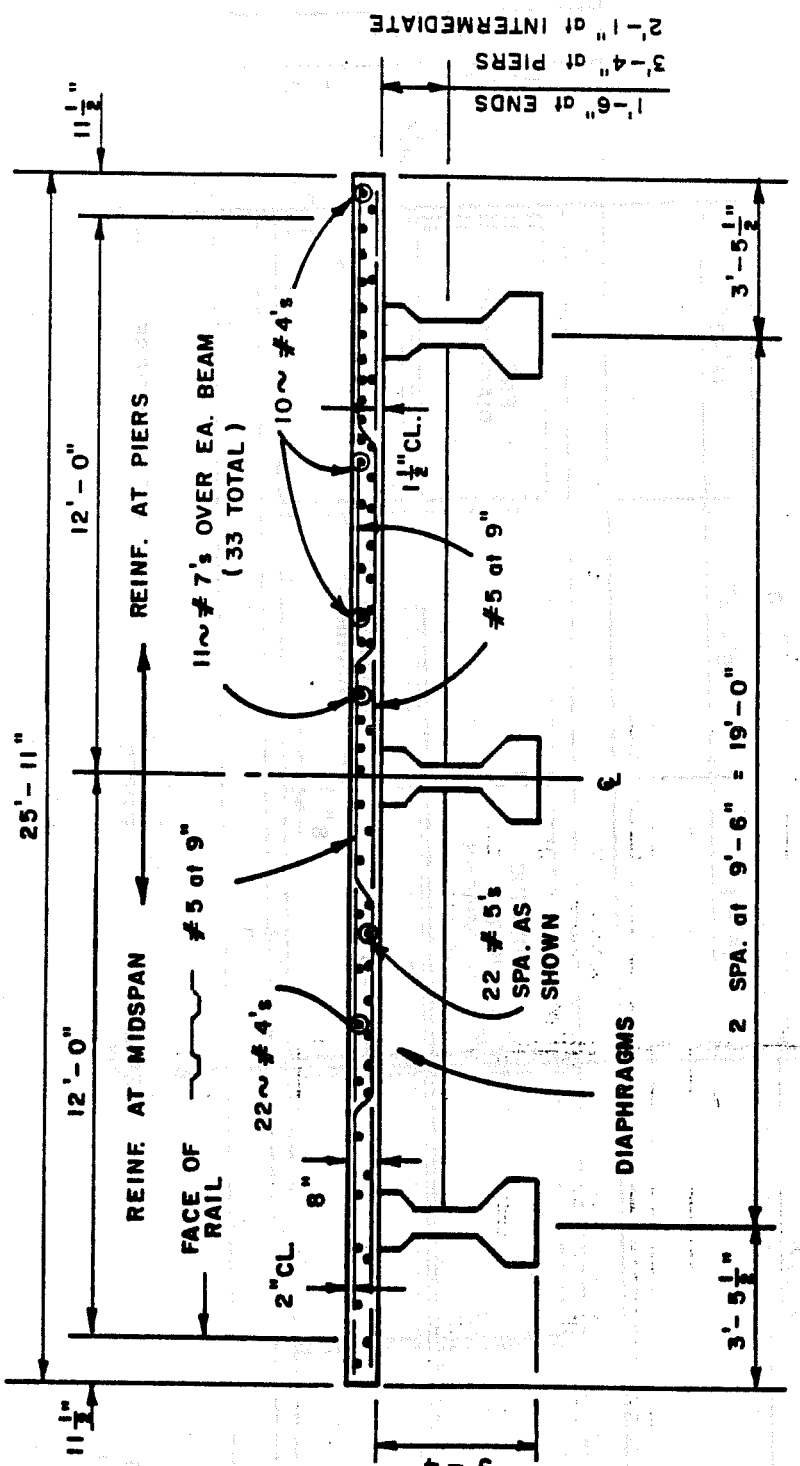


Fig. 3.15 Transverse section of original design for Example3 bridge

Placement of reinforcing in the slab for this example is shown in Fig. 3.16 and is similar to that for Example 2. This bridge, however, will have the longitudinal prestressing tendons located in the precast girders instead of in the slab. Also, the outside diameter of the transverse tendon sheathing is 1-3/4 in. to accommodate the 1-1/4 in. diameter transverse threaded rods. The minimum slab thickness can then be calculated as 7-3/4 in. An 8-in. thick deck will be used.

Instead of decreasing the transverse tendon spacing in the diaphragm regions to account for the diaphragm restraining effects, this bridge will feature transversely posttensioned diaphragms.

3.4.3 Slab Loads. The top flange of a Texas Type C girder is 14 in. wide. The effective transverse slab span is then:

$$S = (\text{girder spacing}) - (\text{flange width})$$

$$= 9.5 - 14/12 = 8.33 \text{ ft}$$

Dead load for the slab is its self weight only since this bridge has no provision for an asphalt overlay:

$$W_D = (8/12)\ (0.15) = 0.10 \text{ k/ft/ft}$$

$$M_{DL} = W_D\ S^2/10$$

$$= (0.10)(8.33)^2/10$$

$$= 0.694 \text{ k-ft/ft}$$

$$M_{LL+I} = (\text{impact factor})\ ((S+2)/32)\ P\ (\text{continuity factor})$$

$$= 1.3\ ((8.33+2)/32)\ 16\ (0.8)$$

$$= 5.37 \text{ k-ft/ft}$$

Total transverse service load moment acting on the deck, M_S, is then:

$$M_S = M_{DL} + M_{LL+I}$$

$$= 0.694 + 5.37 = 6.07 \text{ k-ft/ft}$$

3.4.4 Transverse Prestress Design. Allowable extreme fiber stresses in the concrete are calculated as:

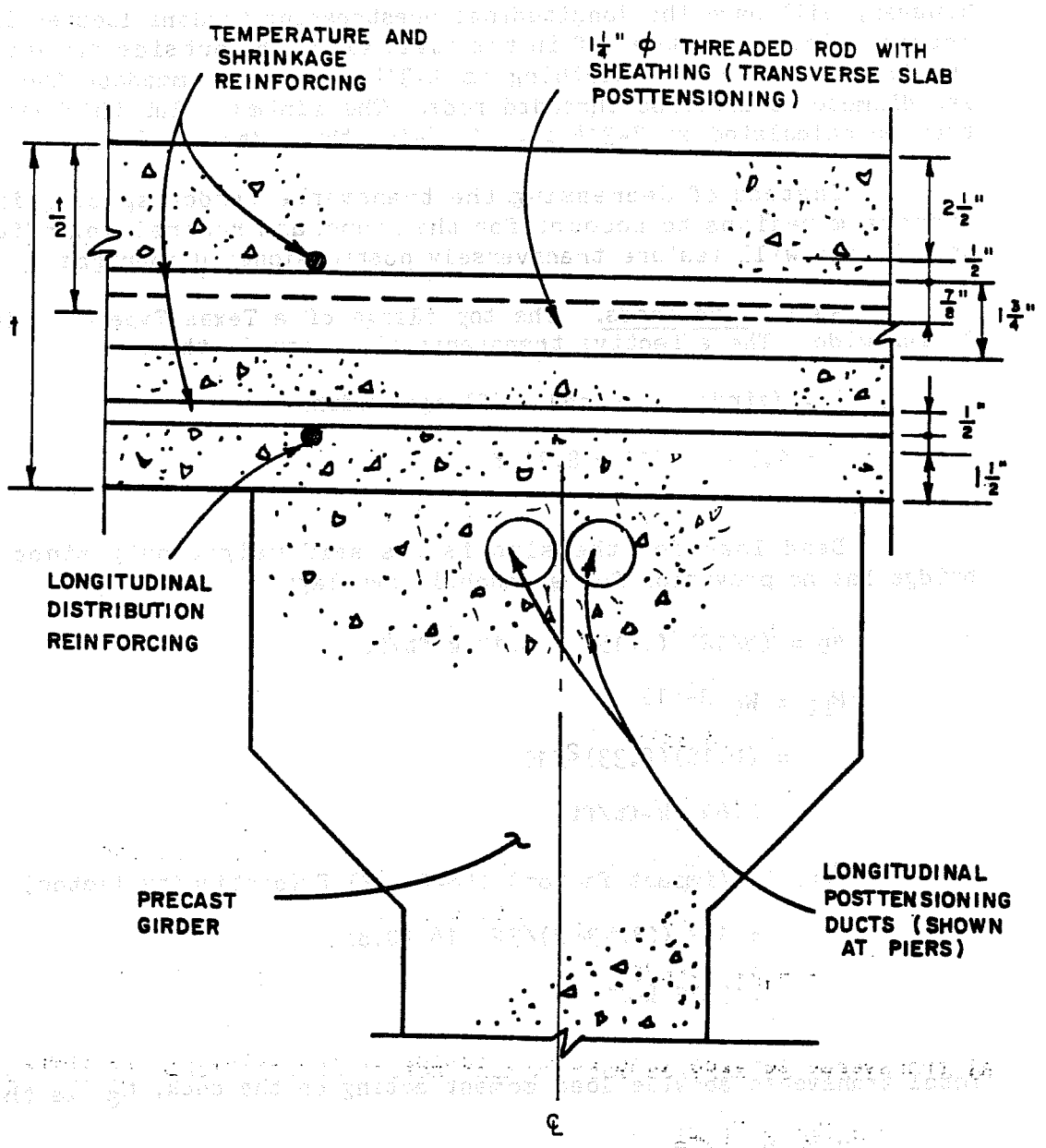


Fig. 3.16 Transverse section of deck showing determination of deck thickness for design Example 3

- Compression:

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

$$= (0.4)4.5 = 1.80 \text{ ksi}$$

- Tension:

$$f_t = 2\sqrt{f'_c} = 2\sqrt{4500/1000}$$

$$= 0.134 \text{ ksi}$$

As for Example 2, the section properties for a 1-ft wide strip of 8-in. thick slab are:

$$A = 92 \text{ in.}^2/\text{ft}$$

$$S' = 128 \text{ in.}^3/\text{ft}$$

The required effective transverse prestress slab force per unit edge length, F_S, is then found by:

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

$$-0.134 = F_S/92 - ((6.07)12)/128$$

$$F_S = 40.0 \text{ k/ft}$$

Maximum compression stress must be checked.

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

$$= 40.0/92 + ((6.07)12)/128$$

$$= 1.00 \text{ ksi} < 1.8 \text{ ksi} = f_c \qquad \text{OK}$$

To find the tendon spacing associated with the required transverse prestress force, F_S, calculated above, the effective force per tendon after all losses, F_T, must be found. In the second design example, it was shown that the maximum allowable tendon stress after seating controlled the effective tendon stress rather than friction losses for these short, straight tendons. The final effective tendon stress calculated for the second example will also apply to this bridge:

$$T_e = 83.0 \text{ ksi}$$

Cross section area of a 1-1/4 in. diameter threaded rod is 1.25 sq.in. Thus:

$$F_T = T_e A_t = (83.0)\ 1.25 = 103.8 \text{ k/tendon}$$

Required tendon spacing is found as:

$$\text{Spacing} = F_T/F_S = 103.8/40.0 = 2.59 \text{ ft} = 31.1 \text{ in.}$$

Use 31 in.

