FINAL
CONTRACT REPORT
VTRC 09-CR1

DESIGN RECOMMENDATIONS FOR THE OPTIMIZED CONTINUITY DIAPHRAGM FOR PRESTRESSED CONCRETE BULB-T BEAMS

STEPHANIE KOCH
Graduate Research Assistant

CARIN L. ROBERTS-WOLLMANN
Associate Professor

Via Department of Civil and Environmental Engineering
Virginia Polytechnic Institute & State University

VTRC
Virginia Transportation Research Council

http://www.virginiadot.org/vtrc/main/online_reports/pdf/09-cr1.pdf

Standard Title Page—Report on Federally Funded Project

1. Report No.: FHWA/VTRC 09-CR1	2. Government Accession No.:	3. Recipient's Catalog No.:
4. Title and Subtitle: Design Recommendations for the Optimized Continuity Diaphragm for Prestressed Concrete Bulb-T Beams		5. Report Date: November 2008 6. Performing Organization Code:
7. Author(s): Stephanie Koch and Carin L. Roberts-Wollmann		8. Performing Organization Report No.: VTRC 09-CR1
9. Performing Organization and Address: Virginia Transportation Research Council 530 Edgemont Road Charlottesville, VA 22903		10. Work Unit No. (TRAIS): 11. Contract or Grant No.: 83136
12. Sponsoring Agencies' Name and Address: Virginia Department of Transportation 1401 E. Broad Street Richmond, VA 23219; Federal Highway Administration 400 North 8th Street, Room 750 Richmond, VA 23219-4825		13. Type of Report and Period Covered: Final 14. Sponsoring Agency Code:
15. Supplementary Notes:		

16. Abstract:

This research focused on prestressed concrete bulb-T (PCBT) beams made composite with a cast-in-place concrete deck and continuous over several spans through the use of continuity diaphragms. The current design procedure in AASHTO states that a continuity diaphragm is considered to be fully effective if a compressive stress is present in the bottom of the diaphragm when the superimposed permanent load, settlement, creep, shrinkage, 50% live load, and temperature gradient are summed, or if the beams are stored at least 90 days when continuity is established. It is more economical to store beams for fewer days, so it is important to know the minimum number of days that beams must be stored to satisfy AASHTO requirements. In addition, if the beams are stored for 90 days before erection, the positive moment detail must have a factored nominal strength (ϕM_n) greater than 1.2 times the cracking moment (M_{cr}).

In 2005, Newhouse tested the positive moment diaphragm reinforcement detail that is currently being adopted by VDOT. The first objective of this research was to determine if the detail was adequate if beams are stored for 90 days. The second objective was to determine if, based on AASHTO requirements, beams could be stored for fewer than 90 days.

After the analysis of all PCBT beam sizes and a wide variety of span lengths and beam spacings, it can be concluded that Newhouse's detail, four No. 6 bars bent 180° and extended into the diaphragm, is adequate for all beams except for the PCBT-77, PCBT-85, and the PCBT-93 when the beams are stored for a minimum of 90 days. For these three beam sizes, three possible solutions are presented: one with two additional bent strands extended into the continuity diaphragm, one with an additional hairpin bar extended into the diaphragm, and one with L-shaped mild reinforcing bars extended into the diaphragm.

To determine the minimum number of storage days required to satisfy AASHTO's requirement for compression at the bottom of the diaphragm, a parametric study was performed. The PCA Method was used in this analysis with the updated AASHTO LRFD creep, shrinkage, and prestress loss models. The parametric study included all sizes of PCBT beams, with two beam spacings, three span lengths and two beam concrete strengths for each size. Both two-span and three-span cases were analyzed.

It was concluded that about half of the cases result in a significant reduction in the minimum number of storage days if the designer is willing to perform a detailed analysis. The other half of the cases must be stored for 90 days because the total moment in the diaphragm will never become negative and satisfy the AASHTO requirement. In general, narrower beam spacing and higher concrete compressive strength results in shorter required storage duration. A recommended quick check is to sum the thermal, composite dead load, and half of the live load restraint moments. The beam must be stored 90 days if that sum is positive, and a more detailed time-dependent analysis will indicate a shorter than 90 day storage period if that sum is negative.

17 Key Words: Continuity diaphragm, PCBT girders, continuous for live load girder detail	18. Distribution Statement: No restrictions. This document is available to the public through NTIS, Springfield, VA 22161.		
19. Security Classif. (of this report): Unclassified	20. Security Classif. (of this page): Unclassified	21. No. of Pages: 70	22. Price:

Form DOT F 1700.7 (8-72) Reproduction of completed page authorized

FINAL CONTRACT REPORT

DESIGN RECOMMENDATIONS FOR THE OPTIMIZED CONTINUITY DIAPHRAGM FOR PRESTRESSED CONCRETE BULB-T BEAMS

Stephanie Koch
Graduate Research Engineer

Carin L. Roberts-Wollmann
Associate Professor

Via Department of Civil and Environmental Engineering
Virginia Polytechnic Institute & State University

Project Manager
Michael Brown, Ph.D., P.E., Virginia Transportation Research Council

Contract Research Sponsored by
the Virginia Transportation Research Council

Virginia Transportation Research Council
(A partnership of the Virginia Department of Transportation
And the university of Virginia since 1948)

In Cooperation with the U.S. Department of Transportation
Federal Highway Administration

Charlottesville, Virginia

November 2008
VTRC 09-CR1

DISCLAIMER

The project that is the subject of this report was done under contract for the Virginia Department of Transportation, Virginia Transportation Research Council. The contents of this report reflect the views of the authors, who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Virginia Department of Transportation, the Commonwealth Transportation Board, or the Federal Highway Administration. This report does not constitute a standard, specification, or regulation. Any inclusion of manufacturer names, trade names, or trademarks is for identification purposes only and is not to be considered and endorsement.

Each contract report is peer reviewed and accepted for publication by Research Council staff with expertise in related technical areas. The contractor performs final editing and proofreading of the report.

ABSTRACT

This research focused on prestressed concrete bulb-T (PCBT) beams made composite with a cast-in-place concrete deck and continuous over several spans through the use of continuity diaphragms. The current AASHTO design procedure states that a continuity diaphragm is considered to be fully effective if a compressive stress is present in the bottom of the diaphragm when the superimposed permanent load, settlement, creep, shrinkage, 50% live load, and temperature gradient are summed, or if the beams are stored at least 90 days when continuity is established. It is more economical to store beams for fewer days, so it is important to know the minimum number of days that beams must be stored to satisfy AASHTO requirements. In addition, if the beams are stored for 90 days before erection, the positive moment detail must have a factored nominal strength (ϕM_n) greater than 1.2 times the cracking moment (M_{cr}).

In 2005, Newhouse tested the positive moment diaphragm reinforcement detail that is currently being adopted by the Virginia Department of Transportation. The first objective of this research was to determine if the detail was adequate if beams are stored for 90 days. The second objective was to determine if, based on AASHTO requirements, beams could be stored for fewer than 90 days.

After the analysis of all PCBT beam sizes and a wide variety of span lengths and beam spacings, it can be concluded that Newhouse's detail, four No. 6 bars bent 180° and extended into the diaphragm, is adequate for all beams except for the PCBT-77, PCBT-85, and the PCBT-93 beams when the beams are stored for a minimum of 90 days. For these three beam sizes, three possible solutions are presented: one with two additional bent strands extended into the continuity diaphragm, one with an additional hairpin bar extended into the diaphragm, and one with L-shaped mild reinforcing bars extended into the diaphragm.

To determine the minimum number of storage days required to satisfy AASHTO's requirement for compression at the bottom of the diaphragm, a parametric study was performed. The PCA method was used in this analysis with the updated AASHTO LRFD creep, shrinkage, and prestress loss models. The parametric study included all sizes of PCBT beams, with two beam spacings, three span lengths and two beam concrete strengths for each size. Both two-span and three-span cases were analyzed.

It was concluded that about half of the cases result in a significant reduction in the minimum number of storage days if the designer is willing to perform a detailed analysis. The other half of the cases must be stored for 90 days because the total moment in the diaphragm will never become negative and satisfy the AASHTO requirement. In general, narrower beam spacing and higher concrete compressive strength results in shorter required storage duration. A recommended quick check is to sum the thermal, composite dead load, and half of the live load restraint moments. The beam must be stored 90 days if that sum is positive, and a more detailed time-dependent analysis will indicate a shorter than 90 day storage period if that sum is negative.

