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Precast Prestressed Concrete Double-T Beams For Short-Span Bridges

OSMAN HAG-ELSAFI

RESEARCH REPORT 169
TRANSPORTATION RESEARCH AND DEVELOPMENT BUREAU
NEW YORK STATE DEPARTMENT OF TRANSPORTATION
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Please find enclosed a copy of Research Report 169 "Precast Prestrssed Concrete Double-T Beams for Short-Span Bridges."

ABSTRACT OF REPORT

Short-span bridges (having span lengths from 25 to 60 ft) account for half the publicly owned structures in New York State, which suggests the potential benefits for the State Department of Transportation and others from further economies in construction of bridges in this span range. Two sets of standard sections and details are developed in this report for precast double-T beams for short-span bridges: 1) for use with a non-composite asphalt wearing surface for rapid construction, and 2) for use with a cast-in-place composite deck for enhanced durability. The proposed standards are intended as viable alternatives to voided slabs (the current New York State standard for precast short-span bridges), based on practicality and economic advantages. Details for rapid construction and durability of double-T beam systems used with asphalt wearing surfaces include 1) standard precast prestressed beams, 2) grouted longitudinal shear keys, 3) transverse post-tensioning of the beams through their flanges, 4) precast post-tensioned diaphragms, 5) waterproofing membranes, and 6) fixed and expansion supports. Proposed standards for beams used with cast-in-place decks include 1) standard precast prestressed beams with shallower flanges, 2) cast-in-place post-tensioned diaphragms, and 3) a composite deck made continuous for live load over fixed and expansion supports. It is concluded that 20- to 30-percent savings in materials may be possible if the proposed beams are used instead of the current voided slabs.

Sincerely,

Dr. Robert J. Perry
Director
Transportation Research and
Development Bureau

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PRECAST PRESTRESSED CONCRETE DOUBLE-T BEAMS FOR SHORT-SPAN BRIDGES

Osman Hag-Elsafi, Engineering Research Specialist I

Report on a Study Conducted in Cooperation With
The U.S. Department of Transportation
Federal Highway Administration

Research Report 169
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16. Abstract

Short-span bridges (having span lengths from 25 to 60 ft) account for half the publicly owned structures in New York State, which suggests the potential benefits for the State Department of Transportation and others from further economies in construction of bridges in this span range. Two sets of standard sections and details are developed in this report for precast double-T beams for short-span bridges: 1) for use with a non-composite asphalt wearing surface for rapid construction, and 2) for use with a cast-in-place composite deck for enhanced durability. The proposed standards are intended as viable alternatives to voided slabs (the current New York State standard for precast short-span bridges), based on practicality and economic advantages. Details for rapid construction and durability of double-T beam systems used with asphalt wearing surfaces include 1) standard precast prestressed beams, 2) grouted longitudinal shear keys, 3) transverse post-tensioning of the beams through their flanges, 4) precast post-tensioned diaphragms, 5) waterproofing membranes, and 6) fixed and expansion supports. Proposed standards for beams used with cast-in-place decks include 1) standard precast prestressed beams with shallower flanges, 2) cast-in-place post-tensioned diaphragms, and 3) a composite deck made continuous for live load over fixed and expansion supports. It is concluded that 20- to 30-percent savings in materials may be possible if the proposed beams are used instead of the current voided slabs.

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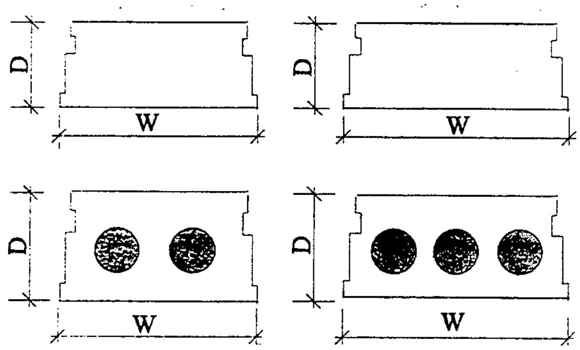
Table 1. Simple-span multi-stringer bridges in New York State.

Span Length Range, ft	Steel	Prestressed Concrete	Reinforced Concrete
21-30	2,334	17	88
31-40	3,623	32	180
41-50	2,952	45	61
51-60	3,159	79	57
61-70	2,353	328	16
71-80	1,556	348	13
81-90	1,231	28	8
91-100	886	49	2
101-110	621	26	3
111-120	644	0	1
121-130	420	3	2
131-140	295	3	0
141-150	177	1	6
151-160	94	1	1
161-170	34	0	1
171-180	44	0	1
181-190	9	0	0
191-200	18	0	1
>200	82	0	11
Totals	20,632	960	452

Figure 1. Voided-slab beams used in New York State (sections modified from Ref. 1).



(a) (b)

Slab Sections in (a) where Width W = 36 in.

Slab Depth D, in.	Area, in.^{2}	Moment of Inertia, in.^{4}	Distance from Center of Gravity To Bottom, in.
12	419	5,081	5.95
15	425	9,506	7.41
18	475	16,117	8.89
21	513	25,100	10.37

Slab Sections in (b) where Width W = 48 in.

Slab Depth D, in.	Area, in.^{2}	Moment of Inertia, in.^{4}	Distance from Center of Gravity To Bottom, in.
12	563	6,809	5.96
15	555	12,681	7.43
18	613	21,460	8.92
21	686	33,873	10.40

I. INTRODUCTION

Use of precast bridge superstructures has grown considerably in recent years, an increase in popularity that can be attributed to their fast, easy construction and relatively low cost (*1*,*2*,*3*,*4*). Precast bridges also reportedly provide 1) minimal maintenance, 2) simple design, 3) minimum span/depth ratios, 4) assured plant quality control, 5) durability, and 6) esthetic attractiveness (*1*). Typically, their construction proceeds by placing a number of precast members alongside each other, then tying them along their edges through grouted keyways and mechanical connectors for vertical shear transfer between adjacent units (*1*,*5*,*6*). End and intermediate diaphragms and transverse post-tensioning are sometimes used to resist lateral loads and maintain section geometry (*7*). Nationally, single-stem (I-beam and T-beam) and multi-stem precast members are widely used, respectively for “medium” spans ranging from 40 to 180 ft and “short” spans from 25 to 85 ft (*5*) -- 90 percent of highway bridges in this country have spans of less than 100-ft length (*2*). (Because both past New York State experience and information received from other states as a result of surveys for this study have generally involved customary rather than metric units, the former are used here; designers should convert to metric values with care.)] In this report, short spans are defined as those having lengths ranging from 25 to 60 ft. Under that definition, about half those in New York listed in Table 1 may be classified as short spans. This suggests the potential benefits for the New York State Department of Transportation (NYSDOT) from using this type of construction for bridges in this span range. Although improved bridge performance is generally expected from cast-in-place composite decks (*6*), adjacent precast units with or without wearing surfaces are sometimes preferred as an even faster construction alternative. This rapid, "fast-track" construction reduces downtime during construction and consequent inconvenience for the traveling public. Standard sections and details are proposed in this report for double-T beams for use with both asphalt wearing surfaces and cast-in-place composite concrete decks.

A. Precast Short-Span Bridges in New York State

Table 1 indicates that precast prestressed concrete bridges account for about 2 percent of short-span structures in New York. A few were built using single-T beams and bulb-T beams during the 1950s and 1960s. Precast voided slabs (rectangular slabs cast with cylindrical voids to reduce dead load) are currently specified as NYSDOT standard for precast short-span bridges on both local and state roads. Standard sections for those structures are shown in Figure 1, and their numbers in this state, grouped by ranges of span lengths, are as follows (this tabulation includes both prestressed and reinforced-concrete construction):

Span Length, ft	Total Voided-Slab Bridges
25-30	98
31-40	206
41-50	156
51-60	68
Total	528

Based on the past 10 years of the Department's bridge inventory data, it is estimated that 15 voided-slab bridges are built annually in New York, including superstrucutre-replacement projects. Precast double-T beams, the subject of this report, offer a potential alternative to voided slabs for use in the span range of 25 to 60 ft.

B. Objective and Study Approach

This reports's objective is to develop standard sections and details for precast prestressed double-T beams for use in short-span bridges, based on their practicality and economic advantages over voided slabs. To accomplish this task, a national survey was conducted to solicit information about these structures, including problems experienced and economic advantages over voided slabs, if any. Of 25 states responding, only 6 had used or are now using double-T beams and standard drawings showing details for such structures were requested from those states. Because double-T beams have never been used in New York, a second survey addressing similar issues was conducted within the state concerning single-T beams (in many respects, the structures most closely resembling double-T beams). Department bridge inventory records revealed a total of 70 existing single-T beam and bulb-T beam bridges. Responses from the two surveys and the bridge standards and plan sheets provided were carefully studied to identify merits and drawbacks of each detail. Development of proposed standard sections and details for double-T beam bridges was based on this evaluation and relevant studies, adhering to New York State and AASHTO standards (*7,8,9,10*). Economic advantages of the proposed systems were established on the basis of materials savings by direct comparison with current voided slabs, considering flexural design of bridges having various numbers of lanes, span lengths, and widths, and using HS-25 loading.

C. Organization of This Report

This report has five more chapters. In Chapter II, New York State and national experience with single- and double-T beams is discussed, and conclusions drawn about performance of details currently used. Standard beam sections and details are proposed in Chapter III for non-composite double-T beams used in rapid construction of bridges with asphalt wearing surfaces. Similar standard beams and details are proposed in Chapter IV for bridges with cast-in-place composite concrete decks. Economic advantages of the proposed beam sections and details as compares to voided slabs are discussed in Chapter V. The study is summarized and conclusions presented in Chapter VI.

Table 2. Summary of New York State experience with T-beam bridges.