Check the maximum tendon spacing requirements:

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

$$\leq 5 \text{ ft} = 60 \text{ in.}$$

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

From Fig. 3.15, it can be seen that the distance y is 11-1/2 in. for this bridge. From manufacturers' literature, the distance a is 9-1/2 in. for a 1-1/4 in. diameter bar with flat plate anchorage. Thus:

$$\text{Spacing} \leq 3\ (11.5 - 9.5 + 12) = 42 \text{ in.}$$

31 in. < 42 in. OK

Concrete stresses were also checked for conditions at the time of stressing as in the other examples and were found to be acceptable. These calculations are omitted here for brevity.

The restraining effects of the diaphragms will be compensated for in this bridge by prestressing the diaphragms. The level of prestressing needed in each diaphragm is determined by Sec. P 1.3.2.3, which gives the diaphragm prestress force, P_D, as:

$$P_D = C_t C_K C_L\ 1.6\ F_S$$

where: $C_t = 8/t$

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

$$C_L = 25/S_D$$

No more than two values for C_t, C_K, and C_L may be taken less than one. For an 8-in. thick deck:

$$C_t = 8/8 = 1.0$$

The spacing of the diaphragms, S_D, is shown in Fig. 3.14. Since the spacings are nearly equal, an average value will be used.

$$\text{Let } S_D = 18.3 \text{ ft}$$

And therefore:

$$C_L = 25/18.3 = 1.36$$

Since the end, intermediate, and pier diaphragms are all different sizes, each type of diaphragm will have a different value of C_K, and therefore a different required prestress force. Assuming the same concrete is used for all the diaphragms and the deck, the elastic modulus of the concrete, E, is:

$$E = 57\sqrt{f'_c} \text{ ksi} \qquad \text{(AASHTO 8.7.1)}$$

$$= 57\sqrt{4500} = 3820 \text{ ksi}$$

- End Diaphragms

Cross section area, A = (height) x (width)

$$= (18)\ 8 = 144 \text{ in.}^2$$

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

$$= 3820\ (144)/640{,}000$$

$$= 0.86$$

$$P_D = C_t C_K C_L\ 1.6\ F_S$$

$$= 1.0(0.86)1.36(1.6)40.0 = 74.9 \text{ k}$$

The effective tendon stress after all losses calculated previously for the transverse tendons is valid also for the diaphragm posttensioning, as long as the same prestressing system is used. For T_e = 83.0 ksi, the effective tendon forces shown in Table 3.2 are obtained.

TABLE 3.2 Effective Tendon Forces for Diaphragm Posttensioning

Threadbar Diameter (in.)	A_t (in.^{2})	F_T (k)
5/8	0.28	23.2
1	0.85	70.6
1-1/4	1.25	103.8
1-3/8	1.58	131.1

For the end diaphragms, use a 1-in. diameter rod for posttensioning.

- Intermediate Diaphragms

$$A = 25\ (8) = 200\ \text{in.}^2$$

$$C_K = 3820(200)/640{,}000 = 1.19$$

$$P_D = 1.0(1.19)1.36(1.6)40.0 = 103.6\ \text{k}$$

Use a 1-1/4 in. diameter rod for posttensioning in the intermediate diaphragms.

- Pier Diaphragms

$$A = 40(12) = 480\ \text{in.}^2$$

$$C_K = 3820(480)/640{,}000 = 2.87$$

$$P_D = 1.0(2.87)1.36(1.6)40.0 = 249.8\ \text{k}$$

Use two 1-3/8 in. diameter rods for posttensioning the pier diaphragms.

3.4.5 Supplementary Bonded Reinforcement. As for the second design example, supplementary bonded reinforcing is not required since the transverse prestressing tendons will be grouted.

3.4.6 Ultimate Moment Check

M_u = (load factor) x ((dead load coefficient) x M_{DL}

+ (live load ccefficient) x M_{LL+I})

= 1.3(1.0(0.694) + 1.67(5.37))

= 12.56 k-ft/ft

$A_s^* = 12/31\ (1.25) = 0.484\ in.^2/ft$

b = 12 in. and d = 4 in.

$p^* = A_s^*/bd = 0.484/(12(4)) = 0.01008$

$f_{su}^* = f_s' (1 - 0.5\ p^* f_s'/f_c')$

$= 150\ (1 - 0.5\ ((0.01008)(150))/4.5) = 124.8\ ksi$

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

$= (0.484)124.8(4)(1 - 0.6\ ((0.01008)124.8)/4.5)$

$= 201.0\ k\text{-}in./ft = 16.75\ k\text{-}ft/ft$

$M_n \geq M_u/\phi$

$16.75 \geq 12.56/0.95 = 13.22\ k\text{-}ft/ft$ OK

3.4.7 Reinforcing Limits

- Maximum Prestressing Steel

$$p^* f_{su}^*/f'_c \leq 0.30$$

$$(0.01008)\ 124.8/4.5 = 0.28 < 0.3 \qquad \text{OK}$$

- Minimum Reinforcing

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

$$(7.5\sqrt{4500})/1000 = M_{CR}/128$$

$$M_{CR} = 64.40 \text{ k-in./ft} = 5.37 \text{ k-ft/ft}$$

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

$$\phi M_n = (0.95)\ 16.75 = 15.91 \text{ k-ft/ft}$$

$$1.2 M_{CR} = 1.2(5.37) = 6.44 \text{ k-ft/ft}$$

$$15.91 > 6.44 \qquad \text{OK}$$

3.4.8 Distribution Reinforcement. The longitudinal distribution reinforcing in the bottom of the deck is the same for this bridge as for the second design example, since both have 8-in. thick decks. Use #4 bars spaced at 10 in.

3.4.9 Shrinkage and Temperature Reinforcing. Bonded non-prestressed reinforcing is needed in both directions in the top of the slab, and in the transverse direction in the bottom of the slab, to control temperature and shrinkage stresses. Use #4 bars spaced at 18 in. in each of these three locations. In addition to the longitudinal mild steel reinforcing in the top of the deck, an average longitudinal compressive stress of 100 psi will be induced in the slab by the longitudinal continuity posttensioning, to prevent transverse cracking due to temperature and shrinkage effects.

3.4.10 Longitudinal Continuity Posttensioning. The original design utilized conventional reinforcing in the slab over the piers to

provide negative moment capacity for longitudinal continuity. As a means of preventing the transverse cracking which would occur but be controlled by the original continuity reinforcing, this design example will use longitudinal posttensioning tendons located in the girders to provide the continuity connection between spans. These tendons will also be used to provide the precompression in the slab discussed in Sec. 3.4.9 for temperature and shrinkage effects.

Because the longitudinal posttensioning will provide a significant portion of the superstructure moment resistance, the design of the precast girders will be dependent on the longitudinal prestressing used. The steps for designing the longitudinal continuity posttensioning are then as follows:

1. Determine loads acting on the structure when it is still a simple span noncomposite bridge, and when it is a continuous composite bridge.

2. Find the noncomposite and composite girder section properties.

3. Select a trial posttensioning tendon profile using the greatest possible eccentricity over the piers at the continuity connections.

4. Determine the required posttensioning force to satisfy stress requirements at the continuity connections.

5. Check concrete stresses along the other portions of the span using a prestressing force determined from the required effective tendon force at the piers found in step 4.

6. If the stresses exceed the allowable values, adjust the drape of the tendon profile and return to step 4. Note that the most efficient profile is one that has the greatest drape while still providing a 100 psi compressive stress in the slab near midspan under negative live load.

7. Once the tendon profile and prestressing force have been determined, find the amount of composite moment capacity contributed by the longitudinal posttensioning.

8. Design the precast girders to carry the full noncomposite moment calculated in step 1 and the difference between the total composite moment and that calculated in step 7.

9. Verify the ultimate capacity of the design.

As mentioned previously, the stresses will only be checked for this example at the pier and midspan locations for brevity. Also, the design for shear has been omitted since it is no different than shear

design for any posttensioned beam. In a complete design, however, concrete stresses should be checked for shear and moment at every tenth point along the span.

Step 1--Loads

A Texas Type C girder weighs 516 lb/ft. The uniform dead load of the cast-in-place slab on one girder, w_S, is:

$$w_S = (8/12)9.5(0.15) = 0.950 \text{ k/ft}$$

Dead load of the railings and diaphragms will be ignored. The portion of live load assigned to each girder is found by:

distribution factor = (girder spacing)/5.5 (AASHTO 3.22.2.2)

= 9.5/5.5 = 1.727 wheel lines/girder

Live load impact fraction, I_L, is determined as:

$$I_L = 50/(L+125) \qquad \text{(AASHTO 3.8.2.1)}$$

$$= 50/(55+125) = 0.278$$

Simple span noncomposite dead loads consist of the weight of the girder and slab. The only live load moments acting on the structure at this time are from construction loads. Construction live loads are ignored for this example. Total noncomposite service load moment, M_{NC}, is then:

$$M_{NC} = w L^2/8 = (0.516 + 0.950)(55)^2/8$$

$$= 554.3 \text{ k-ft}$$

Since there is no asphalt overlay, and the weight of the rails has been ignored, the only load acting on the composite section is the live load.

An elastic analysis of a three-span continuous girder with 55 ft span lengths, loaded with critical configurations of one lane of AASHTO HS20 live load, gives the moments shown in the lane moment column of Table 3.3. These moments are either near the midpoint of the respective span, or over the piers. The girder moment column in Table 3.3 shows that portion of these moments plus impact carried by one girder as found by:

$$\text{girder LL+I moment} = (\text{lane moment})/2 \times (1 + \text{impact fraction}) \times (\text{distribution factor})$$

$$= (\text{lane moment})\,((1.278)1.727)/2$$

$$= (\text{lane moment})\,(1.104)$$

TABLE 3.3 Longitudinal Girder Live Load Plus Impact Moments for Design Example 3

Location	Lane Moment (k-ft)	Girder LL+I Moment (k-ft)
End span positive	564	622
End span negative	-137	-151
Over piers	-407	-449
Center span positive	458	505
Center span negative	-171	-189

Since there is no dead load acting on the composite section on this bridge, the girder live loads in Table 3.3 are the total composite moments.

Step 2--Section Properties

The area, moment of inertia, and distances from the neutral axis to the top and bottom extreme fibers are given in Table 3.4 for both the Texas Type C precast girder alone, and for the composite

girder section. Properties for the composite section were calculated neglecting girder haunches and differences in the modulus of elasticity between the precast girder and the slab. Also, a 9 ft-6 in. wide top flange was assumed. Note that this is the center-to-center spacing of the girders instead of the effective flange width as found by AASHTO Sec. 8.10.1. This is because use of a smaller flange width would result in a smaller value of A and I for the composite girder section. These reduced section properties would in turn result in a smaller post-tension force than actually required to induce compression across the entire slab. The effective flange width as calculated according to AASHTO provisions should be used, however, when checking ultimate strength in step 9.