FINAL CONTRACT REPORT

DESIGN RECOMMENDATIONS FOR THE OPTIMIZED CONTINUITY DIAPHRAGM FOR PRESTRESSED CONCRETE BULB-T BEAMS

Stephanie Koch
Graduate Research Engineer

Carin L. Roberts-Wollmann
Associate Professor

Via Department of Civil and Environmental Engineering
Virginia Polytechnic Institute & State University

INTRODUCTION

Continuity Diaphragms in Composite Systems

A bridge comprising simple-span precast, prestressed bulb-T beams made continuous with a cast-in-place diaphragm and composite with a cast-in-place deck is a very efficient and durable system. In 2005, Newhouse presented a detail for the positive moment connection for Virginia's PCBT beams (Newhouse, 2005). In 2007, the AASHTO specification articles regarding continuity diaphragms were updated to reflect research done as part of NCHRP Project 12-53 (Dimmerling et al.). The research presented in this report was undertaken to determine if the detail recommended by Newhouse is in compliance with the new AASHTO specification articles.

Composite Bridges

A composite bridge system is one in which the deck and the beams are bonded together so that the system strains and deflects as one unit. Composite construction is generally preferred because there is a substantial increase in strength and stiffness when the deck and beams are tied together. However, it is more difficult to calculate the forces in the system due to time-dependent effects, especially in the case of precast prestressed concrete beams with a cast-in-place deck.

The time-dependent effects that occur in the beams and deck include creep, shrinkage, and relaxation of prestressing steel. The most dominant forces and moments develop from differential shrinkage between the deck and beam, which occurs because each component has a different ultimate value and rate of creep and shrinkage. Nevertheless, the entire cross-section must strain compatibly since the beams and the deck are made composite when the deck is poured. The younger concrete in the deck will shrink more than the older concrete in the beam. The beam restrains the deck shrinkage to some degree. The result is that compression develops in the top of the beam and tension develops in the bottom of the deck, since there cannot be discontinuity in the strain through the cross-section of the beam and deck. Figure 1 illustrates this behavior. The forces that develop cause rotation at the end of the beam if it is simply supported, and cause restraint moments to develop in the continuity diaphragm if the bridge is made continuous.

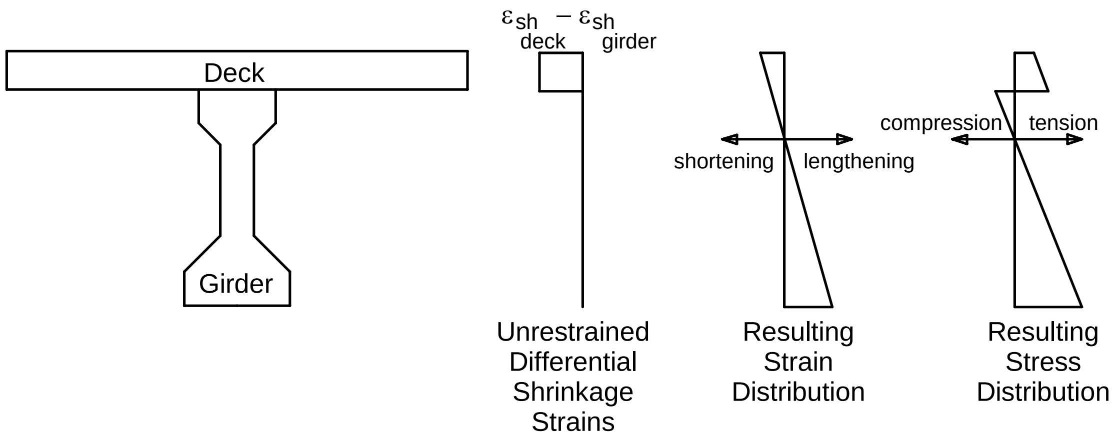


Figure 1. Strains and Stress in a Composite Section

Precast Prestressed Beams Made Continuous

A continuous bridge is one in which two or more simple spans are connected end-to-end with continuity diaphragms (Figure 2). To understand the moments that develop in a continuity diaphragm, consider a simply supported system. The ends of the beam are able to rotate freely throughout the service life of the bridge from the effects of creep, shrinkage, prestress loss, live loads, temperature gradients, and other loading conditions. In a continuous system, no further end rotation is allowed after the continuity diaphragm is poured and the ends of the beams are fixed. Moments must then develop in the continuity diaphragm to restrain the rotations (Figure 3).

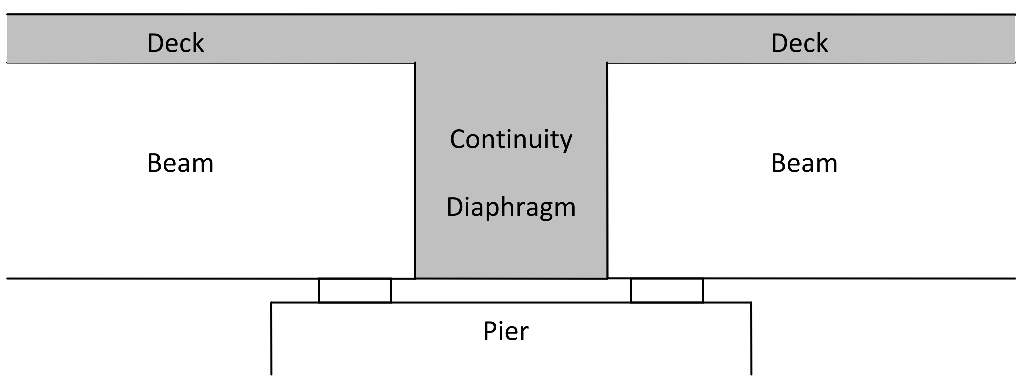


Figure 2. Simple Continuity Diaphragm Illustration

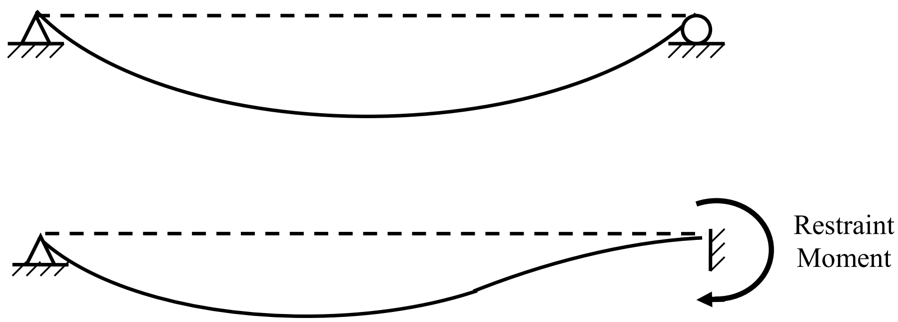


Figure 3. Restraint Moment Illustration

A continuous bridge has several advantages over a series of simple span structures. First, there is a reduction in mid-span bending moment and deflection. This is economical because the beam cross-section can be reduced, or fewer prestressing strands can be used in cases where the member size is fixed (Mattock et al. 1960). Second, making a bridge continuous will improve serviceability by eliminating joints in the deck. The removal of joints will improve the riding surface of the bridge, and durability will be improved because the water and salts from the deck will not drain onto the substructure. Many people consider this the most important advantage (Freyermuth 1969). In addition, the exclusion of joints in a design will reduce the initial cost of the bridge and also reduce bridge maintenance. Finally, a bridge that has been made continuous will redistribute moments if the load capacity is exceeded for a particular beam in the system (Mattock et al. 1960). This provides redundancy.

Although the advantages of continuous systems are numerous and many states are using them, there is not complete agreement on the best method to calculate the restraint moments that develop in the continuity diaphragms or how to detail the positive moment connection. Note that the negative moment connection is not discussed in this document because it is provided through the deck reinforcement, which is much easier to adjust than the positive moment reinforcement which must enter into the end of the beam. This study uses the current design standards, which are the *AASHTO LRFD Bridge Design Specifications,* for the analysis of the positive moment connection in continuity diaphragms (AASHTO 2007).