NYSDOT Region*	Total Bridges and Types	Condition Rating: Primary Members	Condition Rating: Deck	Surface Type	Problems and Maintenance
2	1 Single-T	5	5	Originally 2.5" min, Now 3-5" hot-mix asphalt	Long. joint problems. no visible reflective crscking, leakage reduced by adding membrane, problem not unique to T-beams
4	2 Single-T	5	6	2.5" ACC min	None
5	2 Bulb-T	5	4	Originally 2.5" Type 2B ACC Now 2.5" ACC min over membrane	Minor long. joint cracking & leakage before adding membrane, overextended bearings, end-cap delamination, now relatively maintenance-free
7	17 Single-T, 6 Bulb-T	4-6	4-5	ACC	On 11, long. joint problems, cracking, leakage On 7, some maintenance problems
8	4 Single-T	5-7	4-6	ACC and monodeck	On 2, long. joint problems, cracking & leakage, joint failure led to wearing surface failure, problem not unique to T-beams
9	1 Inverted U-Section	4	6	Originally ACC, membrane added	Long. joint problems, at some locations members cracked & concrete spalled, some leakage

*Region 1 also has experience with T-beam bridges, but did not participate in this survey.

Table 3. Summary of national experience with double-T beam bridges.

State	Total Bridges	Dimensions: Web	Dimensions: Flange	Dimensions: Deck or Surface	Shear Key, Membrane	Longitudinal Joint	Problems and Costs
California	NA	7"	2"	8"**	Neither	None	Girder hogging, costs 40% less than slabs
Florida	5*	7"	7"	None	Shear keys	Post-tensioned	Cracking & leakage, cost-competitive with slabs, used on 40- to 60-ft slabs with low to moderate traffic
Minnesota	110	6"	7"	2-4" ACC	Shear keys & 12" membrane	Weld @ 4'	Cracking & leakage, voided slabs not used, double-T beam use based on span length & traffic volume
Missouri	Very few	--	--	--	--	--	Cracking & leakage, costs $40/sq ft vs $52 for slabs, used for spans under 50-ft length
Nebraska	173	--	2.5"	5" min**	--	Weld @ 4'	Believed the most economical beam, use based on span length & free board
Washington	8	5.25"	6"	2" ACC	Both	Weld @ 4'	Cracking & leakage, costs $40/sq ft vs $52 for slabs, used for spans under 50 ft

NA = not available
*On state roads; also 20 to 30 on local and private roads.
**Cast-in-place reinforced concrete.

II. EXPERIENCE WITH SINGLE- AND DOUBLE-T BEAMS

A. New York State Experience With T-Beams

NYSDOT has 70 T-beam bridges located in Regions 1, 2, 4, 5, 7, 8, and 9. Information about factors influencing or indicating problems was requested in a survey of these seven regions. Specific information requested included T-beam type, condition ratings of primary members and decks, surface types, and problems with longitudinal joints and maintenance. Survey responses are summarized in Table 2. Some observed problems with these bridges included cracking at longitudinal shear-key joints, leakage through such cracks, and resulting damage to the superstructure and substructure. This damage is manifested in 1) longitudinal cracking of the deck at these locations, 2) staining and efflorescence on the bridge underside at shear keys, and 3) concrete cracking, spalling, and delamination around the lower part of the web. These lead to exposure and corrosion of prestressing strands and reinforcing steel, and subsequent weakening of the bridge structure.

B. National Experience With Double-T Beams

A national survey concerning double-T beam bridges was conducted to gather information about beam dimensions (web and flange), deck types, use of shear keys and impermeable waterproofing membranes, existence of longitudinal-joint cracking and leakage problems, and cost comparison with voided slabs. Responses from six states now using these structures are compiled in Table 3.

C. Evaluation of State and National Experience

From the surveys summarized in Tables 2 and 3, the following observations are drawn:

1. A majority of single-T and bulb-T beam bridges in New York have experienced reflective cracking and leakage problems at longitudinal joints.

2. Bridges with asphalt wearing surfaces have suffered cracking and leakage problems, but none were reported for bridges with cast-in-place composite concrete decks.

3. For bridges with asphalt wearing surfaces, impermeable waterproofing membranes helped reduce leakage and subsequent superstructure deterioration problems.

4. Double-T beams are used mainly in bridges on roads having low-to-medium traffic volumes, with additional limitations on bridge skew and span length. An exception is Nebraska, which uses cast-in-place decks on shallow-flanged double-T beams, regardless of traffic volume, for longer spans.

5. Web width of double-T beams ranges from 5.5 to 7 in. Flange thickness varies depending on whether an asphalt wearing surface or a cast-in-place deck is used. With an asphalt wearing surface, flange thicknesses range from 6 to 7 in., and with a cast-in-place deck, from 2 to 2.5 in.

6. Double-T beams are more economical or similar in cost to voided slabs.

7. Only Florida uses a transversely post-tensioned system, with a 1/2-in. concrete wearing surface cast monolithically with the beams.

The following conclusions may be drawn from these observations:

1. Some states have successfully used double-T beams, which are much more stable and have fewer longitudinal joints than single-T beams, for short-span bridges. There is also general agreement among the responding states that double-T beams are economically competitive with voided-slab sections.

2. Two sets of details for double-T beams could be developed for use with a) an asphalt wearing surface for rapid construction, and b) a cast-in-place deck for improved durability.

3. Double-T beams can be used on short- to medium-span bridges with low traffic volumes and less than 30° skew angles.

4. Impermeable waterproofing membranes can be used with asphalt wearing surfaces to minimize leakage problems.

5. When using asphalt wearing surfaces, minimum flange and web dimensions can be 6 and 7 in., respectively. Similar dimensions for a cast-in-place deck can be 2.5 and 6 in., respectively.

6. Precast double-T units used with asphalt wearing surfaces can be connected through shear keys and welding of adjacent members at fixed intervals, or by post-tensioning of the deck and diaphragms.

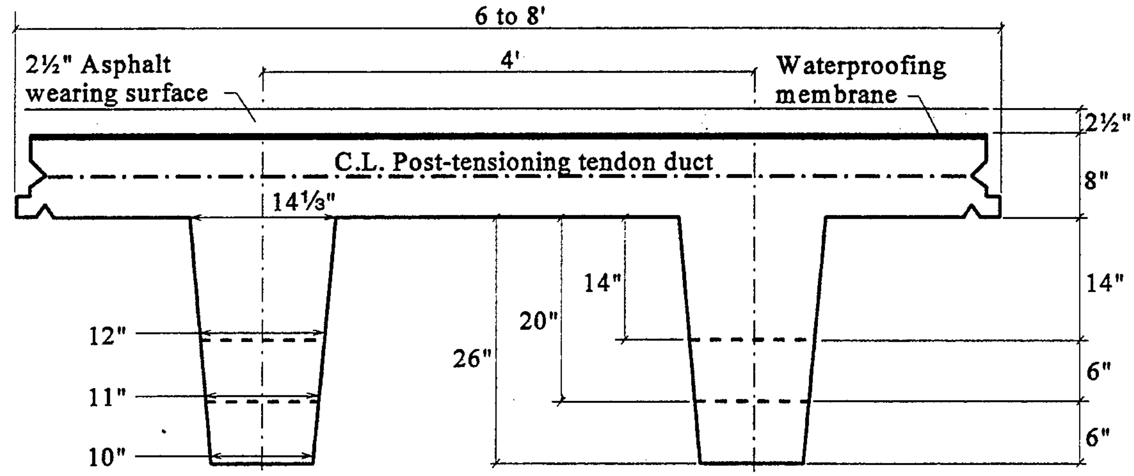
III. PROPOSED STANDARD SECTIONS AND DETAILS FOR USE WITH ASPHALT WEARING SURFACES

Using an asphalt wearing surface as an alternative to a cast-in-place deck has been estimated to save about 14 days of downtime during construction, significantly reducing inconvenience to the traveling public. Details for double-T beams used with an asphalt wearing surface proposed here for non-composite, rapid construction include 1) beam sections and details for longitudinal shear keys, 2) transverse post-tensioning of the deck, 3) integral precast diaphragms, 4) impermeable waterproofing membranes, and 5) fixed and expansion supports. All beams here are assumed to bear equally on two elastomeric pads located at the beam ends on bridges having less than 30° skew (the maximum skew angle specified by Florida and Washington for such structures). Beams also should be designed to carry an additional 15 psf of future surfacing. Prestressing strands should be arranged symmetrically about the web axis, and may have straight and harped profiles with cover of at least 2 in. to the center of the lowest row of strands. Requirements for debonding of prestressing strands should be those given in References 8 and 11. Beam ultimate shear strength should be checked considering the reduction in shear capacity due to debonding, and may be enhanced by locating some strands near the beam's neutral axis (*12,13,14,15,16*).

A. Proposed Double-T Sections

Three types of double-T beams (Types DT-14N, -20N, and -26N, where "N" is for non-composite, as distinguished from the composite sections discussed in the next chapter) are proposed here for use in bridges having non-composite wearing surfaces. Sections of these beams are shown in Figure 2, with dimensions selected to meet the minimum requirements discussed in the previous chapter. Web and flange dimensions and recommended maximum span lengths for each type are also given in the table displayed in this figure. Note that all beams have similar web tapering, flange thickness, and flange width, these latter widths varying from 6 to 8 ft with a 2-ft maximum limitation on beam overhang (measured from the centerline of the nearest web). This is a great advantage because fabricators can use a single casting bed to cast all three types. Also, beam sections have been optimized to maximize structural efficiency while minimizing beam depth, which is beneficial in limited freeboard situations. The proposed web dimensions allow placement of more than two prestressing strands per row, and strands thus can be concentrated in small areas of the webs. This is not only an efficient method of prestressing, but also eliminates the need for deeper webs.

Figure 2. Standard double-T beam for use with an asphalt wearing surface.