TABLE 3.4 Girder Section Properties for Design Example 3

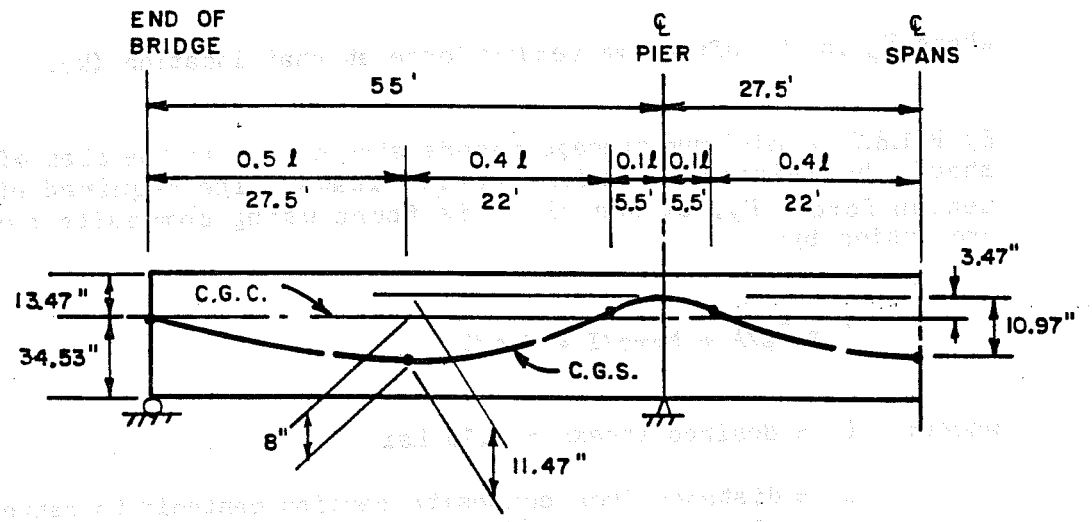
	Precast Girder	Composite Girder
A ($in.^2$)	495	1,407
I ($in.^4$)	82,600	319,800
y_t (in.)	22.91	13.47
y_b (in.)	17.09	34.53

Step 3--Trial Posttensioning Tendon Profile

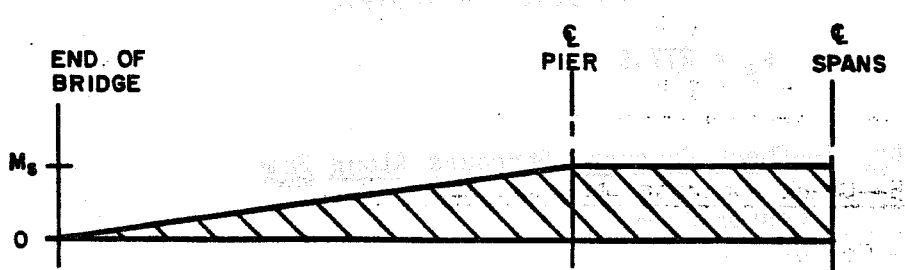
Initially, it is assumed that the required prestressing steel will be placed in two 2-in. diameter ducts as shown in Fig. 3.16. Figure 3.17(a) shows the profile assumed for the longitudinal posttensioning tendon. The 3.47-in. eccentricity at the pier allows a minimal amount of concrete cover over the top of the tendon ducts to be maintained.

Step 4--Required Posttensioning at the Pier

Several methods are available for determining the moments in a continuous beam due to posttensioning. A simple method is that of using equivalent vertical loads imposed on the beam by the tendon. Reference 27 includes a design aid which gives tables of coefficients, based on the equivalent vertical load method, for determining moments over the supports due to posttensioning in continuous beams. Using these tables and the tendon profile of Fig. 3.17(a), the total moment



(a) longitudinal posttension tendon profile for design Example 3



(b) secondary moments due to post-tensioning

Fig. 3.17 Tendon profile and secondary moment diagram for longitudinal posttensioning, design Example 3

due to posttensioning, M_{PT}, at the pier is found to be:

$$M_{PT} = (0.628)\, F_e \quad \text{k-ft}$$

where F_e is the effective tendon force at that location (k).

By P-1.8.2, a minimum average compressive stress in the slab of 100 psi should be maintained under service loads. The required effective tendon force, F_e, at the piers is found using composite section properties by:

$$f = F_e/A + M_{PT}c/I + M_c c/I$$

where: f = desired stress = 0.10 ksi

c = distance from composite section centroid to center of slab

= 13.47 - 4.0 = 9.47 in.

M_c = composite service load moment

= -449 k-ft (from Table 3.3)

$$0.10 = F_e/1407 + ((0.628)12(9.47)F_e)/319{,}800 - (449(12)9.47)/319{,}800$$

$$F_e = 277.9 \text{ k}$$

Step 5--Check Concrete Stresses Along Span

- Center Span

If the longitudinal tendons are stressed from both ends of the bridge, the greatest friction losses will be at midpoint of the center span. Friction losses between two points may be computed by (AASHTO 9.16.1):

$$F_x = F_o\, e^{-(KL' + \mu\alpha)}$$

where: F_o = tendon force at initial point

F_x = tendon force at point under consideration

K = friction wobble coefficient

= 0.0002 [27]

μ = friction curvature coefficient

= 0.25 [27]

L' = length along tendon between points under consideration (ft)

α = total angular change of prestressing steel profile between points under consideration (rad)

This formula will then be used to find the effective tendon force at the center of the middle span when the effective tendon force at the pier is 277.9 k. Since tendon eccentricities are small, use the horizontal distance between the points of 27.5 ft for L'. Total angular change of the tendon between the points is found to be 0.133 rads. Thus, the tendon force at the center of the girder spans is:

$$F_x = (277.9)\, e^{-(0.0002(27.5)+0.25(0.133))}$$

$$= 267.3 \text{ k}$$

The total moment at any section due to posttensioning is the sum of the tendon force times its eccentricity and the secondary moment. A diagram of the secondary moment due to posttensioning for this structure is shown in Fig. 3.17(b). Since the total prestress moment is known at the pier location, the secondary moment, M_S, at the pier may be calculated as:

$$M_S = M_{PT} - F_e(e)$$

$$= (0.628)\, F_e - F_e\, (3.47/12)$$

$$= (0.339)\, F_e \text{ k-ft}$$

Total prestress moment at the center of the middle span is then:

$$M_{PT} = F_e\ (e) + M_S$$

$$= (267.3)\ (-7.5/12) + (0.339)(267.3)$$

$$= -76.4\ \text{k-ft}$$

Stress at the center of the slab under service load is found using:

$$f = F_e/A + M_{PT}c/I + M_c c/I$$

$$= 267.3/1407 - (76.4(12)9.47)/319,800$$

$$- (189(12)9.47)/319,800$$

$$= 0.096\ \text{ksi}$$

While this stress is 4% below the 100 psi desired, it is considered adequate.

- End Span

The tendon force at the center of the end span differs from that at the interior pier due to friction and anchor set. As mentioned earlier, it is assumed the tendons will be jacked from both ends of the structure. Thus, since the end span is closer to the jacking point, there will be less friction losses. Losses due to anchorage set of the tendon are likely to be significant in the end span, but much less, if any, toward the piers because of reverse tendon friction. Both friction and anchor set losses must be accounted for.

The tendon force at the center of the endspan considering only friction losses is first calculated. The length along the tendon between the center of the end span and the pier is 27.5 ft and the total angular change of the tendon between these points is 0.139 rads. The effective tendon force, excluding anchorage losses, at the center of the endspan corresponding to a tendon force of 277.9 k at the pier is:

$$F_x = F_o\ e^{-(KL' + \mu\alpha)}$$

$$277.9 = F_o\ e^{-(0.0002(27.5)+0.25(0.139))}$$

$$F_o = 289.3\ \text{k}$$

Anchor set losses affect the stress in the prestress tendon as shown in Fig. 3.18. The distance x' over which anchor set influences tendon force may be calculated as [27]:

$$x' = \sqrt{E(\Delta L)L/12d} \quad \text{(ft)}$$

where: E = modulus of elasticity of prestress steel (ksi)

= 27,000 ksi [12]

L = length to a point where loss is known (ft)

d = friction loss in length L (ksi)

The change in tendon stress due to anchor set at the jacking location, Δf, is found by [27]:

$$\Delta f = 2dx'/L$$

To find x', the friction loss must be known over a given length. For this example, the total friction loss over the end span will be used. Assume an effective tendon stress, T_x, at the pier of 145 ksi. Then the effective tendon stress at the end of the structure, T_o, is:

$$T_x = T_o\, e^{-(KL' + \mu\alpha)}$$

with: T_x = 145 ksi

L' = 55.0 ft

α = 0.188 rad

$$145 = T_o\, e^{-(0.0002(55)+0.25(0.188))}$$

$$T_o = 153.6 \text{ ksi}$$

Then:

$$d = T_o - T_x = 153.6 - 145 = 8.6 \text{ ksi}$$

Assuming an anchor set of 1/4 in.:

$$x' = \sqrt{(27{,}000(0.25)(55))/(12(8.6))} = 60.0 \text{ ft}$$

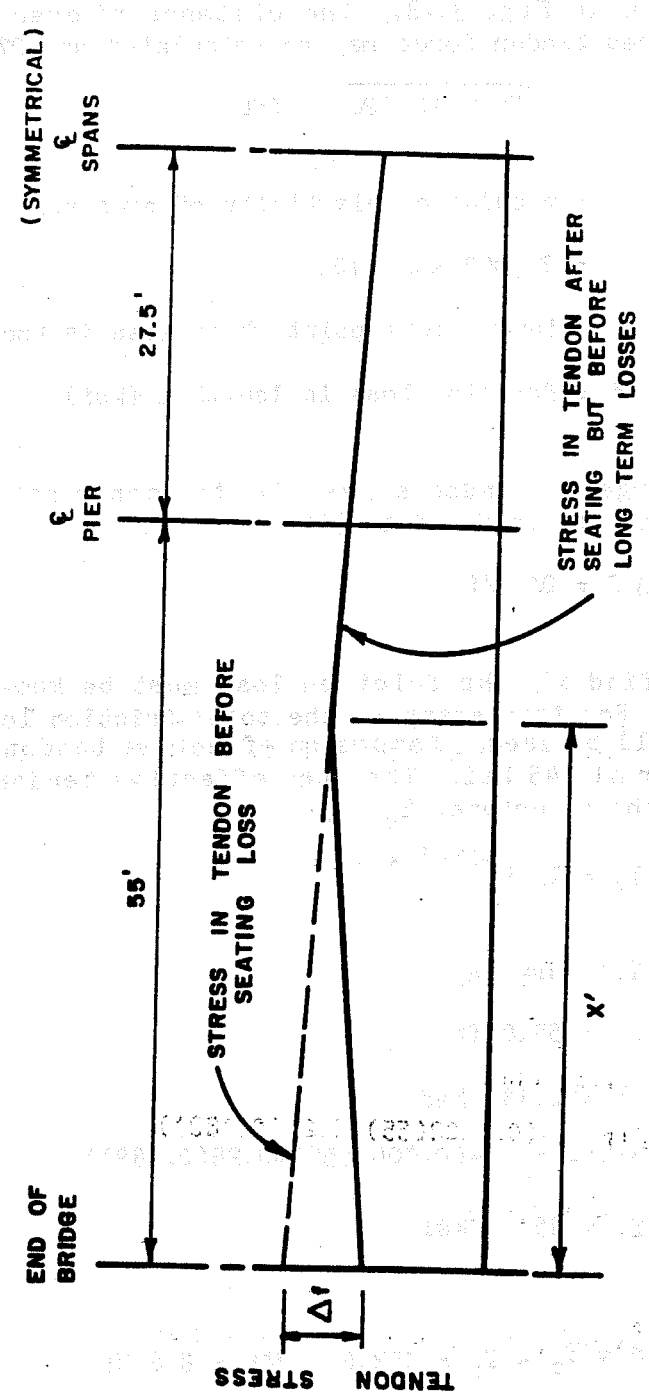


Fig. 3.18 Stress in longitudinal posttensioning tendon

and:

$$\Delta f = (2(8.6)60.0)/55 = 18.8 \text{ ksi}$$

Note that since x' is greater than 55 ft, the tendon stress at the interior pier is affected by the anchor set losses. However, since the magnitude of the loss at the pier will be so low (approximately 1.6 ksi), it will be ignored at that location.