AASHTO LRFD Bridge Design Specifications

The Virginia Department of Transportation (VDOT) has been designing an increasing number of continuous bridges using the relatively new PCBT beams. The primary goal of this research was to determine if the continuity diaphragms in bridges using PCBT beams are in compliance with current LRFD specifications. Section 5.14.1.4.5 of the *AASHTO LRFD Bridge Design Specifications* states:

> The connection between precast girders at a continuity diaphragm shall be considered fully effective if either of the following are satisfied:
>
> The calculated stress at the bottom of the continuity diaphragm for the combination of superimposed permanent load, settlement, creep, shrinkage, 50 percent live load and temperature gradient, if applicable, is compressive.

> The contract documents require that the age of the precast girders shall be at least 90 days when continuity is established and the design simplifications of Article 5.14.1.4.4 are used.

Section 5.14.1.4.4 states:

> The following simplification may be applied if acceptable to the owner and if the contract documents require a minimum beam age of at least 90 days when continuity is established:
> Positive restraint moments caused by girder creep and shrinkage and deck slab shrinkage may be taken to be 0.
>
> Computation of restraint moments shall not be required.
>
> A positive moment connection shall be provided with a factored resistance, ϕM_n, not less than 1.2 M_{cr}, as specified in Article 5.12.1.4.9.

Therefore, the AASHTO specifications are straightforward and relatively simple as long as the beams are older than 90 days before they are made composite and continuous. However, since it is less economical to wait until beams are 90 days old, it is preferable to store them for less than 90 days even though the calculations are more involved. Determining the forces and moments throughout the life of a bridge system can become a fairly in-depth process, especially if both the deck and the beam are creeping and shrinking at different rates. Therefore, a design aid that determines if the continuity diaphragm is fully effective for beams younger than 90 days would be very beneficial.

PURPOSE AND SCOPE

VDOT has been frequently incorporating the fairly new PCBT beam shape into new bridge designs. In 2005, based on analytical and laboratory research, Newhouse determined that the most efficient detail for the continuity diaphragm for PCBTs was four No. 6 bars bent 180° and extended into the diaphragm (Newhouse 2005). This detail is shown in Figure 4. The research presented in this report was initiated to determine if this detail was adequate to satisfy the new provisions for continuity diaphragms in AASHTO LRFD 2007.

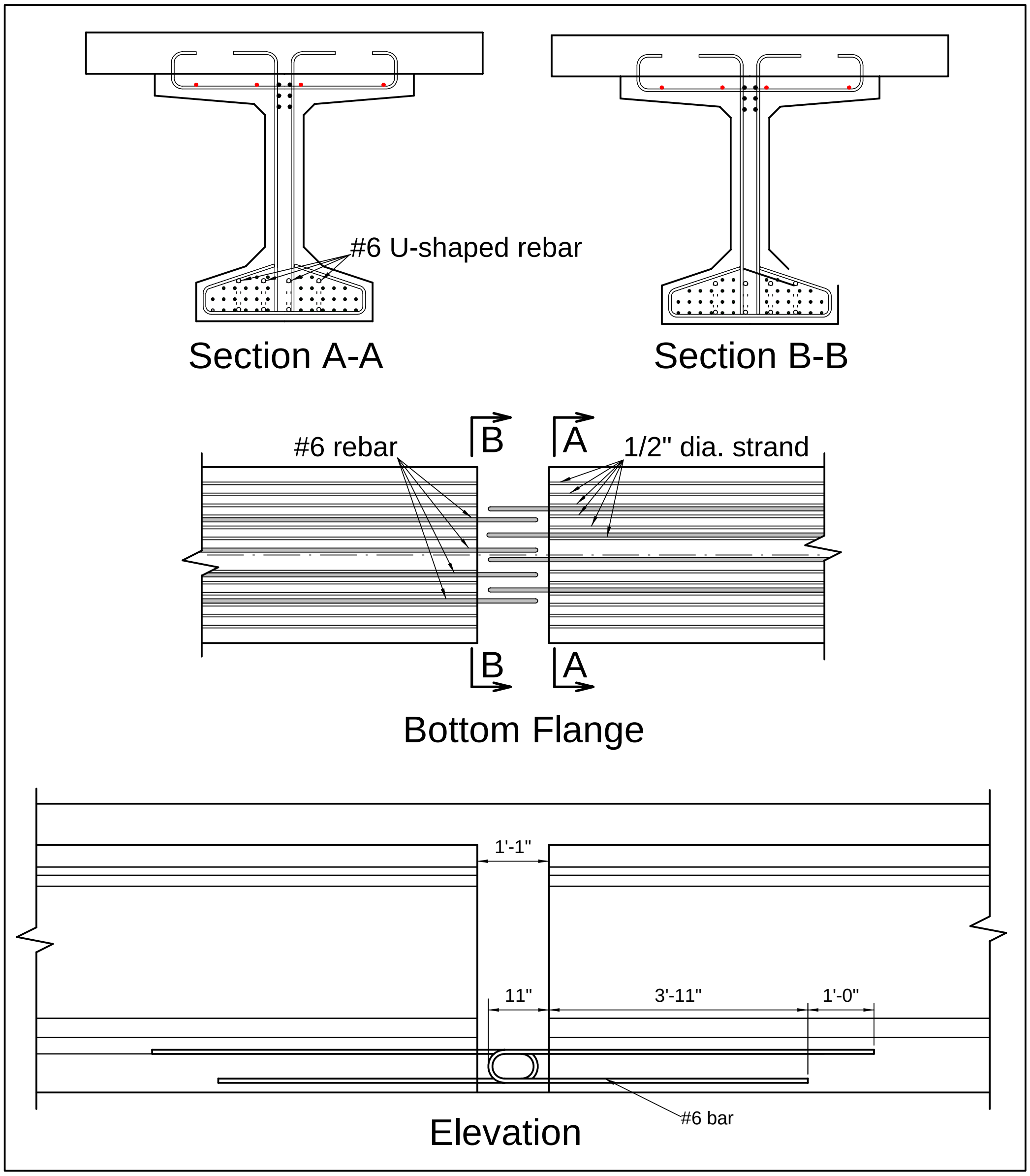


Figure 4. Proposed Continuity Diaphragm Detail

OBJECTIVES

Since Newhouse's work in 2005, *the AASHTO LRFD Bridge Design Specifications* have been updated. This leads to the two primary objectives of this research:

1. Determine if the continuity diaphragm detail developed by VDOT engineers and Virginia Tech researchers and presented in Newhouse (Figure 4) for precast concrete beams made continuous and composite with a cast-in-place deck is adequate for all PCBT beams older than 90 days according to the new AASHTO Specifications. If

the detail is not adequate for particular cases, develop a modified detail with additional strands or mild reinforcing that will provide sufficient moment capacity.

2. Determine the minimum number of days that a particular PCBT beam in a continuous and composite system must age before being erected so that the new AASHTO specifications are satisfied.

To satisfy the first objective of this research, design parameters were varied and calculations were performed for each variation to determine if the Newhouse diaphragm detail for PCBT beams is sufficient for all cases. The parameters that were held constant are as follows:

- The bridge being analyzed is a two-span continuous structure.
- The diaphragm concrete has a compressive strength of 4 ksi.
- The deck thickness is 8 in.
- The haunch height is 1 in.
- The yield strength of the reinforcing bars is 60 ksi.

A two-span continuous system is more critical than a three or more span system, and the other assumptions are typical for VDOT designs. The variable parameters include:

- the beam spacing,
- the span length and
- the beam size.

For each size beam, the design strength (ϕM_n) must be greater than or equal to 1.2 times the cracking moment for all combinations of beam spacings and span lengths. If this requirement is not met, the detail must be modified to satisfy the requirement. For this analysis, the strength of the concrete in the beam did not matter because calculations were based on the strength of the diaphragm concrete, not the beam concrete.

To meet the second objective of this research, it was necessary to develop a design aid (in the form of a MathCAD spreadsheet) that determines for what beam storage duration continuity diaphragms for PCBT beam bridges can be assumed to be fully effective according to Article 5.12.1.4.5 of the AASHTO specifications. A variety of different size PCBT beams at different ages, span lengths, concrete compressive strengths, and deck widths were considered in this study. The design aid simplifies the current procedure and allows for continuous brides to be designed and analyzed more efficiently. This will save time and money in the design and construction processes.