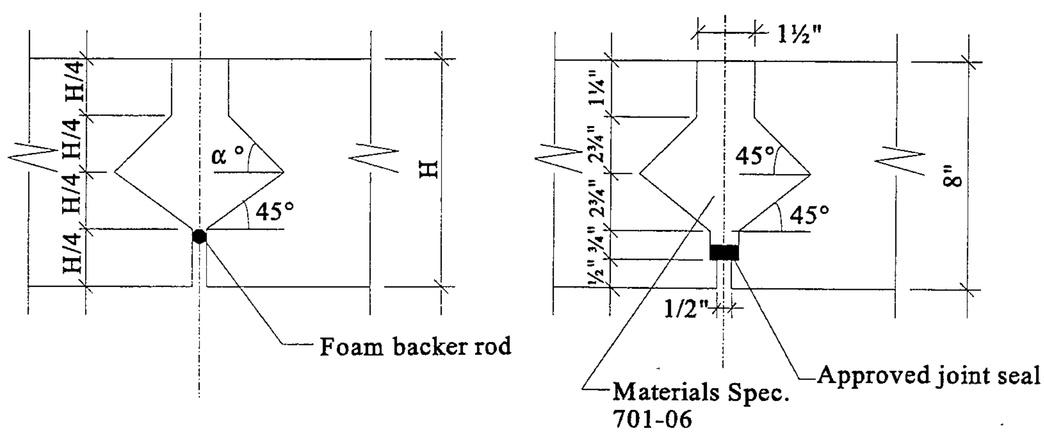
Beam Type	Width: Web, in. Top	Width: Web, in. Bottom	Width: Flange, ft	Depth, in. Web	Depth, in. Flange	Depth, in. Total*	Max Span, ft
DT-14N	14⅓	12	6-8	14	8	22	40
DT-20N	14⅓	11	6-8	20	8	28	50
DT-26N	14⅓	10	6-8	26	8	34	60

*Concrete only.

Figure 3. Longitudinal shear key.

A. SUGGESTED IN REF. 5

B. RECOMMENDED FOR NYSDOT



Further, the proposed webs are much stiffer than those specified for standard PCI and AASHTO double-T sections (*1*), improving their resistance to impact by debris and ice in stream crossings. Figure 2 also shows the recommended 2-1/2-in. asphalt-concrete wearing surface over an impermeable sheet membrane. If a floor pipe is used for drainage, its location and esthetic requirements should meet AASHTO and New York specifications (*7,8,9*). New York State guiderail (BDD 83-72) or any other system approved by the Department may be used. The proposed section includes two 1/2-in. continuous grooves to guard further against malfunctioning of the waterproofing system. Shear-key strength will not be adversely affected by presence of these grooves, because they fall inside the key's failure wedge. The duct for transverse post-tensioning tendons (as will be discussed later) is not shown in this figure.

The proposed cross-section of tapered stems permits placing the prefabricated reinforcement cage and removing the beam from the casting bed without moving the forms, a fabrication advantage not provided by voided-slab sections (*12*). Flanges also can be cast monolithically with the stems due to the inherent stability of this section.

B. Longitudinal Shear Keys

The primary purpose of these keys is to transfer vertical wheel loads to adjoining members, but they should also prevent moisture leakage through the deck. They are used in combination with mechanical connectors, which carry in-plane tension forces and also tie the structure together. Shear keys and connection design for precast multi-beam bridges have been investigated by Martin and Osborn (*6*) and by Stanton and Mattock (*5*). Martin and Osborn indicated the importance of a positive transverse-tie system, and concluded that a properly grouted shear key and transverse tie-rods/welded connectors effectively transfer shear between adjoining precast units. Based on analytical investigation and testing of various types of shear keys to failure, Stanton and Mattock recommend the following for design of grouted shear keys:

1. Flange thickness of precast members should be 6 in. or $6\sqrt{5000/f'_c}$ in inches (where f'_c is compressive strength of the beam in pounds per square inch), whichever is greater.

2. The shear key should be shaped as shown in Figure 3A. (Greatest strength will develop when the key's maximum width is at mid-depth of the deck slab. Grouted shear-key strength is governed by inclined cracking of the tips of the connected flanges along the inclined sides of the key, even when the concrete is 75-percent stronger than the grout.)

3. Additional requirements are given for connectors between adjacent units.

The first recommendation is met by using the 8-in. flange thickness just proposed in the preceding section of this chapter and $f'_c \geq 5000$ psi. For the second recommendation, the shear-key shape suggested by Stanton and Mattock is slightly modified to provide additional area for a larger joint seal, as shown in Figure 3B. The rest of the recommendations are addressed in the next section, when discussing transverse post-tensioning. The non-shrink grout material used in the shear key should conform to NYSDOT specifications for this material (*8,9,11*).

C. Transverse Post-Tensioning of the Deck

1. The Proposed System

In-plane forces at longitudinal shear keys may be caused by transverse bending moments, shrinkage, and temperature change. These moments tend to close a shear key at the top, but under traffic loading, "rebound" and fatigue could also cause tension and cracking at the top. This explains the common practice of placing mechanical beam connectors at mid-depth, where the stress-reversal effect is minimal. Shrinkage and temperature change cause members to decrease in width and separate at keyways. Martin and Osborn (*6*) concluded that without a positive tie force, cracking may occur at keyways or grout may separate from the member, but that the likelihood of shear-key failure is substantially reduced by preventing crack opening. They also ranked epoxy-grouted keys with post-tensioned ties as better than all other methods used to connect precast members, and estimated the total design tie force required to hold the beams together at half the weight of the bridge (assuming a coefficient of friction of 1 between the deck member and its support). Although adjacent precast members are commonly welded together at fixed intervals (*5,6*), as in Minnesota, Nebraska, and Washington (Table 3), transverse post-tensioning of the deck through the flanges is recommended here for four reasons:

1. By maintaining adequate compression perpendicular to the joint, performance of the longitudinal shear key in transmitting shear forces between adjacent beams is greatly enhanced (*5,6*), as previously discussed.

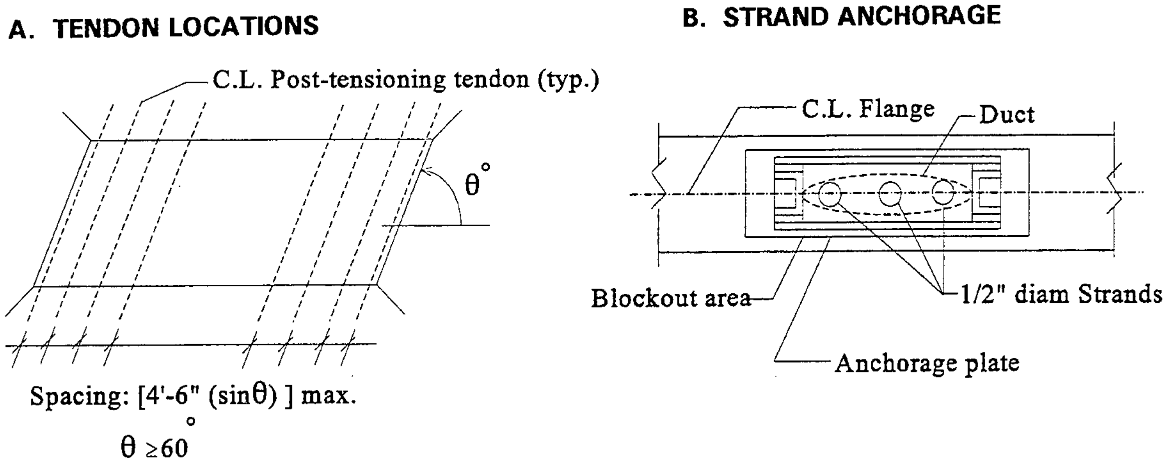
2. It promotes development of in-plane membrane forces (*12,13,14,15,16*), generally enhancing load-carrying capacity (*17,18,19*). Florida uses double-T beams post-tensioned through the flanges in bridges without an asphalt wearing surface or a cast-in-place deck (a 7-in. flange provides 1/2 in. for a grinding surface to improve riding quality). Various studies of the Florida system have concluded that post-tensioning provides for monolithic deck behavior by means of confinement, and that the deck develops in-plane forces similar to cast-in-place construction (*12,13,14,15,16*).

3. It limits longitudinal cracking (*5,6,12,13,14,15,16*), and thus reduces leakage and subsequent superstructure deterioration (*5,6*).

4. It is also viewed as inexpensive and requiring no additional time for installation, thus not affecting total construction time.

Moreover, comparison between mechanical connectors (such as welding) and transverse post-tensioning (*13*) indicates that for the connecters, 1) the live-load distribution factor may be 25-

Figure 4. Post-tensioning tendon locations and strand anchorage detail.



percent higher in some beams, 2) beam flanges may be subjected to considerable stress reversals, 3) absolute and differential deflections are also 25-percent higher, and 4) mechanical connectors may be exposed to concentrated rotations, which may lead to fatigue failure (*20*). The only disadvantage of transverse post-tensioning is an increase in design moment of the deck slab (*13*).

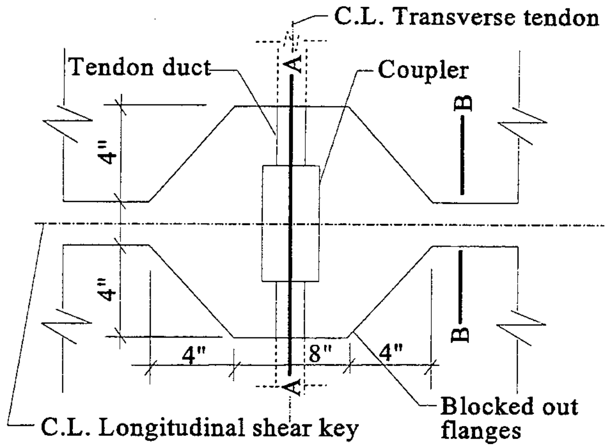
2. *Major Components of the Proposed System*

Locations and major components of the transverse post-tensioning system proposed here are as follows:

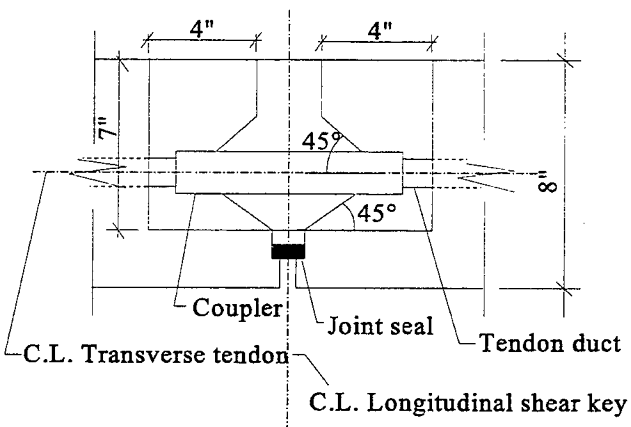
1. Post-tensioning units should be located at a maximum spacing on the deck 4-ft 6-in. x sin θ, where θ is the angle defined in Figure 4A. This may be compared with the maximum spacing for welded connectors, which is recommended not to exceed 5 ft or the flange width of connected members (*5*), whichever is less. Each post-tensioning unit should consist of three 1/2-in. diam, 270 -ksi, low-relaxation, seven-wire strands located side-by-side in the middle third of the deck slab, as shown in Figure 4B, resulting in about 200-psi post-tensioning stress in the deck slab after all losses are accounted for -- 150 psi was proposed initially in Florida studies (*12,13,14,15*), but increased to about 200 psi when Florida's double-T standards were developed (*16*).