By similar triangles, anchor set loss at the center of the endspan is:

$$(60 - 27.5)\ 18.8/60 = 10.2 \text{ ksi}$$

Since tendon stress is proportional to tendon force, the anchorage loss can also be expressed as a percentage of the tendon force equal to:

$$(10.2/145)\ (100) = 7.0 \text{ percent}$$

Finally, the effective tendon force, F_e, including anchorage and friction losses, at the center of the endspan for a force of 277.9 k at the pier is:

$$F_e = (1 - 0.07)\ F_o = (0.93)289.3 = 269.1 \text{ k}$$

From Fig. 3.17(b) is can be seen that the secondary moment due to posttensioning at the center of the endspan is 1/2 the maximum amount.

$$M_S = (0.339/2)F_e = (0.169)F_e$$

Total moment at the midpoint of the endspan due to prestress is then:

$$M_{PT} = F_e\ e + M_S$$

$$= (269.1)\ (-8/12) + (0.169)\ (269.1)$$

$$= -133.9 \text{ k-ft}$$

The stress at middepth of the deck under service load is:

$$f = F_e/A + M_{PT}c/I + M_c c/I$$

$$= 269.1/1407 - (133.9(12)9.47)/319,800$$

$$- (151(12)9.47)/319,800$$

$$= 0.090 \text{ ksi}$$

Thus, the compression stress in the slab is 10 psi less at this location than the desired value of 100 psi. This is considered acceptable for this example. However, in practice a designer might decide a stress value closer to 100 psi is more appropriate. In that case, step 6 would be performed.

Step 6--Adjust Tendon Profile

Because the required 100 psi average compression stress was obtained within reason at the locations checked, the tendon profile shown in Fig. 3.17(a) does not need adjustment. If, however, the designer felt that the compressive stress should not fall below 100 psi, the drape of the prestressing would have to be reduced in the end spans, and calculations recycled from step 4.

Having obtained a satisfactory tendon profile and the effective prestress force required, the amount of prestressing steel can be determined. Since in this example the tendon force at the pier controls, the prestress force at the girder end after long-term losses have taken place is found using:

$$F_x = F_o \, e^{-(KL' + \mu\alpha)}$$

with: $F_x = 277.9$ k (at pier)

$L' = 55.0$ ft

$\alpha = 0.188$ rad

$$277.9 = F_o \, e^{-(0.0002(55.0)+0.25(0.188))}$$

$$F_o = 294.5 \text{ k}$$

For Grade 270 prestressing strand, initial tendon stress after seating at the anchor of $0.7f'_s$, and long-term losses of 33 ksi, the effective tendon stress at the girder end after losses, T_e, is:

$$T_e = (0.7)f'_s - 33$$

$$= (0.7)\ 270 - 33 = 156 \text{ ksi}$$

The required area of posttensioning steel, A_s^*, per girder is found as:

$$A_s^* = F_o/T_e = 294.5/156 = 1.89 \text{ in.}^2$$

For 1/2-in. diameter strands with an area of 0.153 sq.in. each, 13 strands are needed. These can easily be placed in two 2-in. diameter ducts. Use 14 strands total, 7 in each duct.

Step 7--Moment Capacity Due to Posttensioning

The moment capacity due to the longitudinal posttensioning, M_{cp}, can be determined as the moment which would produce an extreme fiber stress in the composite girder equal and opposite to the stress produced by the posttensioning. Effective tendon forces and moments due to prestressing will be conservatively assumed to be calculated as in steps 4 and 5. In the end spans:

$$M_{cp}y_b/I = F_e/A + M_{PT}y_b/I$$

$$(M_{cp}(12)34.53)/319,800 = 269.1/1407 + (133.9(12)34.53)/319,800$$

$$M_{cp} = 281.5 \text{ k-ft}$$

And in the center span:

$$M_{cp}(12)34.53/319,800 = 267.3/1407 + (76.4(12)34.53)/319,800$$

$$M_{cp} = 223.0 \text{ k-ft}$$

Step 8--Precast Girder Design

At this point, the precast girders can be designed using standard manual or computerized methods, considering the net loads imposed on them. These net loads consist of the full loads acting on the noncomposite section, M_{NC}, as found in step 1, plus the total load acting on the composite section (from step 1) less the moment

capacity due to posttensioning, M_{cp} (from step 7). The design moments for the precast girders for this example are given in Table 3.5.

TABLE 3.5 Design Moments for Precast Girders In Design Example 3

	Design Moments (k-ft)	
	Noncomposite Section	Composite Section
End spans	554	341
Center span	554	282

For the section properties given in Table 3.4, loads as shown in Table 3.5, concrete with f'_c = 5400 psi, and an allowable extreme fiber concrete tensile stress of 3 $\sqrt{f'_c}$, approximately 14 1/2-in. diameter pretensioning strands are needed in all the precast girders.

Step 9--Verify Ultimate Capacity

The ultimate strength of the composite girders must be checked. The process is very similar to that described in Sec. 3.4.6 for the transverse prestress design. Since the calculations are similar for all the critical locations (near midspan of end spans, midspan of center span, and at interior piers), only those for the center span are shown here.

At midspan of the center span, the ultimate moment, M_u, is:

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

$$= 1.3\,(1.0(554.3) + 1.67(505))$$

$$= 1817 \text{ k-ft}$$

$$A_s^* = A_s^* \text{ (post-tensioned)} + A_s^* \text{ (pretensioned)}$$

$$= 14(0.153) + 14(0.153) = 4.284 \text{ in.}^2$$

For the top slab in compression, the effective flange width, b_e, is found by AASHTO Sec. 8.10.1 as:

$$b_e \leq \text{span}/4$$

$$\leq 2(6t) + \text{width of top flange of precast girder}$$

$$\leq \text{girder spacing}$$

$$\text{span}/4 = 55/4 = 13.75 \text{ ft}$$

$$2(6t) + \text{precast girder flange} = 2(6(8)) + 14 = 110 \text{ in.}$$

$$= 9.17 \text{ ft}$$

$$\text{girder spacing} = 9.50 \text{ ft}$$

Therefore, $b_e = 110$ in.

For a Texas Type C girder with 14 strands, the distance from the bottom of the girder to the centroid of the strands is approximately 2.6 in. at midspan. From Fig. 3.17(a), the distance from the top of the slab to the centroid of the post-tensioning strands at midspan of the center span is 20.97 in. The distance from the extreme compressive fiber to the centroid of the prestressing force, d, is then:

$$d = (14(20.97) + 14(48-2.6))/28$$

$$= 33.2 \text{ in.}$$

$$p^* = A_s^*/bd = 4.284/(110(33.2)) = 0.00117$$

$$f_{su}^* = f_s' (1 - 0.5 (p^* f_s'/f_c'))$$

$$= 270 (1 - 0.5 (0.00117(270)/4.5)) = 260.5 \text{ ksi}$$

The neutral axis generally falls below the bottom of the slab if:

$$t < 1.4\, d\, p^*\, f^*_{su}/\, f'_c \qquad \text{(AASHTO 9.17.3)}$$

$$= 1.4(33.2)\ 0.00117(260.5)/4.5$$

$$= 3.16 \text{ in.}$$

Since t = 8 in. > 3.16 in., the girder acts as a rectangular section at ultimate.

$$M_n = A^*_s\, f^*_{su}\, d\, (1 - 0.6\, (p^* f^*_{su})/f'_c)$$

$$= 4.284\ (260.5)\ 33.2\ (1 - 0.6\ (0.00117(260.5))/4.5)$$

$$= 35{,}540 \text{ k-in.} = 2962 \text{ k-ft}$$

$$M_n \geq M_u/\phi$$

$$2962 \geq 1817/0.95 = 1913 \text{ k-ft} \qquad \text{OK}$$

3.4.11 Final Details. Section P 1.10 of the proposed specifications requires the areas along the edges of the deck to be provided with nonprestressed reinforcing to resist the moments from lateral impact loads on the railing. The design of this reinforcing is not included here since it is not unique to transversely prestressed bridge decks.

Details similar to those shown in Fig. 3.13 will also be required for use with the transverse deck posttensioning in the third example bridge (P-1.11).

Two details in conjunction with the longitudinal posttensioning must be addressed in the design of the precast girders. First, the pretensioned strand arrangement must be compatible with the posttensioning tendon duct locations. This is especially critical for harped strands. Second, end blocks in the precast girders will be needed where posttensioning anchorages are to be installed.

Figures 3.19 through 3.22 illustrate the details of the transversely prestressed deck design for the three-span continuous example bridge.