Another component of the second objective was to explore how accurately the PCA method (Mattock et al. 1960) calculates stresses and strains in composite concrete sections. This is important because the PCA method is very commonly used and generally accepted for calculating the restraint moment due to time-dependent effects. Results obtained using the PCA method were compared to results acquired using another method that is considered to be a more accurate way to calculate the stress redistribution in composite sections caused by time-dependent effects of creep and shrinkage. Although the PCA method is frequently used, there are doubts as to how accurate it is, especially when the creep characteristics of the beam and deck are different. Results from this analysis were used to determine if the PCA method can be used in conjunction with the AASHTO LRFD specifications in the development of the design aid for PCBT continuous spans.

METHODS

This section describes the methods used to check the Newhouse diaphragm detail for compliance with AASHTO LRFD 2007 provisions. First the methods used to check requirements for beams older than 90 days are described. Then the methods used to determine the minimum duration of storage required are described.

Beams Older than 90 Days

Background

This portion of this document analyzes prestressed concrete beams that are a minimum age of 90 days when continuity is established. Appendix C presents dimensions for all PCBT beams sizes that were analyzed in this study. Figure 5 is a generic sketch of the beam after it is made composite with a cast-in-place deck.

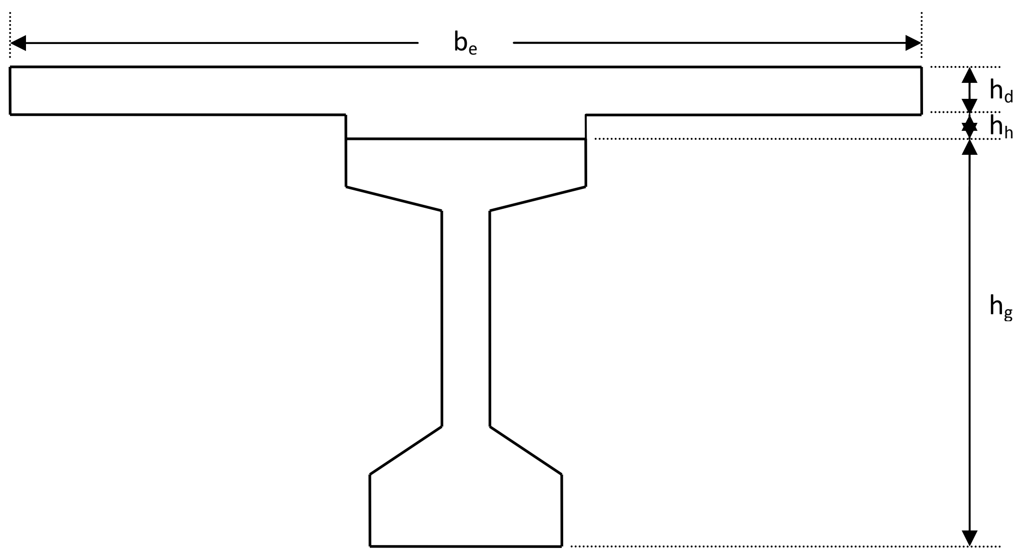


Figure 5. Sketch of PCBT Beam with Deck and Haunch

The objective of this section is to determine if the standard continuity diaphragm detail presented by Newhouse (Figure 4) provides sufficient moment capacity for all PCBT beams older than 90 days. The applicable AASHTO LRFD article requires that the factored nominal moment of the diaphragm be greater than or equal to 1.2 times the cracking moment, as long as the beams are stored 90 days before establishing continuity. Newhouse's detail consists of four No. 6 bars bent at a 180° angle, with a total area of 3.52 in^2. If the detail, with mild reinforcing bars only, does not provide adequate strength, it is necessary to determine how many 0.5-in-diameter prestressing strands or additional No. 6 bars must be extended into the section to provide sufficient moment capacity.

Design Variables and Assumptions

Not all combinations of possible design parameters can be tested. So, some basic assumptions were made to analyze PCBT continuity diaphragm details. Two-span cases are considered in the analysis because they are the most critical. The deck and diaphragm concrete compressive strength is assumed to be 4 ksi, the deck thickness is 8 in, and the haunch height is 1 in. The area of steel in one No. 6 bar is 0.44 in^2, and the yield strength of the bars is 60 ksi. Therefore, the design parameters that are varied are the beam spacing, span length, and beam size. Note that the strength of the concrete in the beam is not a variable, because all calculations are based on the strength of the diaphragm concrete.

Calculations

The diaphragm details are evaluated based on the *AASHTO LRFD Bridge Design Specifications* for diaphragms connecting beams older than 90 days. In particular, the applicable equation is from Article 5.14.1.4.9.

$$\phi M_n \geq 1.2 M_{cr} \tag{1}$$

It is important to provide reserve capacity past cracking in the diaphragm. This ensures that if a crack opens, the steel will not immediately yield and the cracks should remain well controlled. This strength requirement is necessary because additional capacity past cracking will allow for a warning before larger problems occur. Article 5.5.4.2 defines the appropriate resistance factor, ϕ, as 0.9.

The nominal moment resistance, M_n, of the continuity diaphragm, is defined in Article 5.7.3.2.2. The following equation results after the appropriate terms are eliminated for the particular cases being analyzed:

$$M_n = A_{ps} f_{ps}\left(d_p - \frac{a}{2}\right) + A_s f_s\left(d_s - \frac{a}{2}\right) \tag{2}$$

where:

A_{ps} = area of prestressing steel
f_{ps} = average stress in prestressing steel at nominal flexural resistance
d_p = distance from extreme compression fiber to the centroid of prestressing tendons
a = depth of the compression block
A_s = area of non-prestressed tension reinforcement
f_s = stress in the mild steel tension reinforcement at nominal flexural resistance
d_s = distance from extreme compression fiber to the centroid of non-prestressed tensile reinforcement

The area of prestressed and non-prestressed steel, and the distance from the extreme compression fiber of the member to the centroid of the prestressing tendons and to the centroid of the non-prestressed steel, can be easily determined. The area of mild steel is based on the detail developed by Newhouse and is held constant for the initial calculation. For a modified detail with extended strands, the area of the prestressing steel is variable, so it is necessary to adjust the number of strands extended into the continuity diaphragm until the factored nominal moment is greater than 1.2 times the cracking moment for all of the cases that are tested.

Assuming that the strains are within the elastic range, the stress in one strand at general slip can be taken as (Salmons 1975):

$$f_{ps} = \frac{L_e - 8.25}{0.163} \tag{3}$$

where:

L_e = length of the strand

The strand length, L_e, is calculated as the summation of “a” and “b” in Figure 6. According to the research done by the Missouri Department of Transportation and considering the minimum lengths specified by AASHTO, the detail considered for this analysis has a value “a” of 10 in and a value “b” of 20 in. So, the total strand length is 30 in.

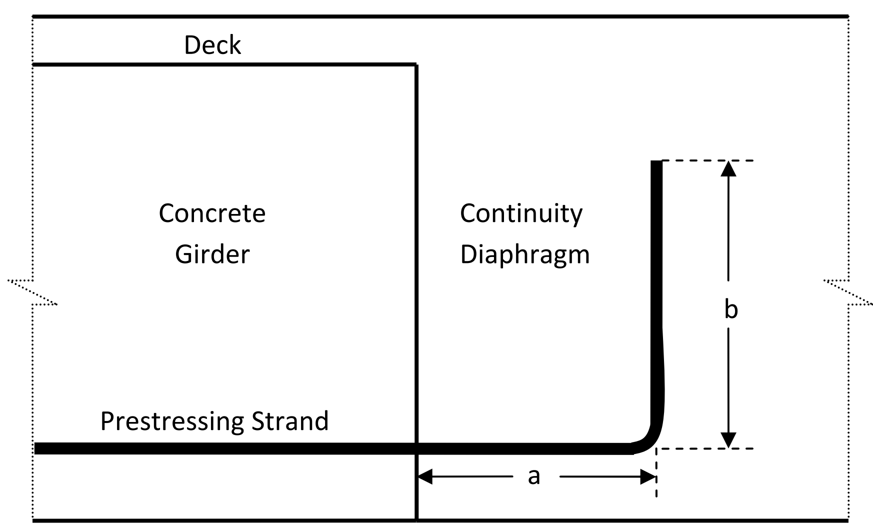


Figure 6. Length of Prestressing Strand Extended into the Continuity Diaphragm

Due to the very broad top flange width, the depth of the compression block, a, can be assumed to be less than the depth of the deck. The equation to determine this value is:

$$a = \frac{A_s f_s + A_{ps} f_{ps}}{0.85 f'_c b_{eff}} \tag{4}$$

where

b_{eff} = the effective flange width
f'_c = the specified compression strength.