2. Couplers fabricated from high-density polyethylene should be installed in the beams at all tendon crossing points at longitudinal shear keys, and should be securely wrapped with duct tape to prevent grout material from entering the post-tensioning ducts. Details for couplers and beam blockouts (similar to those used by Florida) are shown in Figure 5. Shear-key grout material should be used to fill beam blockouts after post-tensioning is completed.

Figure 5. Post-tensioning couplers at the shear key for a 0° skew bridge.



SECTION A-A



SECTION B-B

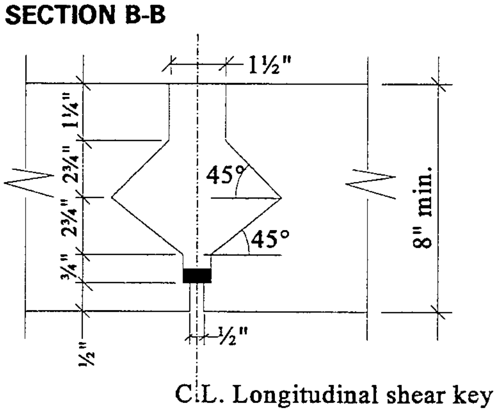
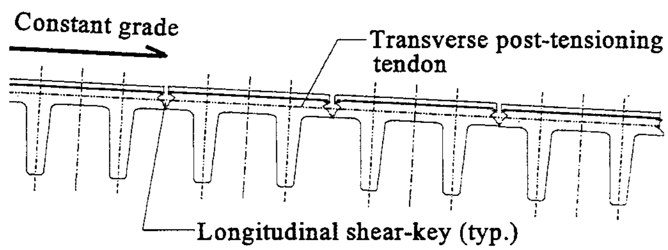
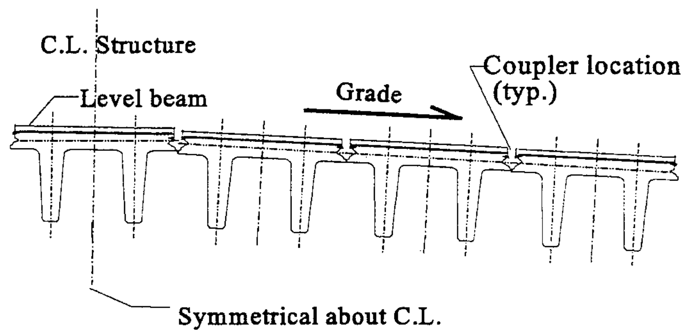


Figure 6. Three common transverse deck profiles.

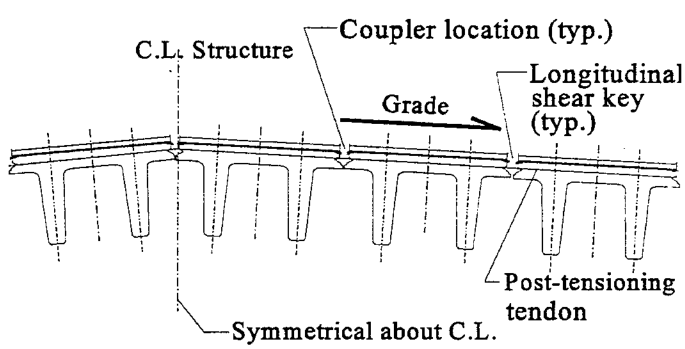
A. BRIDGE WITH CONSTANT TRANSVERSE GRADE



B. CROWN BRIDGE WITH ODD NUMBER OF BEAMS



C. CROWN BRIDGE WITH EVEN NUMBER OF BEAMS



3. Special reinforcement should be installed at tendon deviation points to strengthen these locations to withstand the force component due to post-tensioning, if this is deemed to be intolerable. Locations for this special reinforcement are determined based on transverse deck profile. Three common profiles are constant grade (Figure 6A), crown with an odd number of beams (Figure 6B), and crown with an even number of beams (Figure 6C). For a bridge with constant transverse grade, no special treatment is needed due to absence of the force component, because tendons can be post-tensioned in straight profiles without touching duct walls. Special reinforcement should be installed at both sides of the crown beam on a crowned bridge having an odd number of beams. For a crowned bridge with an even number of beams, special reinforcement should be installed at the sides of the beams meeting at the crown. Special reinforcement of the flanges of exterior beams is required in all cases at tendon-anchorage locations.

3. Strength of the Proposed System

This strength can be estimated relative to systems without post-tensioning, and from punching shear tests on the Florida double-T system (*12,13,14,15,16*). Stanton and Mattock (*5*) estimated cracking strength of the flange near the keyway in a post-tensioned system to increase by a factor K_p times that without post-tensioning, where

$$K_p = (1 + 10\, f_{pc}/f'_c)^{0.5}$$

f'_c is compressive strength of the deck slab and f_{pc} is average prestress in the flange, both in pounds per square inch. For an example of estimating strength of the proposed system using this equation, consider a bridge having the proposed shear key, post-tensioned with three 1/2-in. diam, 270-ksi, low-relaxation prestressing strands, located at 4 ft 6 in. center-to-center. Compressive strength of the deck slab f'_c is 5000 psi. Average prestress in the flange f_{pc} can be calculated as 217 psi, assuming 33 ksi for losses (*7*), and thus $K_p = 1.20$. Shear-key capacity of the proposed system consequently is 20-percent higher than a similar system without post-tensioning.

Florida tested a half-scale model of a 60-ft long, 24-ft wide, transversely post-tensioned double-T bridge with 6.5-in. flange, 7.5-in. web, and 2-ft overhangs during their research leading to development of their double-T standards (*12,13,14,15,16*). This model consisted of three beams with V-shaped shear keys, post-tensioned transversely to a stress of 150 psi at the middle portion of the bridge, and 300 psi over a distance of 3 ft at both ends. Flexure and shear tests were conducted at both internal and end-zone locations. Load was applied at the joint (for flexure), near both sides of three other joints (for shear), and between the stems in a seventh test (used as a benchmark for jointless slabs). Crack sizes and deflections were recorded in each test. Flexural testing at the end zone was continued to the ultimate, and failure occurred in a typical punching mode at 69.1 kips (corresponding to 276.4 kips in the full-size prototype). Shear testing resulted in first cracking at 15 kips (corresponding to 60 kips in the prototype) and the maximum observed crack width at twice AASHTO's service load was 0.001 in., which is much lower than AASHTO and ACI limits. There was no evidence of relative joint movement due to shear in any of these tests. Absence of cracks at stress levels at the joint exceeding those required to cause cracking was attributed to presence of compressive membrane forces in the deck slab. Additional testing showed that the bridge system satisfied requirements for load-carrying capacity, ductility, spacing and size of flexural cracks, deflection, and fatigue.

In summary, adequate service-load performance and ultimate-load behavior should be expected for the proposed system, which is estimated to be 20-percent stronger than similar systems without post-tensioning. Based on Florida's testing, adequate service-load performance as well as ultimate-load behavior also should be expected, noting the similarity between the two systems.

D. Diaphragms

Discussing load distribution in stemmed multi-beam bridges, Stanton and Mattock (*5*) emphasized the importance of end diaphragms to ensure proper load distribution, and also that these should be deep and rigid, especially for beams resting on elastomeric bearings. Presence of intermediate diaphragms reduces the load carried by inner members and increases the load for outer members. If used, they should be as deep and rigid as possible, with full moment connection at joints between precast units. They also concluded that steel-truss diaphragms are considerably less effective than cast-in-place full-depth concrete diaphragms. AASHTO (*7*) specifies that diaphragms should be used at ends of bridge spans unless other means are provided to resist lateral forces and maintain section geometry. For T-beam construction, one intermediate diaphragm is recommended at the point of maximum positive moment for spans exceeding 40-ft length (*5*).

Proposed spacings for intermediate diaphragms are very similar to current NYSDOT standards for box-beam and slab-beam bridges (*8,21*). No intermediate diaphragms are required for spans shorter than 30 ft, but one is needed at half-point for spans of 30- to 50-ft length and at quarter-points for those exceeding 50 ft, as shown in Figure 7.

Use of full-depth end diaphragms decreases bearing stresses that could lead to premature failure of beam webs at the supports (*16,22,23*), and also provides an additional area for placement of steel dowels. Diaphragm units may be cast separately or integrally with the beams. Details for an integral diaphragm are shown for end and intermediate locations in Figures 8 and 9, respectively. Integral diaphragms precast with the beams should include embedded steel plates for later connection of adjacent units at longitudinal shear-key locations. Connecting plates with slotted holes may be bolted to adjacent diaphragms after completing post-tensioning. For diaphragms cast separately, precast-beam units may be cast with embedded steel plates to allow either welding or bolting to beam webs. Precast-diaphragm units should also be cast with embedded 2-in. diam ducts for passage of post-tensioning tendons. These tendons should be located at the centroid of the diaphragm section, post-tensioned at 35 kips (minimum force) and welded after post-tensioning is completed. This force is equivalent to the additional post-tensioning applied at ends of beams in the Florida double-T system, used as a substitute for end diaphragms.

Figure 7. Locations of precast diaphragms and tendons.