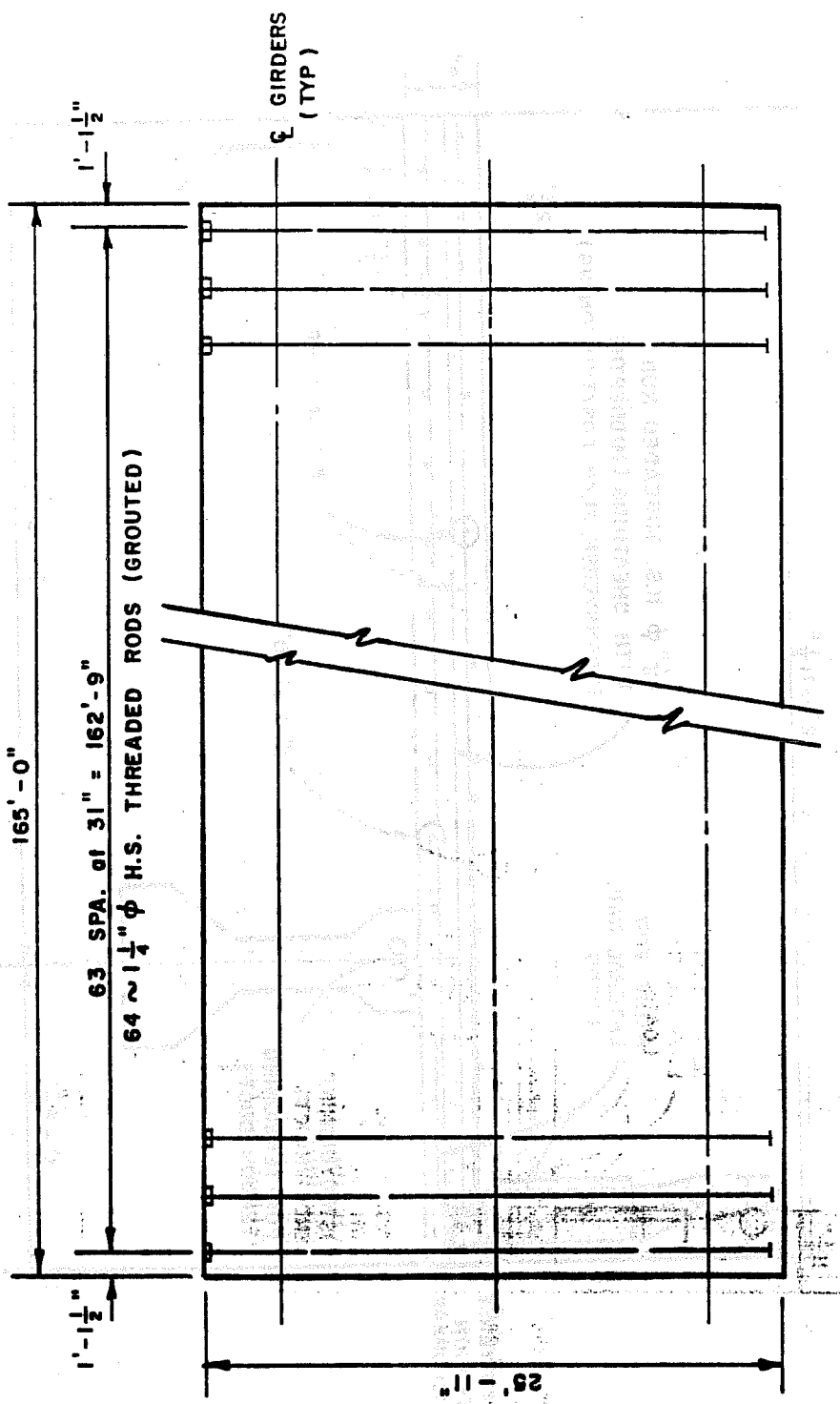


Fig. 3.19 Prestressing tendon layout in deck for design Example 3

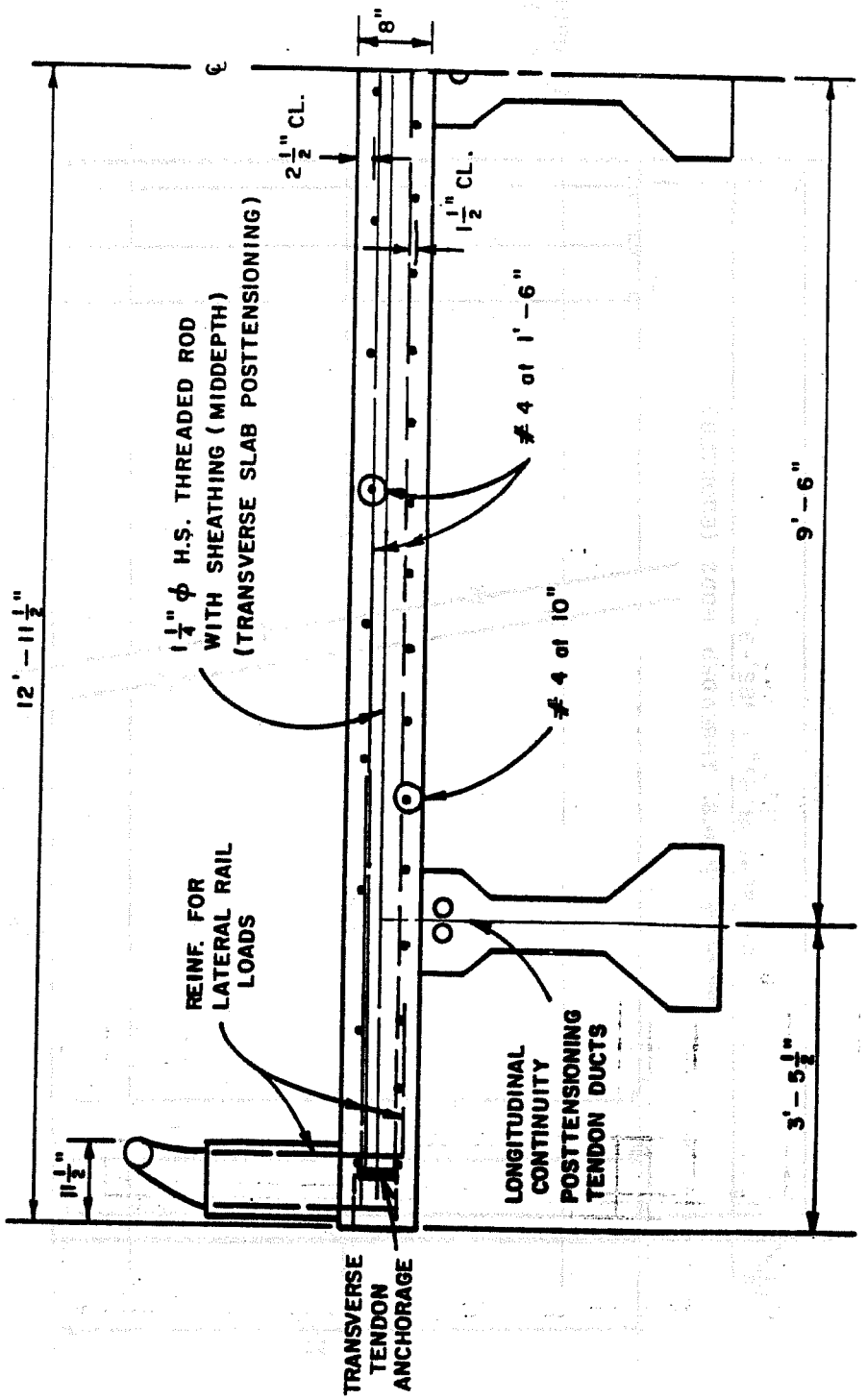


Fig. 3.20 Transverse section of deck for design Example 3

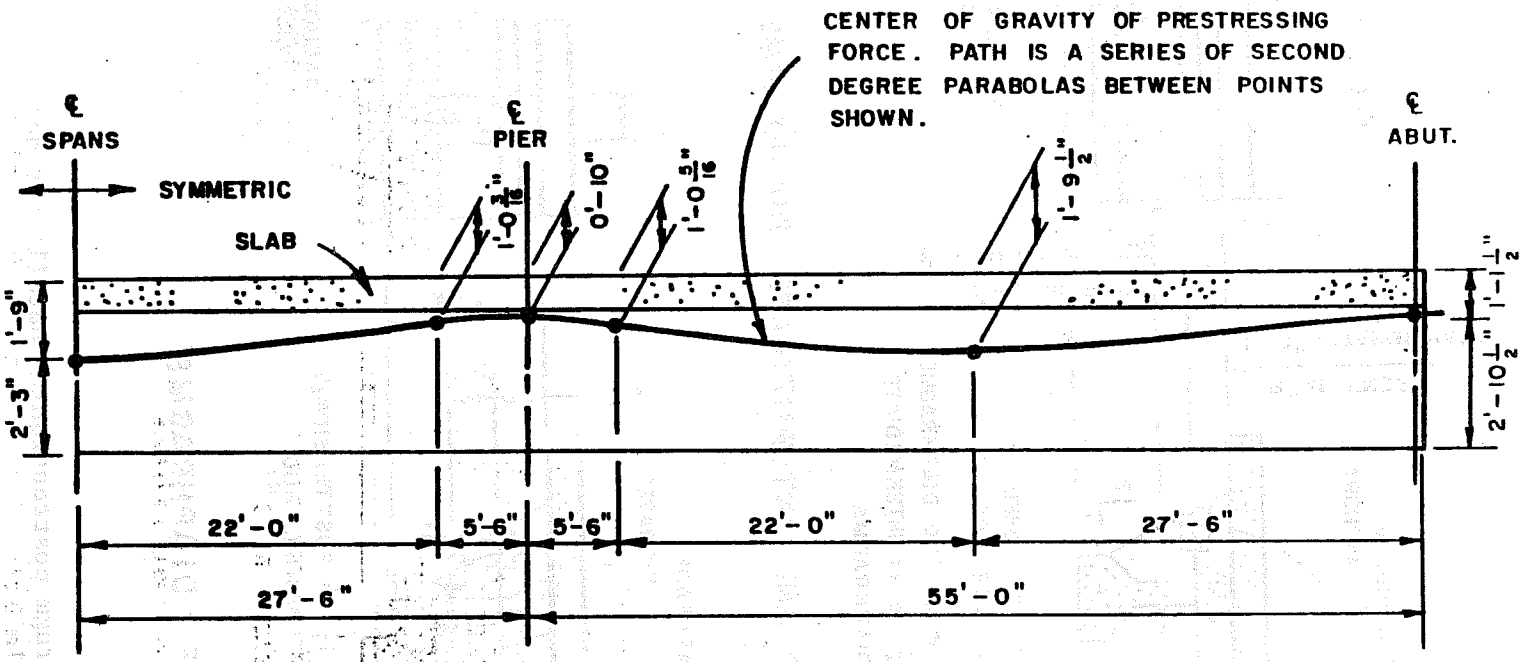
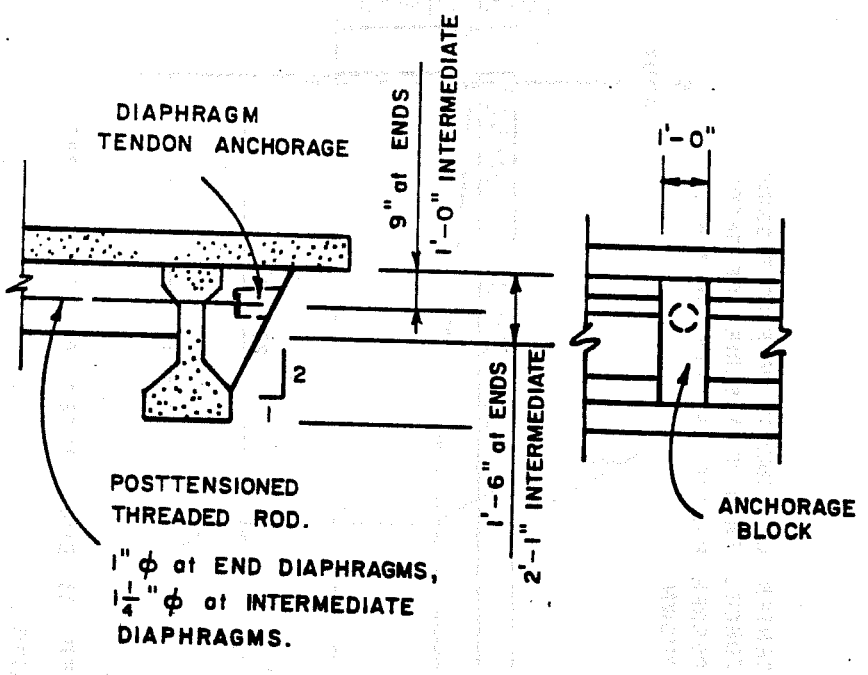
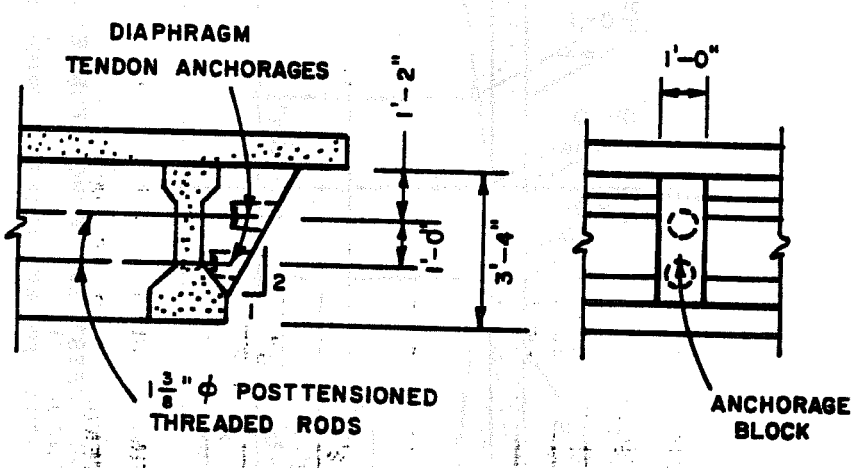