Note that it is important to confirm that the depth of the compression block is indeed less than the depth of the deck after it has been calculated.

The effective flange width, or b_{eff}, is defined in Article 4.6.2.6.1 of AASHTO, and must be calculated to determine the depth of the compression block, a. So, b_{eff} is the least of:

- ¼ the effective span length
- 12 times the average depth of the deck, plus the greater of the web thickness or ½ the width of the top flange of the beam
- average adjacent spacing of the beams.

In addition, Article 4.6.2.6 defines the effective span length for a continuous span as being “the distance between the points of permanent load inflection.” So, for a two-span bridge, the distance between points of permanent load inflection is half the span length.

The cracking moment must be found to determine if the diaphragm reinforcement is adequate for beams older than 90 days. Article 5.7.3.3.2 of AASHTO defines the cracking moment, M_{cr}, as:

$$M_{cr} = S_c(f_r + f_{cpe}) - M_{dnc}\left(\frac{S_c}{S_{nc}} - 1\right) \le S_c f_r \tag{5}$$

where

f_{cpe} = compressive stress in concrete due to effective prestress forces only (after allowance for all prestress losses) at extreme fiber of section where tensile stress is caused by externally applied loads
M_{dnc} = total unfactored dead load moment acting on the monolithic or noncomposite section
S_c = section modulus for the extreme fiber of the composite section when tensile stress is caused by externally applied loads.
S_{nc} = section modulus for the extreme fiber of the monolithic or noncomposite section where tensile stress is caused by externally applied loads
f_r = modulus of rupture.

The parameters f_{cpe} and M_{dnc} in the previous equation are equal to 0 for the purpose of calculating the diaphragm cracking moment. Therefore, the above equation can be reduced to:

$$M_{cr} = S_c f_r = \frac{I_{composite}}{y_{composite}} \cdot f_r \tag{6}$$

Where

$I_{composite}$ = the moment of inertia for the composite section
$y_{composite}$ = the distance from the bottom of the beam to the centroid of the composite section.

Article 5.4.2.6 in AASHTO defines the modulus of rupture, f_r, for normal weight concrete as:

$$f_r = 0.24\sqrt{f'_c} \tag{7}$$

with f_r and f'_c in ksi.

Note that f'_c, for the calculations in this section, refers to the compressive strength of the diaphragm concrete, not the compressive strength of the beam.

Sample Calculations

Consider a PCBT-77 beam with a beam spacing of 8 ft and a span length of 130 ft. For this study, the following parameters are considered to be constant:

- Diaphragm compressive strength of 4 ksi
- Slab thickness of 8 in
- Haunch height of 1 in
- Area of steel bars of 3.52 in^2 (Newhouse standard detail of four No. 6 bars bent 180°).

For this particular beam size, the following parameters can be found in Appendix C:

- Beam moment of inertia of 788,700 in^4
- Beam area of 970.7 in^2
- Beam height of 77 in
- Distance from bottom of beam to centroid of 37.67 in.

(See Appendix A for the calculations.)

1.2 M_{cr} = 16,490 in-k
ϕM_n = 16,930 in-k.

No additional strands or mild reinforcing bars are needed.

This calculation was performed for a wide variety of beam sizes, span lengths, and beam spacings.

Beams Younger Than 90 Days

Introduction

Section 5.14.1.4.5 of the *AASHTO LRFD Bridge Design Specifications* gives two conditions that can be used to determine if a bridge can be considered fully continuous for live loads. Either the calculated stress at the bottom of the continuity diaphragm for the combination of superimposed permanent load, settlement, creep, shrinkage, 50 percent live load and temperature gradient, if applicable, is compressive or the beams must be at least 90 days old. It is important to determine for what beam age at the time continuity is established the diaphragm moment is negative, because it is not profitable to store beams longer than necessary. However, one must be able to predict long-term effects in order to determine if the AASHTO requirement is met for beams that are stored less than 90 days before continuity.

This section presents the methods used to calculate the time dependent moment, the live load moment, the composite dead load moment and the thermal gradient restraint moment. By far the most difficult to calculate is the time dependent moment, and there is continued debate about the best method to use to calculate this moment. For the purposes of this research, the PCA method (Freyermuth 1969) was used in the analysis with the updated creep and shrinkage models presented in the *AASHTO LRFD Bridge Design Specifications.* The model is described in the following section.

PCA Method

In the 1950s, the Portland Cement Association (PCA) undertook several projects that focused on composite construction so that an analysis method could be developed (Hognestad et al. 1960). The findings of a well-known researcher, Mattock, were also included in the development of the PCA method which states, "the effects of creep under prestress and dead load can be evaluated by an elastic analysis assuming that the beam and slab were cast and prestressed as a monolithic continuous beam" (Mattock et al. 1961). The result was the "Design of Continuous Highway Bridges with Precast Prestressed Beams" bulletin (Freyermuth 1969), which laid out the PCA method that is still used in the calculation of restraint moments in continuity diaphragms today.

The article published by Freyermuth (1969) stated that the effects of the prestressing force and dead load can be modified to account for creep by multiplying by a factor of:

$$1 - e^{-\phi} \tag{8}$$

The negative restraint moment due to shrinkage can be modified by a factor of:

$$\frac{1 - e^{-\phi}}{\phi} \tag{9}$$

where

ϕ = creep coefficient for the beam

The PCA method also outlined a method to calculate the creep coefficient. The specific creep strain for a loading that occurs at a beam age of 28 days is based on the modulus of elasticity at the time of the loading. This modulus is obtained from a 20-year loading curve, assuming that the ultimate creep occurs at 20 years. Another figure is then used to adjust the creep strain for the actual age of the concrete at loading, which occurs when the beam is prestressed. A size coefficient is used to adjust the creep strain for a particular volume to surface-area ratio that is being analyzed. Since this method is used to analyze composite and continuous systems, another figure is used to determine the coefficient that represents the percent of the ultimate creep that will have occurred at the time the connection is made. The creep strain that must be developed by the continuity diaphragm must, therefore, be adjusted by a factor of 100 percent minus the percent of creep strain that has occurred up to the time of continuity.

The PCA method also defined the differential shrinkage moment due to the different shrinkage rates of the beam and the deck. The differential shrinkage moment can be calculated as follows:

$$M_s = \varepsilon_{diff} \cdot E_b \cdot A_b \cdot \left(e'_2 + \frac{t}{2} \right) \tag{10}$$

where

ε_{diff} = differential shrinkage strain
E_b = elastic modulus for the deck slab concrete
A_b = cross-sectional area of deck slab
e'_2 = centroid of the composite section, measured from top of beam
t = thickness of the slab.

The differential strain was to be calculated, if measurements were not available, based on an ultimate shrinkage strain for both the deck and the beam of 600$\mu\varepsilon$. A time development curve for shrinkage was provided to determine the remaining shrinkage for the beam based on its age at erection.

The 1969 PCA bulletin contained the following equation to calculate the final restraint moment over the pier:

$$M_r = (Y_c - Y_{DL}) \cdot (1 - e^{-\phi}) - Y_s \cdot \left(\frac{1 - e^{-\phi}}{\phi} \right) + Y_{LL} \tag{11}$$

where

M_r = final restraint moment
Y_c = restraint moment at a pier due to creep under prestress force
Y_{DL} = restraint moment at a pier due to creep under dead load

Y_s = restraint moment at a pier due to differential shrinkage between the slab and beam
Y_{LL} = positive live load plus impact moment

The PCA bulletin also recommended positive and negative moment reinforcement. It was determined that a viable option for the positive moment continuity reinforcement was reinforcing bars at right angles that were extended into the diaphragm. This detail was tested at the PCA Labs, and it was recommended that 60% of the yield stress be used in design of the diaphragm so that the live load plus impact stress range is reduced and there is more assurance against the possibility of diaphragm cracking. It was also suggested that the negative moment continuity reinforcement be designed using the compressive strength of the beam concrete.