SPAN LENGTH L <50 FT **SPAN LENGTH L >50 FT**

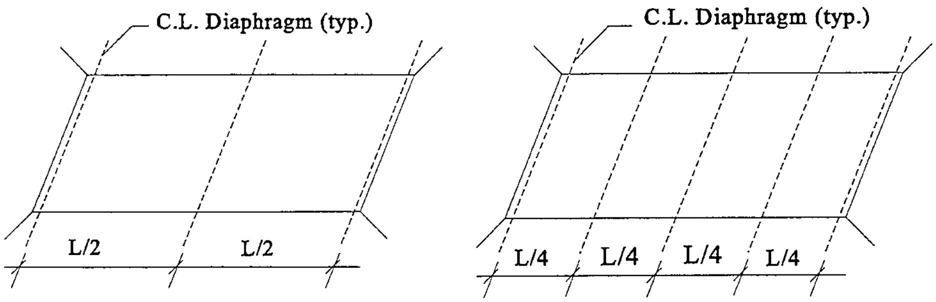


Figure 8. End diaphragm.

SECTION A-A

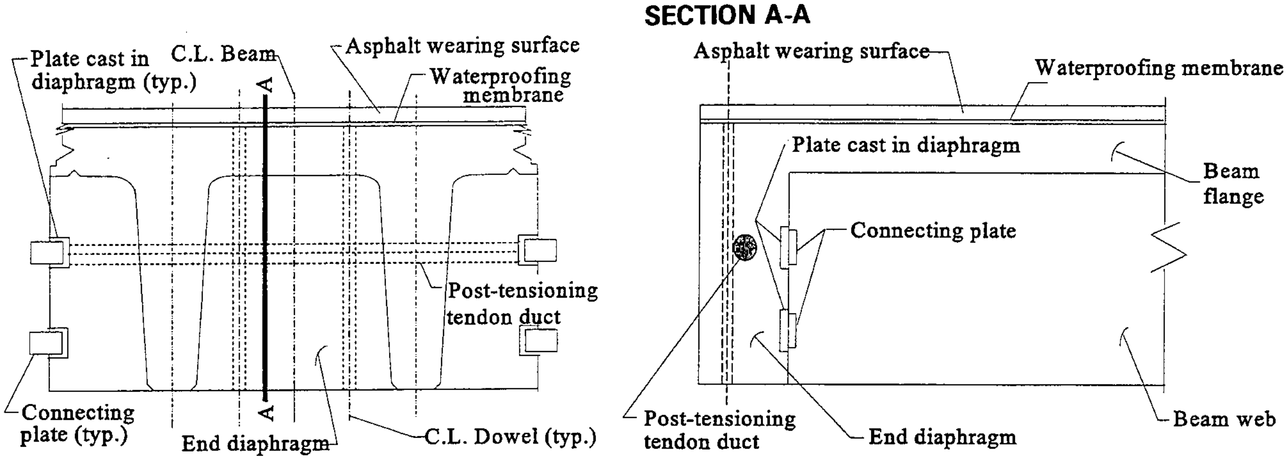
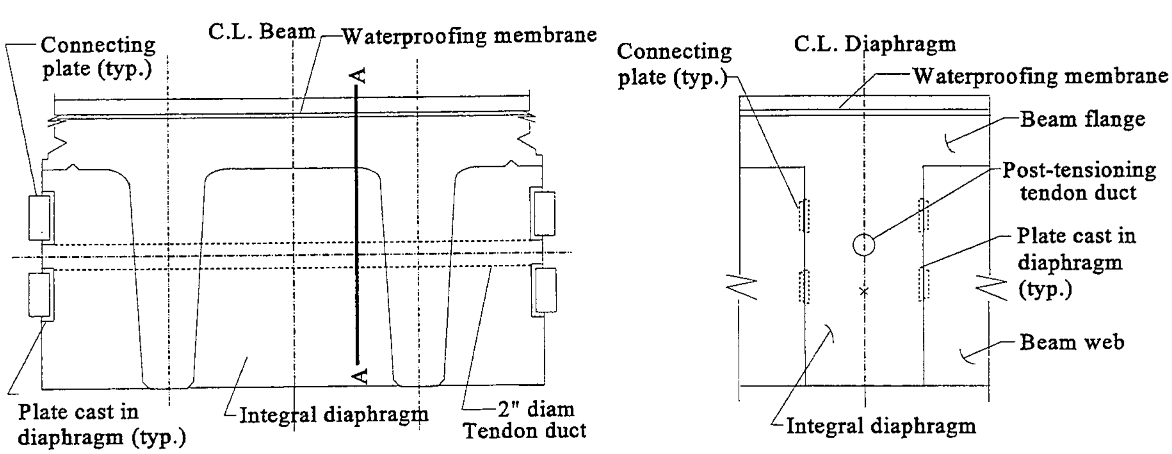


Figure 9. Intermediate diaphragm.

SECTION A-A



E. Impermeable Waterproofing Membranes

These have been used in New York for deck waterproofing for some time. New York began a long-term study of effectiveness of five membrane systems in 1977 after they had been in service for 1 year (*24,25,26,27*) -- three preformed (heavy-duty bituthene, bituminous-epoxy, and Royston No. 10 bridge membrane) and two liquid (bituminous-epoxy and NEA-4000LT). Each was applied to a deck and overlaid with a 2-1/2-in. thick asphalt-concrete wearing surface, using 3/8-in. maximum-size aggregate in the mix to minimize damage to the sheets. The criteria for evaluation were based on visual observations, electrical resistance, and corrosion potential. Final evaluation of these systems was completed in 1989 after 13 years of service (*27*), producing the following recommendations:

1. Four materials should continue to be used for deck waterproofing: heavy-duty bituthene, Protecto Wrap M-400A, Royston No. 10 bridge membrane, and NEA-4000LT membrane.

2. Two-coat bituminous-epoxy membranes should not be used.

3. Cracks forming along longitudinal paving joints should be sealed to prevent premature failure of membrane systems.

Further, based on various consultations and recent visits to bridge sites in New York where these membranes have been in place for more than 5 years without visible leakage problems, they can be expected to function adequately for at least the lifetime of the wearing surface. In addition, the proposed transverse post-tensioning of the deck should ensure monolithic behavior, thus minimizing damage to the membrane from excessive relative movement of adjacent beams (as just noted in Recommendation 3). Extended service life of the entire deck may thus be expected. Based on these observations, the recommended impermeable waterproofing membranes (or equivalent) are proposed to cover the entire deck, with special precautions at drainage locations. Deck preparation, surface treatment, and membrane application should meet NYSDOT requirements in this report's References 8, 9, and 28. Proper surface treatment and membrane installation were shown to contribute significantly in extending their service life (*29*). Figure 2 shows a section through the beam, including the membrane and wearing surface.

F. Fixed and Expansion Supports

Details for these supports at piers are shown in Figures 10 and 11. Beam fixity to the pier is achieved by using two 1-in. diam steel dowels partially embedded in the pier caps (1-ft minimum) and extending to the flange soffits as shown in Figure 10A (one dowel per web at the beam end), and by filling the dowel holes with non-shrink grout to restrict superstructure movement. Beams from adjacent spans (shown in Fig. 10B) should be welded to create live-load continuity over bridge supports. Transverse joints should be filled with an approved sealing material (*9,10*). A similar dowel system is proposed at expansion supports, with the following modifications to allow movement of the superstructure:

Figure 10. Fixed support at pier.

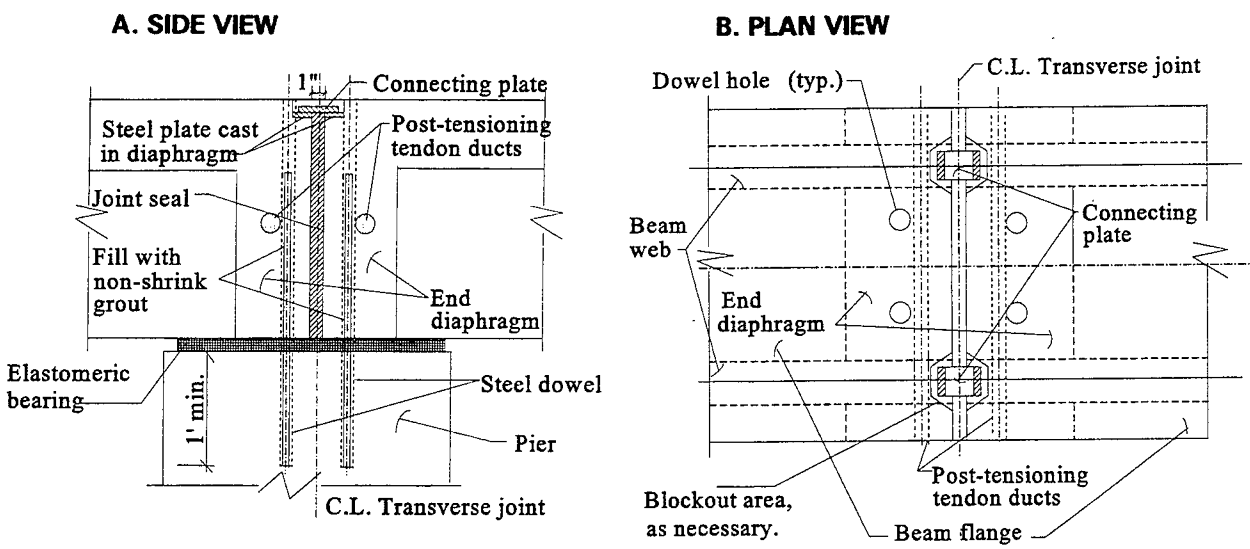
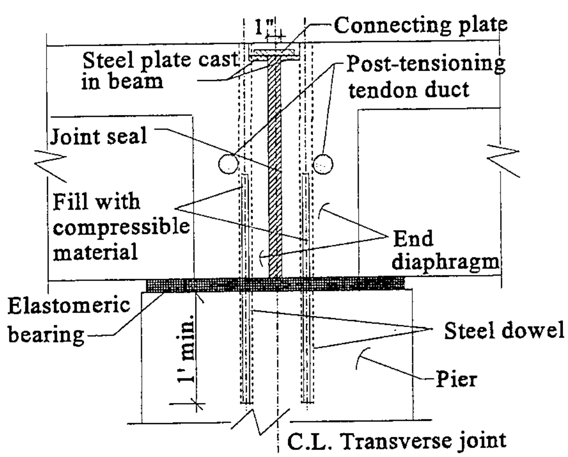


Figure 11. Expansion support at pier.