Fig. 3.21 Longitudinal section detail showing longitudinal post-tensioning requirements for design Example 3



a) END AND INTERMEDIATE DIAPHRAGMS



b) PIER DIAPHRAGMS

Fig. 3.22 Diaphragm posttensioning details for design Example 3

CHAPTER 4

COMPARISON OF TRANSVERSELY PRESTRESSED AND CONVENTIONALLY REINFORCED BRIDGE DECKS

4.1 Introduction

The choice of the type of deck to use for a given bridge depends on the characteristics each design considered imposes on the structure. This chapter examines the characteristic advantages and disadvantages of conventionally reinforced decks, of decks reinforced with epoxy-coated bars, and of transversely prestressed decks in terms of design, construction, performance, and cost. Costs are examined for design examples 2 and 3 of Chapter 3 since those bridges had previously been built in Texas with conventionally reinforced decks, and thus allowed for a more direct cost comparison between transversely prestressed and conventionally reinforced decks.

4.2 Design Effort

The level of effort required to design a concrete bridge deck for transverse moment is nearly the same for either a prestressed or a conventionally reinforced slab. The one difference is that while nonprestressed reinforcing may be determined using either working stress or ultimate design, the required prestressing is first found to satisfy allowable stresses, then must be checked for ultimate conditions.

Longitudinal nonprestressed reinforcing is determined by simple relationships for both reinforced and prestressed decks in simple span bridges. When longitudinal prestressing is used in the deck a few extra calculations are needed, but they are not lengthy.

The longitudinal design of bridges made continuous for live loads and for dead loads applied after composite action is effective is more complex for both conventionally reinforced and prestressed decks. For both types of decks, moment envelopes for the loads acting on the continuous structure must be computed. The required amount of negative moment reinforcing to be placed in the slab, together with the bar cut-off locations, are then found for a conventional deck. However, the calculations for a prestressed deck on a continuous structure which utilizes longitudinal posttensioning are more involved. An iterative process is used to determine an efficient longitudinal tendon profile and corresponding prestress force. This process includes calculations of secondary moments, prestress losses, and concrete stresses at numerous points along the spans. In addition, the girders must be designed in accordance with the longitudinal posttensioning scheme.

When done by hand, these calculations require significant effort. However, the nature of the calculations is such that any electronic computer program for the analysis and design of continuous prestressed concrete bridges could be utilized for this application with little or no modification.

Conventional reinforced concrete bridge decks are most often detailed using tabulated standard designs, with the exception of the longitudinal negative moment reinforcing in continuous structures. Such tables could also be easily developed for transversely prestressed bridge decks.

For a nominal increase in design effort, a more efficient deck design may be produced. As discussed in Report 316-2, the slab live load moments calculated by the empirical AASHTO formula (Sec. 3.24.3.1) appear to be conservative by a factor of 35 to 45 percent for a transversely prestressed deck on a typical slab and girder bridge. Substantial savings in prestressing steel could therefore be achieved by analyzing the deck (as allowed by AASHTO Sec. 3.24.3) to determine the design moments more exactly. Methods of analysis based on elastic theory are most appropriate for this purpose since a transversely prestressed deck exhibits linear elastic behavior well beyond service load levels as shown in Report 316-2. One such method which is easily applied is the use of influence surfaces [28,29]. Note the reported elastic behavior for the slab through factored load was for the laboratory model which had been designed using the conservative AASHTO live load moments. If a slab is designed using a more "exact" analysis of live load moments, it is expected that linear elastic slab behavior will cease at a load level lower than factored, but certainly greater than service.

4.3 Construction

Both conventionally reinforced and transversely prestressed bridge decks have their own distinct construction features. Because the entire concrete section is used to resist the loads in a prestressed deck, the slab thickness may sometimes be less than that for the same deck designed with nonprestressed reinforcing. This is not always true, however, since the slab thickness is also dependent on the clearance requirements of the prestressed and nonprestressed steel within the deck. For instance, in the second design example in Chapter 3, the prestressed deck was 8 in. thick as opposed to 7-1/2 in. for the conventional design. This was because of the required 2-1/2 in. concrete cover over the top reinforcement, and the 1-1/2 in. diameter sheathing for the mid-depth transverse tendons. If a multi-strand prestressing system with flat tendon ducts had been used for this design, a slab thickness of 7-1/4 in. could have been used.

Conventionally reinforced bridge decks feature transverse reinforcing bars spaced 4 to 7 in. apart. On the other hand, when multi-strand or threaded rod tendons are used, the spacing of the transverse tendons in a prestressed deck will usually be from 1.5 to 3.5 ft. The greater spacing of transverse steel in a prestressed deck allows easier placement and vibration of the concrete.

When exposure to a corrosive environment is anticipated, a conventionally reinforced deck will often be designed with epoxy-coated reinforcing in the top layer, greater concrete cover over the top reinforcing, and a waterproofing membrane on the upper slab surface. A prestressed deck, posttensioned in both directions, exposed to the same conditions, should require only the 2-1/2 in. concrete cover recommended in the suggested provisions. There should be less need for epoxy-coating of the nonprestressed reinforcing or for applying a waterproofing membrane.

The amount of labor required to construct a transversely prestressed bridge deck will most likely be greater than for a conventionally reinforced deck. While the reinforced deck has many more pieces of steel to be placed, the prestressed deck must be stressed and grouted (if bonded tendons are used), and the stressing pockets filled with mortar. Since the concrete must be cured before stressing, and the tendons must be stressed before grouting, these operations tend to be less efficient than the simple placement of reinforcing steel.

Existing bridges are often widened to accommodate additional traffic lanes. On a conventionally reinforced deck, the concrete along the edge where the widening is taking place is removed back towards the outside girder to allow the transverse reinforcing of the new structure to splice with that of the existing deck. Widening of a transversely prestressed deck would be somewhat more difficult. If the new deck was to be conventionally reinforced, the transverse bars could be drilled and grouted into the edge of the existing deck. If the new deck was to be transversely prestressed, one possibility would be to remove the end anchors of the existing tendons (bonded tendons only), couple the ends of the new and existing tendons, and stress the new tendons from their outside anchorages. This would be especially easy to do if the transverse prestressing consisted of high-strength threaded bars, as simple couplers are readily available for this system.

4.4 Performance

The performance of a bridge deck may be evaluated with respect to its structural behavior, maintenance requirements, and length of service life.

Both a conventionally reinforced and a transversely prestressed bridge deck should exhibit very satisfactory structural behavior, including a large capacity for overloads. Because the prestressed deck is flexurally stiffer, it will deflect less under load than a comparable conventional slab.

Maintenance requirements will likely be significantly greater for a conventionally reinforced deck than for a prestressed deck. This is because unlike the prestressed deck, the reinforced slab must crack in order to develop significant moment resistance. As discussed in Chapter 1, and assuming adequate concrete cover, quality and compaction, it is cracking of the deck which provides the major avenues of penetration for corrosion-producing elements and exposes the concrete to freeze-thaw deterioration. It then follows that a prestressed deck, particularly if posttensioned both longitudinally and transversely, will not need as much maintenance as a conventionally reinforced deck since it remains largely uncracked. If cracked by overload or other causes, the cracks are closed by the prestress action in contrast to the wider permanent cracks in conventionally reinforced decks. This assertion is supported by the results of the durability study, as discussed in Report 316-1.

Should damage to the deck occur, such as due to traffic impact against the railing, repair procedures would be more difficult for a transversely prestressed deck. This is particularly true if the transverse tendons are unbonded. While damage to a conventional deck is usually localized, a damaged unbonded tendon anchor may affect the entire deck width. Furthermore, replacement of a tendon anchorage or of the tendon itself could be a difficult procedure. Again, it should be noted that if the prestressing consists of threaded rods, coupling tendon ends and installing tendon anchorages are greatly simplified.

Because of the almost universal concrete cracking to be expected in decks reinforced with mild steel as discussed previously, it is expected that transversely prestressed bridge decks will have a longer service life than conventional decks. Due to the possibility of freeze-thaw deterioration, this is true whether or not the conventional deck utilizes epoxy-coated reinforcing steel. The service life of transversely prestressed bridge decks should be extended even further through the use of longitudinal slab prestressing.

4.5 Cost

The primary criteria for selecting a particular structural system is most often cost. The cost considered can be the initial construction cost or the life cycle cost of the system. This section discusses these two types of costs with reference to design examples 2 and 3 in Chapter 3. These two bridges were constructed in Texas as conventionally reinforced deck, and thus a more direct cost comparison

between transversely prestressed and conventionally reinforced decks could be made. Four construction options for each bridge were studied. These are:

1. Conventionally reinforced deck as shown in Figs. 3.9 and 3.15.

2. Conventionally reinforced deck except with epoxy-coated reinforcing in the top layer and a waterproofing membrane with 2 in. of asphalt over the top slab surface.

3. Transversely prestressed deck with epoxy-coated non-prestressed reinforcing in the top layer.

4. Combined transversely and longitudinally prestressed deck as shown in Figs. 3.11, 3.12, and 3.19 through 3.22, with uncoated reinforcing steel.

4.5.1 Construction Cost. Initial construction cost estimates for the four construction options of example bridges 2 and 3 are shown in Tables 4.1 and 4.2. These estimates include all concrete, reinforcing, prestressing and surfacing for the deck, as well as the transverse diaphragm prestressing where used. The longitudinal continuity prestressing steel for the third example is included, but it is assumed that the decreased pretensioned strand costs are offset by the posttensioning ducts in the precast beams. All construction options of the second example bridge include a 2-in. asphalt wearing surface since it was specified in the original conventionally reinforced slab design. Excluded from the estimates are traffic railings, diaphragm concrete and reinforcing, armored expansion joints, girders, and substructure elements.