The theoretical basis for the PCA method is widely considered to be valid and the outcomes are generally conservative. In this case, conservative means that the error is in predicting a more positive moment than actually occurs. However, it is also widely recognized that the creep and shrinkage models presented in the original bulletin are not accurate for today's high-performance, high-strength concretes.

Testing the PCA Method

The time-dependent restraint moment that develops in a continuity diaphragm includes the differential shrinkage restraint moment, the prestress losses restraint moment, the moment to restrain prestress creep rotations, and the moment to restrain dead load creep rotations. The PCA method (Equation 11) is a widely accepted method used to calculate the restraint moment due to time-dependent effects. It is often preferred because it is relatively simple and considered to be conservative.

The work of Alan Mattock is the basis for what is known as the PCA method today (Mattock et al. 1961). He states that moments develop to restrain the end rotation that would have occurred if the beams in continuous spans were not rigidly connected. Mattock concluded from his research that these moments, "are similar in character and distribution to the secondary moments which are set up in monolithic prestressed continuous beams, prestressed by a non-concordant prestressing tendon". He also concluded that, "for design purposes, and assuming usual construction procedures, it may be assumed that the distribution of moments and forces will change toward that which would have occurred if the loads applied to the individual elements before continuity was established had instead been applied to the structure after continuity was established" (Mattock et al. 1961). Mattock assumes that the creep coefficients of the beam and deck are the same, but problems arise because that is not the case for most real bridge structures. Also, it has been debated if the prestress force should be applied to the beam alone instead of the composite cross-section as Mattock suggests. Therefore, there are two questions that needed to be answered before the PCA method was used in the calculation of prestress restraint moments for this research:

- Should the prestress moment be applied only to the beam or to the composite cross-section?

- Does the PCA method accurately predict the restraint moment in continuity diaphragms if the creep coefficients for the beam and the deck are different?

Separate Sections Method

To answer these questions, results from the PCA method were compared to an alternative method to determine if the PCA method accurately calculates the diaphragm restraint moment due to the prestress force. A method was developed by Trost and updated by Menn to calculate the final stresses in a composite cross-section (Menn 1986). This method is referred to as the separate sections method in this report, and is considered to be an accurate method for calculating the stress redistribution in composite sections caused by time-dependent effects of creep and shrinkage. The stresses from the PCA method were compared to the stresses from the separate sections method to determine the accuracy of the PCA method.

Figure 7 shows the initial creep producing forces and moments, the changes in forces and moments, and the change in strain that occur in a composite system over time due to creep and shrinkage.

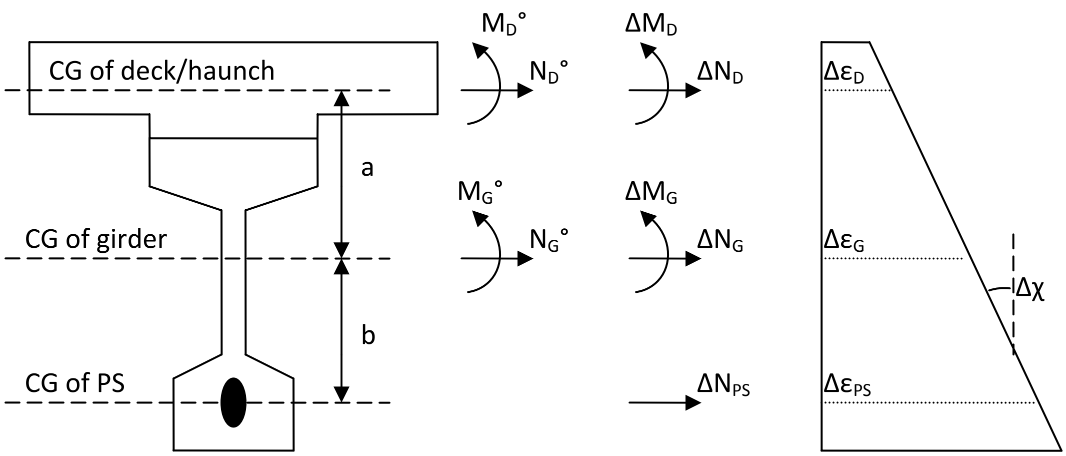


Figure 7. Forces, Moments, and Strain Distribution for a Composite Cross-Section

where

M_D^0 = Initial moment in the deck
N_D^0 = Initial force in the deck
M_G^0 = Initial moment in the beam
N_G^0 = Initial force in the beam
ΔM_D = Change in the moment in the deck
ΔN_D = Change in the force in the deck
ΔM_G = Change in the moment in the beam
ΔN_G = Change in the force in the beam
ΔN_{PS} = Change in the force in the deck
$\Delta \varepsilon_D$ = Change in strain in the deck
$\Delta \varepsilon_G$ = Change in strain in the beam

$\Delta\varepsilon_{PS}$ = Change in strain in the prestress
$\Delta\chi$ = Change in curvature of the system
a = Distance from centroid of the deck and haunch to the centroid of the beam
b = Distance from the centroid of the beam to centroid of prestress.

The separate sections method is based on the equations of internal equilibrium, the equations relating forces to deformations in the beam and the deck (constitutive equations), and compatibility of deformations through the depth of the cross-sections using the above listed variables. It is assumed that all changes in moments and forces are positive, so a negative change in force resulting from the solution of the simultaneous equations denotes a more compressive force. Tensile stresses and elongating strains are considered to be positive, whereas moments and curvatures with compression at the top and tension at the bottom are positive.

Equilibrium:

$$\Delta N_D + \Delta N_G + \Delta N_{PS} = 0 \tag{12}$$

$$a \cdot \Delta N_D + b \cdot \Delta N_{PS} + \Delta M_D + \Delta M_G = 0 \tag{13}$$

Constitutive:

$$\Delta\varepsilon_D = \frac{N_D^{\ o}}{E_D A_D}\phi_D + \frac{\Delta N_D}{E_D A_D}(1 + \mu \cdot \phi_D) \tag{14}$$

$$\Delta\varepsilon_G = \frac{N_G^{\ o}}{E_G A_G}\phi_G + \frac{\Delta N_G}{E_G A_G}(1 + \mu \cdot \phi_G) \tag{15}$$

$$\Delta\varepsilon_{ps} = \frac{\Delta N_{ps}}{A_{ps} \cdot E_{ps}} \tag{16}$$

$$\Delta\chi = \frac{M_D^{\ o}}{E_D I_D}\phi_D + \frac{\Delta M_D}{E_D I_D}(1 + \mu \cdot \phi_D) \tag{17}$$

$$\Delta\chi = \frac{M_G^{\ o}}{E_G I_G}\phi_G + \frac{\Delta M_G}{E_G I_G}(1 + \mu \cdot \phi_G) \tag{18}$$

Compatibility:

$$\Delta\varepsilon_D = \Delta\varepsilon_G - \Delta\chi \cdot a \tag{19}$$

$$\Delta\varepsilon_{PS} = \Delta\varepsilon_G + \Delta\chi \cdot b \tag{20}$$

After the equations are derived, they can be solved simultaneously. The initial forces and moments are considered to be known parameters because they are found from the initial loads on the section. Solving the system of equations gives the changes in the forces, moments, and strains in the system. Note that shrinkage was ignored for this study. Also notice that the prestress relaxation is not included in this analysis because it is considered to be negligible compared to the other forces.

Calculation of Change in Stresses Using the Separate Sections Method

Equations 12 through 20 are solved simultaneously so the stress distribution changes can be determined for the cross-section. Consider an example of a PCBT 77 beam with a span length of 130 ft, beam spacing of 6 ft, 38 prestressing strands, and a creep coefficient of 2.0 for the beam and deck. The given parameters at mid-span are found in Table 1. The stress distribution for this example at mid-span, found using the separate sections method, is shown in Figure 8.