1. Larger dowel holes to provide adequate space for free movement.

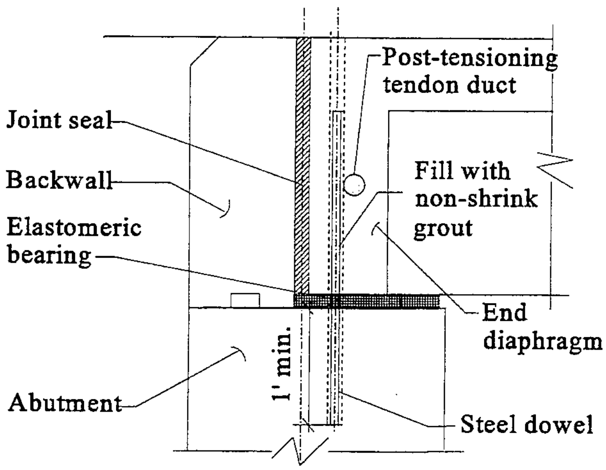
2. Shorter dowels extending to only half the depth of the beam.

3. Compressible material to fill the dowel holes.

Figure 11 shows a section at an expansion support including these details. Proposed details for fixed and expansion support conditions at the abutment, similar to corresponding details at the pier, are shown in Figures 12 for a backwall abutment. For abutments without backwalls, similar details are shown in Figure 13.

Figure 12. Supports at abutment with backwall.

A. FIXED SUPPORT



B. EXPANSION SUPPORT

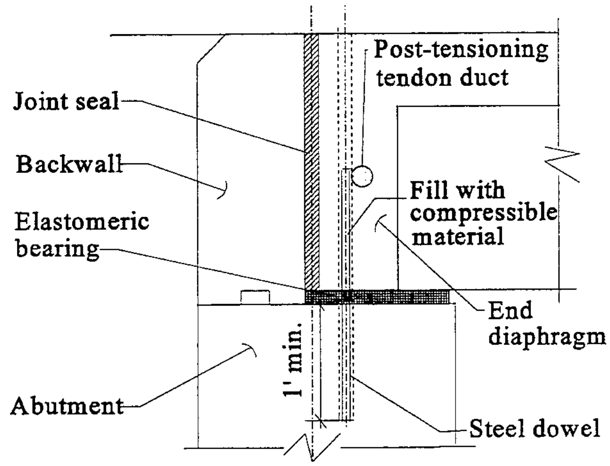
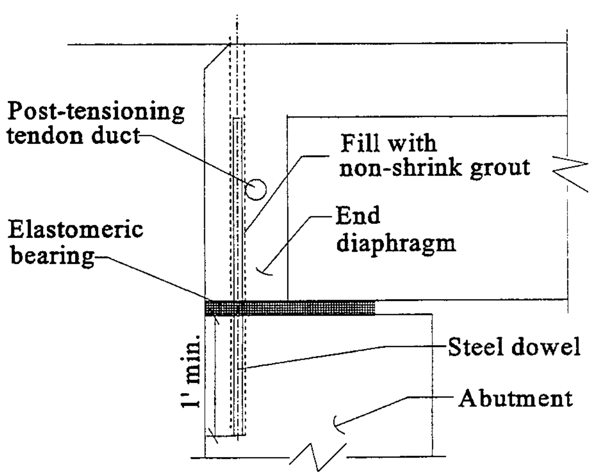
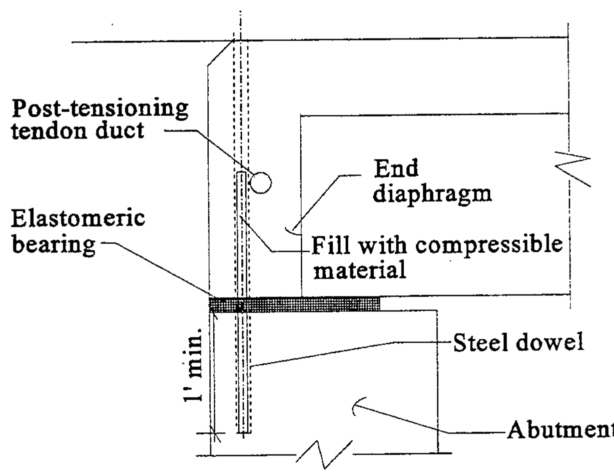


Figure 13. Supports at abutment without backwall.

A. FIXED SUPPORT



B. EXPANSION SUPPORT



G. Notes on Fabrication and Construction

1. Fabrication

1. Concrete for precast sections should have compressive strength of at least 4000 psi at transfer of prestress and 5000 psi at 28 days. Concrete air content should be 7 ± 2 percent (*7,8,11*).

2. All prestressing steel (including that for transverse post-tensioning) should be 1/2-in. diam, 270-ksi, low-relaxation, seven-wire strands.

3. Design, fabrication, and construction should be in accord with AASHTO and New York State standards (*7,8,11*)

4. Dimensional tolerances of precast sections should be in accord with NYSDOT standards (*10*) and the PCI *Manual for Quality Control of Plants and Production of Precast Prestressed Concrete Products* (*30*).

2. Construction

The following installation sequence should be followed while erecting the beams:

1. Outline theoretical pad locations on the bridge seats for the centermost beam, adjust these locations (if necessary) to center the pads under the beam ends, place the beam, and determine the need for shims or grinding as appropriate.

2. Check beam position and alignment after its placement before proceeding with erection of the next beam. Temporary loads can be applied to eliminate differential camber, and removed after transverse po-t tensioning is completed. Repeat Step 1 for each beam, and check as necessary before proceeding to the next beam.

3. After all beams are erected, a final check is required before any additional work. A similar sequence should be followed in erecting beams for multi-span bridges.

4. Insert backer rods in keyways between beams.

5. Join transverse post-tensioning ducts and securely wrap couplers at joints with duct tape to protect them during keyway grouting. Post-tensioning strands should not be installed at this time.

6. Prepare longitudinal keyway surfaces, place plugs, and fill shear keys with an approved non-metallic, non-shrink grout material. Total curing period for the grout should be as specified by the manufacturer.

7. Install tendons in the ducts (each tendon should consist of three 1/2-in. diam, 270-ksi, low-relaxation, seven-wire strands). Apply transverse post-tensioning after the grout has cured (at least 48 hr) and reached a minimum strength of 400 psi. Post-tension the midspan or centermost tendon first. Proceed by post-tensioning every other tendon on either side of the span in a sequence that keeps eccentricity about midspan to one tendon. Grouted keyways should not be disturbed by heavy construction activities before completion of post-tensioning and total curing of grout.

8. For multispan bridges, these steps for transverse post-tensioning should be executed alternating between spans, so as to avoid differences in transverse stress between any two neighboring spans.

9. Install all precast diaphragms, if used, and post-tension them in a sequence similar to that just described in Step 8. Connect adjacent units using bolted slotted plates.

10. Fill dowel holes with non-metallic, non-shrink grout or compressible material as shown for specific dowel locations on the plan sheets. Allow time for grout to cure as recommended by the manufacturer before starting other activities requiring operation of heavy equipment.

11. Prepare the deck surface, apply the waterproofing membrane, and pave the deck and approaches.

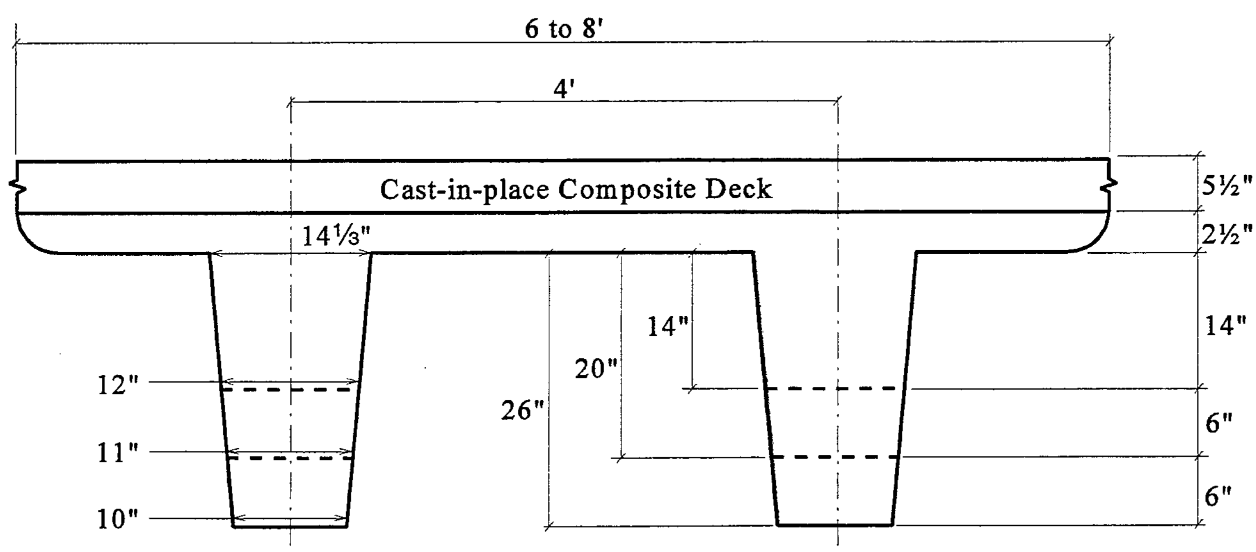
12. Install guiderails.