The prices used in developing the cost estimates were obtained from three sources. Prices for the 3 ksi concrete, asphalt surfacing, and concrete surface treatment were taken from average 1984 Texas Highway Department bid tabulations. An average price quote from material suppliers in Texas was reported to be 5 percent greater for 5 ksi concrete than for 3.6 ksi concrete [12]. Thus, unit costs for the 4 and 4.5 ksi concretes were determined as 5 percent greater than the 3 ksi concrete. Reinforcing steel, epoxy-coated reinforcing steel, and waterproofing membrane prices were obtained from average 1983 Colorado Highway Department bid tabulations, increased by 5 percent to adjust to 1984 costs. Note that the Colorado figures were used for these items because nearly all bridges constructed with concrete decks in Colorado in recent years have been provided with epoxy-coated reinforcing in the top layer of the deck and a waterproofing membrane under an asphalt wearing surface. The unit costs for prestressing steel were obtained from 1984 price quotes from material suppliers in Texas. All prices include associated costs such as formwork, labor, and stressing and grouting of prestress tendons.

TABLE 4.1 Construction Costs for Deck of Design Example 2

Bid Item Descrip.	Unit Cost	Conventionally Reinforced Qty.	Conventionally Reinforced Cost	Conventionally Reinforced w/Top Bars E.C. Qty.	Conventionally Reinforced w/Top Bars E.C. Cost	Transversely Prestressed w/Top Bars E.C. Qty.	Transversely Prestressed w/Top Bars E.C. Cost	Transversely & Long. Prestressed Qty.	Transversely & Long. Prestressed Cost
Conc. (3 ksi) (cy)	185	46.6	8621	46.6	8621	—	—	—	—
Conc. (4 ksi) (cy)	195	—	—	—	—	48.9	9536	48.9	9536
Reinf. Stl (lb)	0.35	10,190	3567	3970	1390	2480	868	4266	1493
Reinf. Stl (coated) (lb)	0.45	—	—	6220	2799	1790	806	—	—
Prestr. Stl (bars) (lb)	1.15	—	—	—	—	3070	3531	3070	3531
Prestr. Stl (mono-strand) (lb)	1.15	—	—	—	—	—	—	500	575
Water-proof Memb. (sy)	7.50	—	—	220	1650	—	—	—	—
Asph. Conc. (surf.) (cy)	63.00	12.2	769	12.2	769	12.2	769	12.2	769
Total Deck Cost ($) (nearest $100)		13,000		15,200		15,500		15,900	

TABLE 4.2 Construction Costs for Deck of Design Example 3

Bid Item Descrip.	Unit Cost	Conventionally Reinforced Qty.	Conventionally Reinforced Cost	Conventionally Reinforced w/Top Bars E.C. Qty.	Conventionally Reinforced w/Top Bars E.C. Cost	Transversely Prestressed w/Top Bars E.C. Qty.	Transversely Prestressed w/Top Bars E.C. Cost	Transversely & Long. Prestressed Qty.	Transversely & Long. Prestressed Cost
Conc. (3 ksi) (cy)	185	105.6	19,536	105.6	19,536	—	—	—	—
Conc. (4.5 ksi) (cy)	195	—	—	—	—	106.7	20,807	106.7	20,807
Reinf. Stl (lb)	0.35	24,950	8733	7940	2779	4440	1554	7340	2569
Reinf. Stl (coated) (lb)	0.45	—	—	17,010	7655	5980	2691	—	—
Prestr. Stl (lb)	1.15	—	—	—	—	8480	9752	12,156	13,979
Water-proof Memb. (sy)	7.50	—	—	158.4	1188	—	—	—	—
Asph. Conc. (surf.) (cy)	63.00	—	—	26.4	1663	—	—	—	—
Conc. Surf. Treat (sy)	0.85	158.4	135	—	—	158.4	135	158.4	135
Total Deck Cost ($) (nearest $100)		28,400		32,800		34,900		37,500	

The estimated construction costs are shown in a more useful form in Tables 4.3 and 4.4. These tables give the cost of the deck per sq.ft. of area (unit deck cost) and the cost of the deck normalized with respect to the conventionally reinforced slab without coated bars. Also presented are the total bridge costs per sq.ft. of deck area (including both superstructure and substructure) and the normalized bridge costs with respect to the conventionally reinforced deck bridge with uncoated reinforcing. Total bridge costs were calculated based on an initial cost of $25 per sq.ft. for a conventionally reinforced slab on prestressed girders. This cost was obtained from the Texas Highway Department and is the approximate average construction cost per sq.ft. of concrete slab and girder bridges built in Texas during 1984.

It can be seen from Tables 4.3 and 4.4 that any type of design which incorporates features which should appreciably increase the durability of bridge decks increases the construction cost somewhat. This cost increase appears to be the least for the conventionally reinforced deck with epoxy-coated reinforcing in the top layer and a waterproofing membrane, and the greatest for the transversely prestressed deck with longitudinal posttensioning. Note, however, that the increase in cost is fairly close for the different construction options designed to increase durability, especially for the simple span bridge. The construction cost increase for the durable designs on the simple span bridge ranges from 17 to 22 percent of the deck cost, or 4 to 6 percent of the total bridge cost. On the continuous bridge, this cost increase is from 15 to 32 percent of the deck cost, and 4 to 9 percent of the bridge cost.

It can be said that the differences in estimated initial costs of the various construction methods for increasing deck durability are small enough that all the methods appear to be competitive solutions with each other on a first cost basis. However, as discussed in Sec. 4.3, the performance of these deck systems may vary. Also the construction cost of the prestressed deck could be reduced by utilizing Grade 270 prestressing steel instead of Grade 150 since it is more efficient.

4.5.2 Life Cycle Cost. The life cycle cost of the bridge deck should take into account the initial construction cost, all maintenance and rehabilitation costs, and the expected service life. Since each of the four construction options may have a different service life, the most convenient method of judging relative costs is by average annual cost of the slab. The average annual cost can be defined as the equivalent uniform annual cost of a nonuniform series of money disbursements where money has a time value. The procedure for calculating this annual cost once assumptions have been made regarding the timing and amount of disbursements can be found in any engineering economy text [30].

TABLE 4.3 Initial Cost Figures for Bridge of Design Example 2

	Conv. Reinf.	Conv. Reinf. w/Top Bars Epoxy Coated	Trans. Prestressed w/Top Bars Epoxy Coated	Trans. & Long. Pre-stressed
Total deck cost ($)	13,000	15,200	15,500	15,900
Unit deck cost ($/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

TABLE 4.4 Initial Cost Figures for Bridge of Design Example 3

	Conv. Reinf.	Conv. Reinf. w/Top Bars Epoxy Coated	Trans. Prestressed w/Top Bars Epoxy Coated	Trans. & Long. Pre-stressed
Total deck cost ($)	28,400	32,800	34,900	37,500
Unit deck cost ($/ft^2)	6.64	7.67	8.16	8.77
Normalized deck cost	1.00	1.15	1.23	1.32
Total unit bridge cost ($/ft^2)	25.00	26.03	26.52	27.13
Normalized bridge cost	1.00	1.04	1.06	1.09

Average annual costs for each of the four construction options for bridge decks of design examples 2 and 3 were calculated per sq.ft. of slab. For these comparisons, a time value of money (interest rate) of 6 percent per year was chosen [31]. Other assumptions concerning required expenditures and service life of each option are discussed below.

a) Construction Cost

The initial construction cost assumed for each option is the unit deck cost found in Tables 4.3 and 4.4.

b) Routine Maintenance

The cost of minor repairs required to maintain a bridge deck in a safe condition is assumed to be $0.10/sq.ft. per year for all the slab construction options.

c) Deck Rehabilitation

An examination of bid tabulations for 13 bridge deck rehabilitations carried out in the state of Colorado in 1983 and 1984 produced the following statistics. Deck rehabilitations were required at an average slab age of 17 years and cost an average of $11.36/sq.ft. These projects typically involved: removal and replacement of the top 3/4 in. of concrete over most of the deck; more extensive removal and replacement of concrete over an average of 43 percent of the slab area; replacement of the guardrail and transverse slab expansion joints; and installation of a waterproofing membrane and asphalt wearing surface. Signing, traffic control, and contractor mobilization accounted for an average of 19 percent of the rehabilitation cost.

In consideration of the above information, it is assumed that rehabilitation of conventionally reinforced decks not provided with epoxy-coated steel is required 15 years after initial construction, and costs $11.00/sq.ft. It is further assumed that major restoration work is not required for conventionally reinforced decks with epoxy-coated bars or for transversely prestressed bridge decks over their entire service lives.

d) Wearing Surface

It is assumed that an asphalt riding surface must be overlaid every five years [32], and that a bare concrete riding surface is initially overlaid with asphalt after ten years of service. The asphalt riding surface is assumed to cost $0.30/sq.ft. This figure is derived from Texas Highway Department 1984 bid tabulations for asphalt concrete surfacing, assuming a 1-in. thick surface and including factors for removal of the previous asphalt surface, signing, traffic control, and mobilization.

e) Expansion Joints

The transverse deck expansion joints are assumed to require complete replacement every ten years regardless of the deck type. The cost of removing and replacing the joints is assumed to be $45 per linear ft of joint and was derived from Texas bid tabulations with factors included for signing, traffic control, and mobilization. With expansion joints at each end of the example bridges, this cost is equivalent to $1.75/sq.ft. for the second example deck and $0.55/sq.ft. for the third example deck. Joint replacement is included in deck rehabilitation for the conventionally reinforced deck with uncoated reinforcing.

f) Service Life

The actual total service life of a deck is dependent on many factors and is difficult to determine accurately. The same deck design will have a significantly varying length of service influenced by the quality of construction, exposure conditions, type and volume of traffic, and the amount and quality of maintenance. For this study, the following average total service lives are assumed, considering the decks exposed to deicing salts and freeze-thaw conditions:

Conventionally reinforced	--25 years
Conventionally reinforced with top bars epoxy-coated and with waterproofing membrane	--30 years
Transversely prestressed with top bars epoxy-coated	--30 years
Transversely and longitudinally prestressed with uncoated reinforcing	--35 years

These service lives were chosen on the following bases. The data from the State of Colorado shows that major rehabilitation of conventionally reinforced decks with uncoated steel is required at an average slab age of 17 years. It is most likely that not all chloride contaminated concrete will be removed during a deck rehabilitation. Furthermore, the original reinforcing steel usually is left uncoated. Thus, it is reasonable to expect the remaining service life of a rehabilitated deck to be less than the time from construction to first restoration. If a conventionally reinforced deck with uncoated steel is rehabilitated at 15 years of age (as previously assumed), then a total service life of 25 years would seem to be appropriate.

A conventionally reinforced deck with the top bars epoxy-coated and a waterproofing membrane is protected to some degree against

both corrosion and freeze-thaw deterioration. However, since this protection is not absolute, it is felt that a service life of 30 years without major restoration may be reasonably expected.

The transversely prestressed deck with top bars epoxy-coated is protected from corrosion and freeze-thaw by the prevention of cracking in the longitudinal direction. Cracking of the slab in the transverse direction is very likely, however, and will allow some amount of contaminants to penetrate the deck. Although the epoxy-coated steel will inhibit the corrosion process, freeze-thaw deterioration remains a threat. It is estimated that the durability of this deck system is approximately equivalent to that of the conventionally reinforced deck with epoxy-coated steel, and thus a service life of 30 years is deemed reasonable.