The first term in Equation 11 (final moment per the PCA method) calculates the time dependent moment due to the prestressing moment minus the dead load moment times the reduction factor $\left(1-e^{-\phi}\right)$. This is the portion of the PCA method equation that is being analyzed in this study. So, the initial moments and forces, inserted into Equations 12 through 20, are computed due to the prestress force and the deck and self-weight moments only. Differential shrinkage moments and live load moments are not considered for this comparison.

Table 1. Sample Given Parameters for Testing the PCA Method

A_D	576 in^2
E_D	3605 ksi
N_{D0}	0 kips
ϕ_D	2.0
I_D	3072 in^4
M_{D0}	0 in-kips
μ	0.8
A_G	970.7 in^2
E_G	5098 ksi
N_{G0}	-942 kips
ϕ_G	2.0
I_G	788700 in^4
M_{G0}	9167 in-kip
A_{PS}	5.814 in^2
E_{PS}	28000 ksi
a	43 in
b	29.7 in

Calculation of Rotation Using the Separate Sections Method

The goal of this analysis is to determine if the PCA method accurately predicts the restraint moments in continuity diaphragms due to prestressing forces when compared to the separate sections method. Therefore, it is important to determine if the two methods give similar rotations at the end of the beam, since the rotation at the ends of the beams is needed to compute the restraint moments that develop in continuity diaphragms. Note that change in curvature is defined as the rate of strain change through the depth of a section, while change in rotation is considered to be the amount that the section rotates (in radians) over a given time.

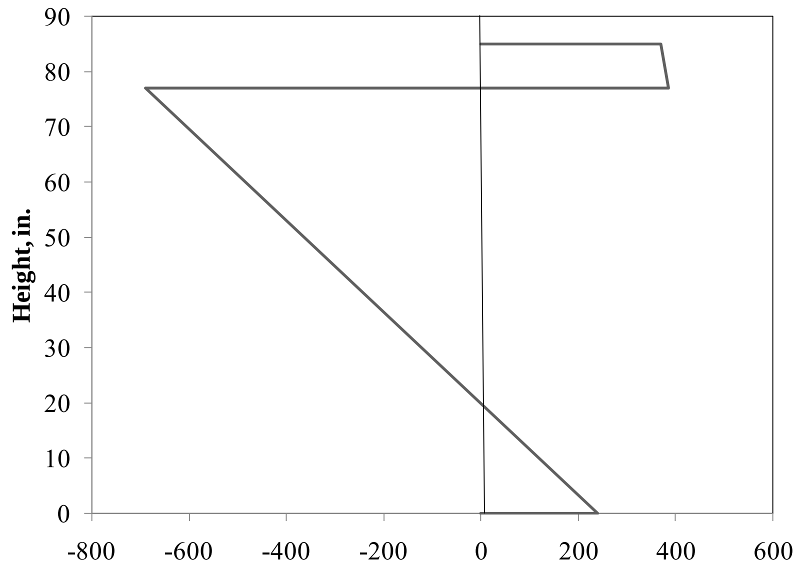


Figure 8. Change in Stress Distribution through Cross-Section

To estimate the change in rotation, the change in curvature should be obtained for several critical points. These include the end of the beam, the end of the transfer length, half the distance to the harping point, the harping point, and mid-span. It is necessary to calculate the change in rotation at these points, if not more, because the location of the center of gravity of the strands will cause a variable prestressed moment along the length of the beam. Once a plot of the change in curvature vs. length along the span is created, the moment area method is used to compute the change in rotation.

Figure 9 presents a general plot of how the change in curvature varies from the support to mid-span. Note that, for simplicity, it is assumed that the plot is a straight line between each of these points so that the area under a portion of the curve (and therefore the change in rotation) can be found by averaging the changes in curvature between points and multiplying by the distance between them. The total change in rotation at a support is then the total area under half of the change in curvature diagram, if the beam is symmetric about mid-span.

Note that a certain distance is needed to develop the prestressing force, which is known as the transfer length. AASHTO states that testing indicates that the transfer length is about 50 times the strand diameter. Therefore, the prestressing force, and consequently the moment caused by the prestressing force, is zero at the end of the beam. It is assumed in our example, which uses 0.5 in diameter prestressing strands, that the full prestressing force is transferred at 25 in from the end of the beam.

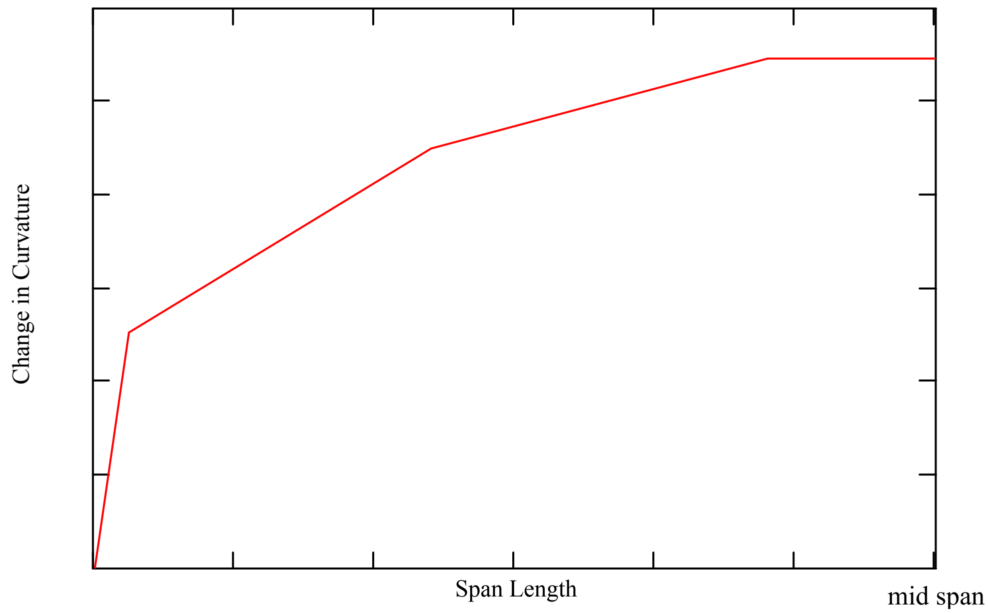


Figure 9. Sample of Change in Curvature along Half of the Span Length

PCA Method

As previously mentioned, the PCA method only considers the creep coefficient of the beam. This allows stresses and strains to be computed directly, which avoids the complicated simultaneous equations necessary in the separate sections method. In this section, stresses are computed with the PCA and compared to the results from the separate sections method. This example is again for a PCBT 77 beam with a span of 130 ft, beam spacing of 6 ft, 38 strands, and creep coefficient of 2.0 for the beam and deck, as was discussed in the previous section.

Calculation of Change in Stresses Using the PCA Method

The PCA method assumes that, with a change in statical system, a system creeps from its original state toward the state it would have been in if it was originally constructed in its final configuration. The difference, in moments or forces or stresses, between the original and final state is multiplied by $\left(1-e^{-\phi}\right)$ to reflect the influence of creep on the change in system.

First, the stress in the original configuration due to the prestress and dead load must be calculated. This calculation uses the section properties of the bare beam. This is because there would be no transfer of force or moment from the prestress in the beam to the deck or haunch at the time the deck is placed. Also note, if creep of the beam were zero, this would also be the final distribution of stress through the cross-section. Figure 10 shows the initial stress distribution on the bare beam at mid-span.