IV. PROPOSED STANDARD SECTIONS AND DETAILS FOR USE WITH CAST-IN-PLACE COMPOSITE DECKS

A. Proposed Double-T Sections

Three beam types are proposed for use in bridges having cast-in-place composite decks: DT-14, -20, and -26. Requirements for web dimensions and overhang for these beams are similar to those proposed in Figure 2 for use with an asphalt wearing surface -- this is an advantage in fabrication, because both systems can be cast using the same casting bed. Proposed flange thickness is 2-1/2 in. to maintain beam structural integrity during fabrication, transportation, and erection, and also to serve as a form for casting the composite concrete deck. Note that flange thickness has been increased over that used by California, to minimize the girder hogging problem reported in Table 3. Flanges should also be cast with circular access openings for pouring concrete for the cast-in-place diaphragms. A 5-1/2-in. thick deck slab (5-in. effective thickness) should be cast-in-place composite with the beams. Figure 14 shows proposed beam sections and suggested maximum span length for each.

Figure 14. Standard double-T section with cast-in-place composite deck.



Beam Type	Width: Web, in. Top	Bottom	Flange, ft	Depth, in. Web	Flange	Total	Max Span, ft
DT-14	14-1/3	12	6-8	14	2-1/2	22	40
DT-20	14-1/3	11	6-8	20	2-1/2	28	50
DT-26	14-1/3	10	6-8	26	2-1/2	34	60

Figure 15. Locations of cast-in-place diaphragms.

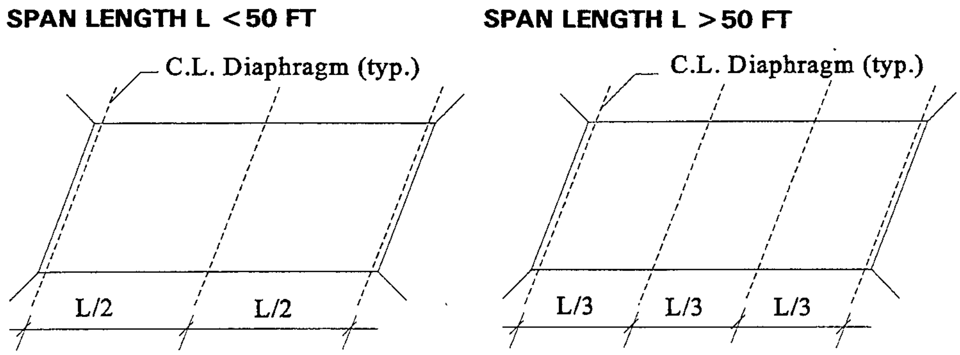


Figure 16. End diaphragm.

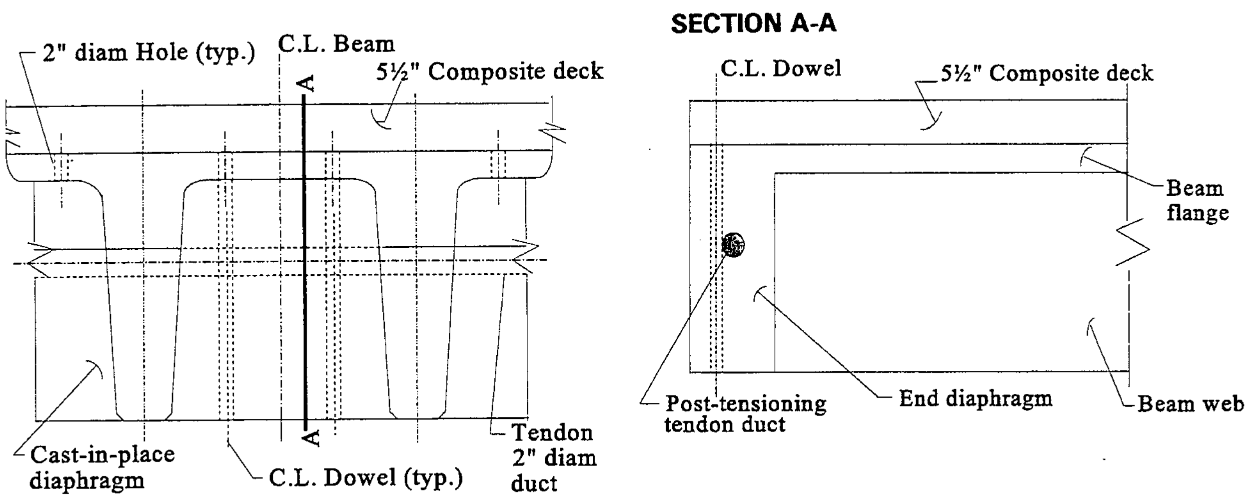


Figure 17. Intermediate diaphragm.

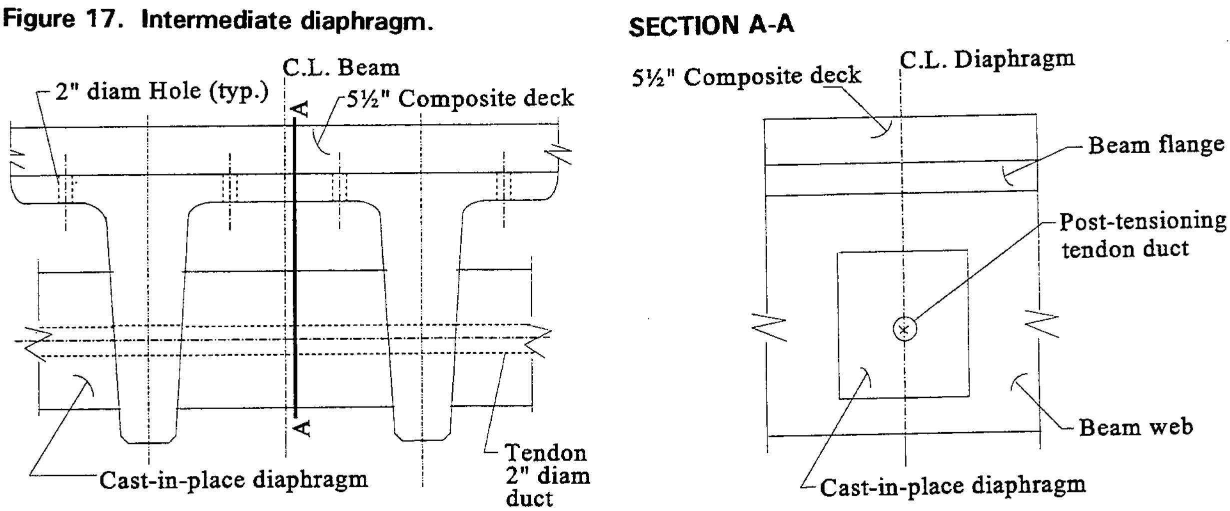
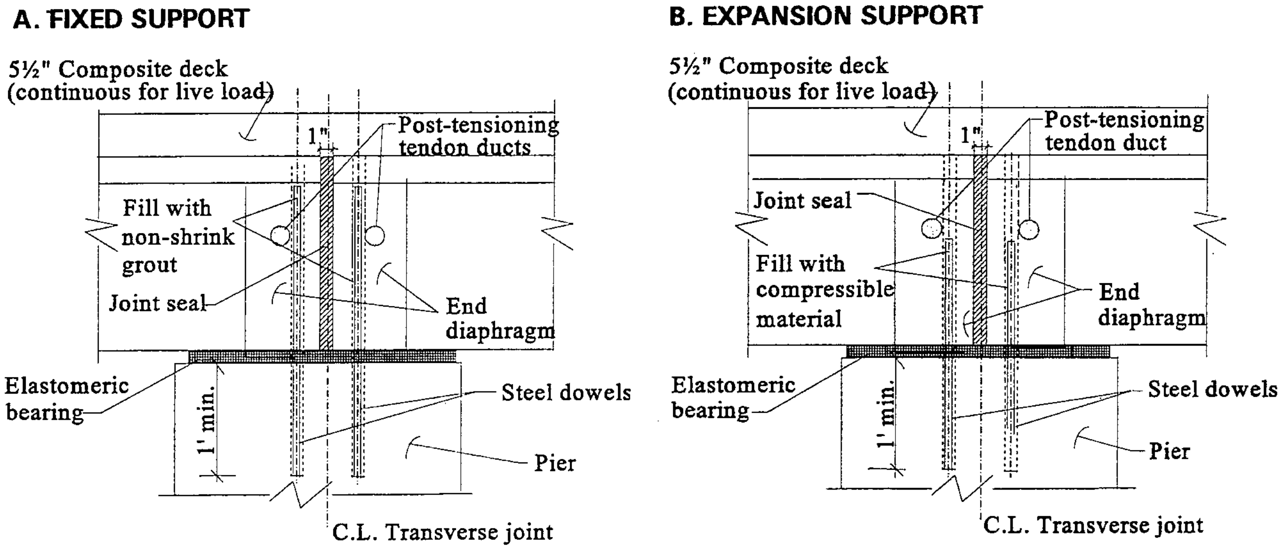


Figure 18. Support details at pier.



B. Cast-in-Place Diaphragms

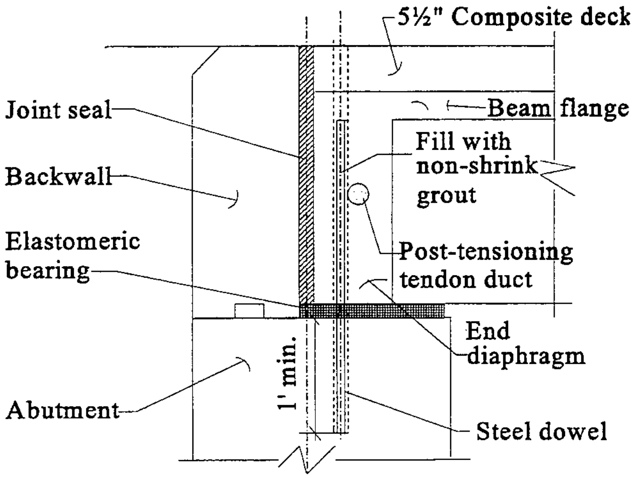
Recommended diaphragm locations are shown in Figure 15. Their spacing for spans longer than 50 ft has been increased over that for beams used with an asphalt wearing surface, because a cast-in-place composite concrete deck provides better lateral load distribution and improves monolithic behavior. Sections through end and intermediate diaphragms are shown in Figures 16 and 17, respectively.