A bridge deck prestressed in both the transverse and longitudinal directions should remain essentially uncracked. Any cracks which do develop should close again after the load is removed. Thus, as shown in the durability phase of the study, water and chloride penetration will be minimal. This method of protection against corrosion and freeze-thaw damage seems more reliable than either the conventionally reinforced deck with epoxy-coated bars and waterproofing membrane or the deck prestressed only in the transverse direction with epoxy-coated reinforcing. An expected life of 35 years then seems appropriate.

At the end of its service life, a deck is assumed to have zero salvage value.

The assumed required expenditures throughout the service life of each deck type are summarized in Table 4.5.

The disbursements shown in Table 4.5 have been reduced to average annual costs for each deck construction option. These average annual costs are presented in Tables 4.6 and 4.7. Besides annual costs per sq.ft. of slab, an annual deck cost normalized with respect to the conventionally reinforced slab with uncoated bars is also given.

From Tables 4.6 and 4.7, it can be seen that based on these assumptions the life cycle cost of bridge decks constructed with features for increasing durability is significantly less than that of an unprotected conventionally reinforced deck. The amount of savings in deck cost appears to be approximately 20 percent for the simple span bridge, and 25 percent for the continuous structure (larger area of deck for same number of expansion joints). Between the three construction options for increased durability, the life cycle cost is fairly uniform. Thus, a conventionally reinforced deck with epoxy-coated steel, a transversely prestressed deck with epoxy-coated reinforcing, and a deck prestressed both longitudinally and

TABLE 4.5 Assumed Required Expenditures Throughout Deck Service Life

Deck Type	Year of Required Expenditure					
	Routine Maintenance ($0.10/ft^2)	Rehabilitation ($11.00/ft^2)	Resurfacing ($0.30/ft^2) Example 2	Resurfacing ($0.30/ft^2) Example 3	Expn. Joint Replacement ($85.00/lin ft)	Total Service Life
Conventionally reinforced	annual	15	5,10,20	10,20	-----	25
Conv. reinforced w/top bars E.C.	annual	—	5,10,15...25	5,10,15...25	10,20	30
Trans. prestressed w/top bars E.C.	annual	—	5,10,15...25	10,15...25	10,20	30
Trans. & long. prestressed	annual	—	5,10,15...30	10,15...30	10,20	35

TABLE 4.6 Average Annual Deck Costs ($/ft^2) for Bridge of Example 2

	Conv. Reinf.	Conv. Reinf. w/Top Bars Epoxy Coated	Trans. Prestressed w/Top Bars Epoxy Coated	Trans. & Long. Pre-stressed
Initial construction	0.51	0.56	0.57	0.55
Routine maintenance	0.10	0.10	0.10	0.10
Rehabilitation	0.36	----	----	----
Resurfacing	0.04	0.05	0.05	0.05
Expansion joint replacement	----	0.11	0.11	0.11
Total avg. annual deck cost	1.01	0.82	0.83	0.81
Normalized annual deck cost	1.00	0.81	0.82	0.80

TABLE 4.7 Average Annual Deck Costs ($/ft^2) for Bridge of Example 3

	Conv. Reinf.	Conv. Reinf. w/Top Bars Epoxy Coated	Trans. Prestressed w/Top Bars Epoxy Coated	Trans. & Long. Pre-stressed
Initial construction	0.52	0.56	0.59	0.61
Routine maintenance	0.10	0.10	0.10	0.10
Rehabilitation	0.36	----	----	----
Resurfacing	0.02	0.05	0.03	0.03
Expansion joint replacement	----	0.03	0.04	0.03
Total avg. annual deck cost	1.00	0.74	0.76	0.77
Normalized annual deck cost	1.00	0.74	0.76	0.77

transversely all are competitive with each other on a life cycle cost basis.

The life cycle costs of the decks constructed for improved durability are not particularly sensitive to the assumed service life. This is because after a certain number of years, the reduced annual cost of construction is balanced by the increased maintenance costs. For instance, if the service life of the deck prestressed both transversely and longitudinally is assumed to be 40 years instead of 35, the annual deck cost remains at $0.81/sq.ft. for the simple span bridge, and only decreases from $0.77/sq.ft. to $0.76/sq.ft. for the continuous bridge. In this case, the increased service life requires another deck resurfacing and replacement of the expansion joints.

4.6 Summary of Advantages and Disadvantages

The level of effort required to design a transversely prestressed bridge deck is nearly the same as that for a conventionally reinforced deck. Computations for longitudinally posttensioning a continuous slab and girder bridge are more complex than those for determining conventional longitudinal continuity reinforcing in the deck, but existing computer programs for the design of continuous posttensioned concrete bridges may be used for this application. Tables of standard designs for transversely prestressed decks, similar to those used for conventionally reinforced decks, may be easily developed.

A transversely prestressed deck may be thinner than a conventionally reinforced slab for the same application, and offers less congestion of steel in the slab, easing concrete placement. A higher degree of care must be taken in placement and inspection of the deck posttensioning to ensure that correct corrosionproofing encapsulation and proper cover over the tendons is provided. Future widening of a deck constructed with transverse prestressing may be more difficult than widening of a reinforced concrete slab.

While both transversely prestressed and conventionally reinforced decks perform well structurally, the prestressed deck should require less maintenance and have a longer service life, especially if longitudinal deck posttensioning is used. However, repairs that are required will likely be more difficult for a prestressed deck than for a reinforced concrete deck.

The initial construction cost of the three types of decks intended to increase durability (conventional with coated reinforcing and waterproofing membrane, transversely prestressed with coated reinforcing, and prestressed both directions) is greater than for a conventional slab with uncoated reinforcing. The increased cost in example bridges 2 and 3 varied from 15 to 32 percent of the deck construction and from 4 to 9 percent of the total bridge construction

cost. Although the conventionally reinforced deck with coated bars appears to be the least expensive of the more durable designs, it is believed that these three designs are close enough in construction cost to be competitive with each other.

Life cycle costs of all three deck designs for greater durability appear to be nearly equal and between 20 and 25 percent less than the life cycle cost of a conventionally reinforced concrete deck with uncoated reinforcing steel.

CHAPTER 5

SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

5.1 Summary

Premature deterioration of concrete bridge decks has become a major problem in the last twenty years. The primary causes of this deterioration are corrosion of the reinforcing steel and freeze-thaw action. This study has 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 experimental program associated with this study was presented in Reports 316-1 and 316-2. One phase of the experimental work studied transverse stress distribution in a bridge slab due to prestressing through the use of 1/2.23 scale laboratory model and a finite element model of a slab and girder bridge. A durability phase of the experimental program involved accelerated exposure testing of 24 full-thickness specimens simulating a portion of a bridge deck. Based on the results of the structural and durability studies, design recommendations for prestressed bridge decks were proposed. Those design recommendations were then translated into AASHTO Specification provisions for the application of prestressing to bridge decks.

Finally, the proposed design recommendations were applied to a series of design examples. The advantages and disadvantages of a prestressed and a conventionally reinforced bridge deck were discussed in terms of design effort, construction, performance, and cost. Initial construction and life cycle costs for two of the example decks were calculated and compared for four construction options, ranging from a conventionally reinforced deck to a deck prestressed transversely and longitudinally.

5.2 Conclusions

The major conclusions are based on the overall research study of the use of deck prestressing as a method of improving bridge deck durability.

1. The most desirable approach for the design of concrete bridge decks exposed to aggressive environments is to eliminate cracking altogether under normal loading conditions through the use of prestressing. This is supported by the results of the durability study (Report 316-1) 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, greatly reduces the potential for corrosion and 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 or box-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 posttensioning stress distribution tests (Report 316-2). 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 posttensioning the diaphragms themselves.

3. A prestressed deck designed in accordance with the recommendations presented in this report and the AASHTO slab live load moments, should exhibit essentially linear elastic behavior through factored load levels. 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.

4. The design recommendations, proposed AASHTO Specification provisions and design examples found in this report should give ample guidance for the design of prestressed bridge decks.

5. 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. Initial construction cost of a prestressed deck is 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. The life cycle cost of a prestressed deck is equivalent to that of a reinforced concrete slab with coated steel and is approximately 20 to 25 percent less than the life cycle cost of conventionally reinforced decks with uncoated steel.

6. Based on the above conclusions, the application of deck prestressing to complement adequate concrete cover, quality and compaction as protective measures is an effective and cost competitive method for increasing the durability of concrete bridge decks. The durability of a prestressed deck is substantially improved by combining properly encapsulated transverse and longitudinal posttensioning to achieve virtually a crack-free deck under service load conditions. Unlike a conventionally reinforced deck with epoxy-coated steel, a crack-free deck is resistant to freeze-thaw deterioration and fatigue in addition to reinforcing corrosion.

7. For maximum effectiveness in corrosion resistance, the posttensioning 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.

5.3 Recommendations

The concept of prestressing bridge decks has been shown to be viable as well as advantageous, and guidelines have been developed for its implementation. The following actions are therefore recommended:

1. The proposed AASHTO provisions for prestressed bridge decks included in this report should be assimilated into the AASHTO Specifications. These provisions, together with the design

recommendations of Chapter 2 should be followed for the design of prestressed bridge decks. Specific guidance is given in these sources as well as in the examples of chapter 3 on aspects of design such as the amount of prestressing required to overcome diaphragm restraint, maximum spacing of prestressing tendons, maximum allowable tensile concrete stress, bonded reinforcing requirements, and detailing requirements for exposure to corrosive environments.

2. Highway bridges located in areas where exposure to deicing salts and freeze-thaw conditions are expected, or in a marine environment, should be constructed with prestressed decks designed in accordance with the recommendations presented in this report. Such decks should preferably be prestressed in both the longitudinal and transverse directions. If only transverse prestressing is utilized, however, the non-prestressed reinforcing in the top of the deck should be epoxy-coated.

3. As a minimum of protection, it is recommended that conventionally reinforced concrete bridge decks which are exposed to deicing salts use epoxy-coated reinforcement for at least the top mat of steel. For conventionally reinforced decks exposed to chlorides on both top and bottom deck surfaces such as from marine environments, epoxy-coated reinforcement should be used for all deck steel.

4. Further research should be carried out in relation to developing a simple yet accurate method of analysis for determining moments in bridge decks. Research to develop guidelines for the design of anchorage zones where multiple prestress tendons are anchored in a thin concrete slab is also needed.

5. The first prestressed bridge deck to be built using the recommendations in this report is to be constructed over the Colorado River in Fayette County, Texas, in 1985-6. Located on State Highway 71 near LaGrange, the 15-span bridge features an 86-ft wide concrete deck supported on 10 lines of Texas Type C prestressed girders. Portions of the deck will be constructed as conventionally reinforced, transversely prestressed, and prestressed in both the transverse and longitudinal directions. A program should be set up to monitor both construction and service behavior of this bridge.

6. Feedback from the construction and performance of full-scale prestressed bridge decks, together with the results of further research when available, should be used to refine the design recommendations presented in this report.

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