C. Fixed and Expansion Supports

Details for these supports at the pier are shown in Figure 18. Beam fixity to the pier is accomplished in a manner similar to that described earlier for beams proposed for use with an asphalt wearing surface. The cast-in-place composite deck should be continuous over bridge supports, and designed for the specified live load. A system of sole plates and dowels similar to that proposed for beams to be used with an asphalt wearing surface is shown in this figure. Full-depth diaphragms should be cast with open ducts around the dowels to allow for free movement of the expansion end. Similar details are shown for fixed and expansion conditions at an abutment with backwall in Figure 19, and for an abutment without backwall in Figure 20.

Figure 19. Supports at abutment with backwall.

A. FIXED SUPPORT



B. EXPANSION SUPPORT

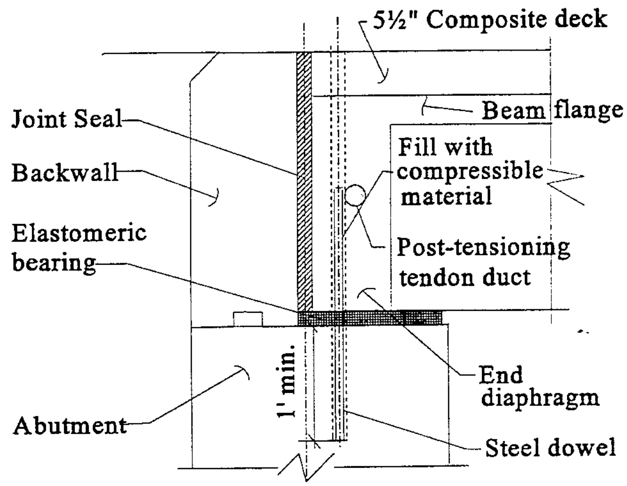
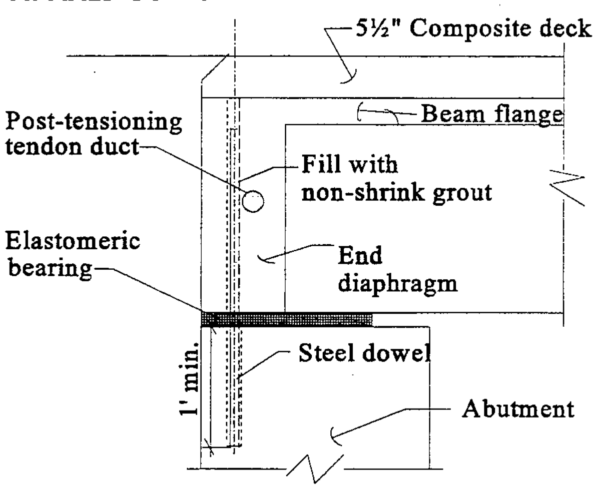
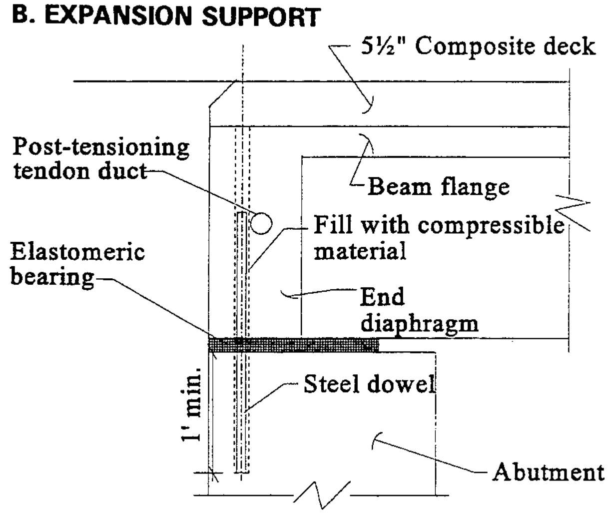


Figure 20. Supports at abutment without backwall.

A. FIXED SUPPORT



B. EXPANSION SUPPORT



V. ECONOMIC FEASIBILITY

Estimating costs of standard bridge details is difficult until they have been used in a number of bridges, because of numerous influencing factors unique to each bridge, such as its size, site accessibility, and whether it was contracted as a single project or in combination with others. Highway agencies thus rely on data from previously awarded contracts to estimate average costs. Estimating life-cycle cost is also difficult, because of complex effects of bridge environment, quality of construction, available maintenance, etc.

Economic feasibility of the double-T beams proposed here is discussed in direct comparison with voided slabs, on the basis of efficient use of materials. A computer program was specially developed for this purpose. Flexural design was prepared using an HS-25 live load and NYSDOT specifications (*7*) for bridges having similar numbers of lanes, widths, and span lengths for current (voided-slab) and proposed (double-T) sections. Assumed initial and final beam compressive strengths were $f'_{ci} = 4800$ psi and $f'_c = 6000$ psi, respectively, and final compressive strength of the composite deck slab was $f'_c = 3000$ psi. Voided-slab bridges were assumed to have 6-in. (5-in. effective) composite decks in accord with current practice. Four span lengths (33, 41, 47, and 54 ft) were compared, covering the proposed ranges in Figures 2 and 14, with the results summarized in Table 4 for non-composite and composite bridges. For these spans, this table shows savings in concrete of 20 to 30 percent by using double-T beams. Identical savings in concrete resulted because of similar dimensions of the proposed sections for non-composite and composite bridges. Regarding prestressing strands, non-composite double-T bridges in Table 4 also show savings of 16 to 43 percent. Similar savings for composite sections range from 12 to 44 percent.

Table 4. Beam depths for proposed double-T beams and current voided slabs.

Beams Compared	Span Length, ft	Voided Slab Depth, in.	Double-T % Increase In Depth	% Concrete Savings	% Strand Savings
A. NON-COMPOSITE BRIDGES					
DT-14N vs SI-36/SI-48	33	18	36	30	43
DT-20N vs SII-36/SII-48	41	21	45	20	38
DT-20N vs SIII-36/SIII-48	47	24	27	25	16
DT-26N vs SIV-36/SIV-48	54	27	35	23	19
Average			36	25	29
B. COMPOSITE BRIDGES					
DT-14 vs SI-36/SI-48	33	18	22	30	44
DT-20 vs SII-36/SII-48	41	21	33	20	35
DT-20 vs SIII-36/SIII-48	47	24	16	25	19
DT-26 vs SIV-36/SIV-48	54	27	26	23	12
Average			25	25	28

Table 5. Bridges of 25- to 60-ft span length crossing water features.

Bridge Type	State-Owned Total Statewide	State-Owned Total Over Water	State-Owned Percent Over Water	Locally Owned Total Statewide	Locally Owned Total Over Water	Locally Owned Percent Over Water
Steel	5,144	756	15	9,774	3,351	34
Concrete						
Prestressed*	674	274	41	990	509	51
Reinforced*	649	350	54	1,103	392	36
Voided-Slab	223	111	50	646	261	40
Timber	2	0	0	293	214	73
Total	6,692	1,491	22	12,806	4,727	37

*Excluding voided-slab decks.

Also included in Table 2 are percentage increases in beam depth resulting from using the proposed sections instead of voided slabs -- 27 to 45 percent for non-composite bridges, and 16 to 33 percent for composite. Note that the relatively higher percentages of 45 and 33 in this table are associated with increase in depth of double-Ts of Types DT-20N and -20, as opposed to Types SII-36/SII-48 voided slabs at 40-ft span length. This is expected because these types are proposed for use in the span range of 40 to 50 ft, and their depth thus may appear deeper than required for spans closer to 40 ft. This information is important because bridge depth is sometime restricted by available freeboard.

To investigate this issue further, data summarized in Table 5 compiled from the Department's bridge inventories indicate that 22 and 37 percent, respectively, of total state and local bridges in the span range of 25 to 60 ft cross water features. Similar percentages for voided slabs are 50 and 40, respectively.

In summary, the proposed double-T sections can be viable alternatives to voided slabs for at least 50 percent of bridges in the proposed span range. Also, realizing that freeboard is not always restricted, the proposed sections may still be economical options in many of these cases. In addition, these double-Ts can be competitive alternatives to steel structures for bridges carrying utility lines, when voided slabs may not be used.

From this discussion, it may be concluded that when use of the proposed double-T beams is appropriate, they result in more efficient use of materials, compared to current voided slabs. This agrees with conclusions of other states as summarized in Table 3, as well as other studies (*3,4*). The proposed details are also expected to be more economical than voided slabs in terms of life-cycle costs, because transverse post-tensioning of the deck creates monolithic behavior and reduces longitudinal cracking and leakage, thus also reducing deterioration of the bridge structure (*12,13,14,15,16,27*).

VI. CONCLUSIONS

Standard sections and details for double-T beams used with either asphalt wearing surfaces or cast-in-place composite decks are proposed in this report for short-span bridges, ranging in length from 25 to 60 ft. Standards proposed for beams used with asphalt wearing surfaces are intended for rapid, non-composite construction when minimal downtime is desirable during construction. Standards for composite construction to enhance durability are also presented. Three double-T sections are proposed for bridges with asphalt wearing surfaces, and another three of similar dimensions for composite construction.

The proposed beams are classified by web depth and recommended maximum span, and may be fabricated using the same casting bed, which is a notable advantage. The beam sections are also optimized for maximum structural efficiency and minimum depth, to increase their applicability in stream crossings restricted by available freeboard. For beams proposed for use with asphalt wearing surfaces, details have been presented for longitudinal shear keys, transverse post-tensioning of the deck, precast diaphragms, waterproofing membranes, and fixed and expansion supports. For double-T beams with composite, cast-in-place decks, details are presented for cast-in-place diaphragms and for fixed and expansion supports. The standard beam sections proposed here are not intended as substitutes for voided slabs, but rather as alternatives when conditions for their use are appropriate and cost-effective. Investigation has shown that the proposed standard beams may provide average savings of about 25 and 30 percent in concrete and prestressing strands, respectively, when used instead of voided slabs. Double-Ts are not only economical but also beneficial for bridges carrying utility lines, where voided slabs may not be satisfactorily used, and in reducing construction downtime in non-composite bridge applications.

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