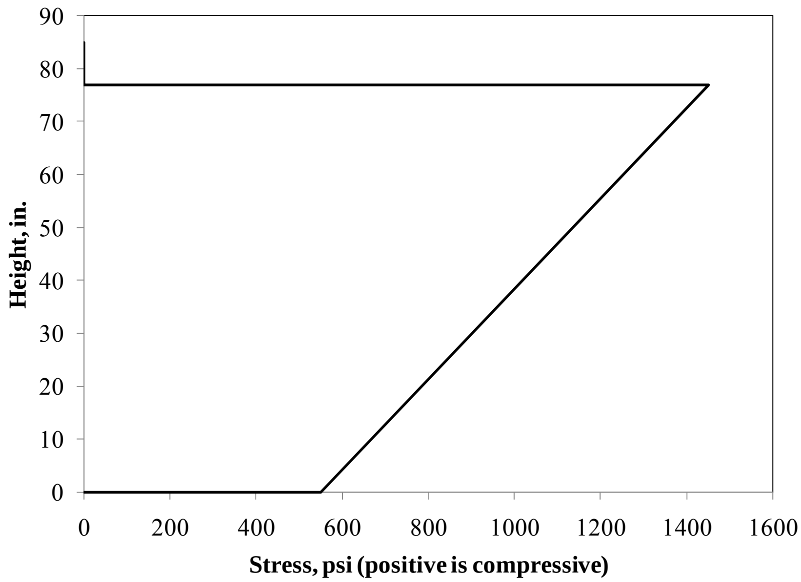


Figure 10. Initial Stress through Cross-Section

Then, the stress is computed if the creep is infinite, which is the state of stress if all forces and moments were applied to the final composite system. The composite cross-sectional properties of the beam and deck are used in the calculations. The stress distribution is shown in Figure 11. Note the sudden change in stress at the deck-beam interface (at the beam height, 77 in). This is due to the difference in the moduli of elasticity at this point.

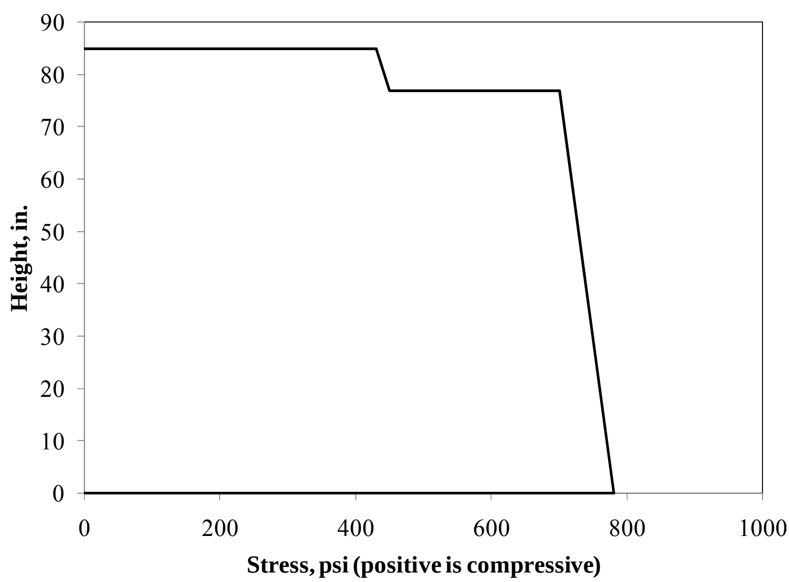


Figure 11. Stress Through Cross-Section If Creep Is Infinite (forces applied to final system)

Subtracting the stress in the initial configuration from the stress in the final configuration (stresses if creep is zero from the stresses if creep is infinite) will yield the change in stress due to creep. This is shown in Figure 12.

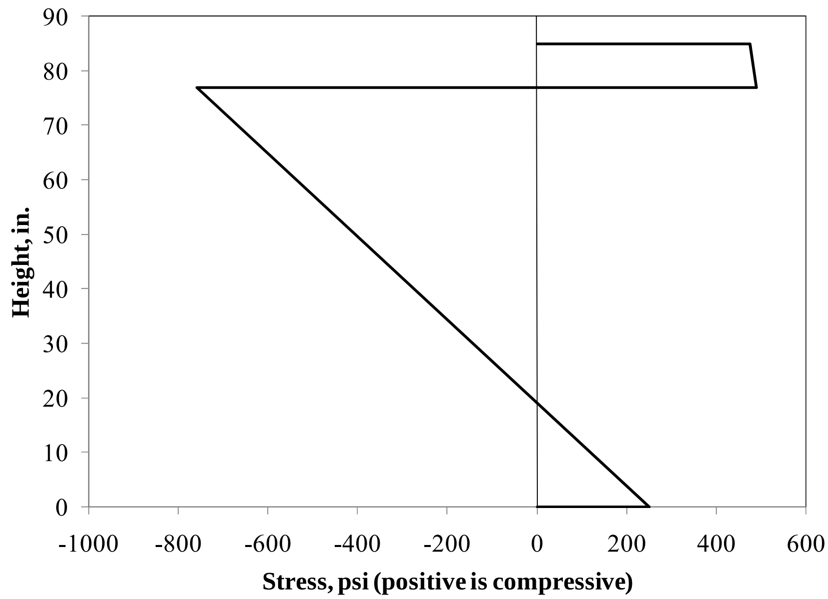


Figure 12. Change in Stress Through Cross-Section (from Zero to Infinite Creep)

Then, the final stress for the top of the deck, bottom of the deck, top of the beam, and bottom of the beam can be found using the relationships defined in the PCA Method. They are:

$$f_{td,f} = f_{td,o} + f_{td,c} \cdot \left(1 - e^{-\phi g}\right) \tag{21}$$

$$f_{bd,f} = f_{bd,o} + f_{bd,c} \cdot \left(1 - e^{-\phi g}\right) \tag{22}$$

$$f_{tg,f} = f_{tg,o} + f_{tg,c} \cdot \left(1 - e^{-\phi g}\right) \tag{23}$$

$$f_{bg,f} = f_{bg,o} + f_{bg,c} \cdot \left(1 - e^{-\phi g}\right) \tag{24}$$

where

$f_{td,f}$ = stress in the top of the deck at final time
$f_{t\,d,o}$ = stress in the top of the deck in initial configuration
$f_{t\,d,c}$ = change in stress in the top of the deck from initial to final with infinite creep
ϕ_g = creep coefficient of the beam.

The other variables in Equations 22 through 24 are defined similarly to those in Equation 21 for the bottom of the deck, the top of the beam, and the bottom of the beam. Note that differential shrinkage is ignored.

Calculation of Rotation Using the PCA Method

As mentioned previously, in addition to comparing stresses, it is important to compare the unrestrained rotation at the end of the beams since that will cause diaphragm restraint moments to develop in continuous spans. The PCA Method states that the rotation at the end of the beam due to prestress can be computed using the following equations:

$$\theta pca_s = \frac{S_s \cdot P_{eff} \cdot (y_{comp} - e_s) \cdot L}{2 \cdot E_g \cdot I_{comp}} \tag{25}$$

$$\theta pca_h = \frac{S_h \cdot P_{eff} \cdot \left[0.1(y_{comp} - eh_{ms}) - 0.2(eh_{end} - y_{comp})\right] \cdot L}{E_g \cdot I_{comp}} \tag{26}$$

$$\theta pca = \theta pca_s + \theta pca_h \tag{27}$$

where

θ_{pcas} = rotation due to straight strands
θ_{pcah} = rotation due to harped strands
θ_{pca} = total rotation
S_s = number of straight strands
S_h = number of harped strands
P_{eff} = effective prestressing force per strand
y_{comp} = composite centroid measured from the bottom of the beam
e_s = centroid of straight prestressing strands measured from the bottom of the beam
eh_{ms} = centroid of harped prestressing strands from the bottom of the beam at mid-span
eh_{end} = centroid of harped prestressing strands from the bottom of the beam at end of beam
L = span length
E_g = modulus of elasticity of the beam
I_{comp} = composite moment of inertia.

One of the objectives of this comparison was to determine if θ_{pca} should be computed by applying the prestress force at an eccentricity relative to the centroid of the bare beam or relative to the composite centroid. Also, notice that Equations 25 and 26 are derived using the moment area method, so for reference, the M/EI diagrams for straight and harped strands are shown in Figures 13 and 14.

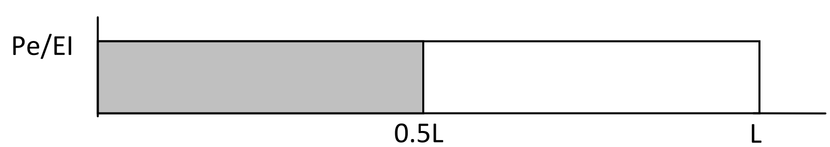


Figure 13. M/EI Diagram for Straight Strands

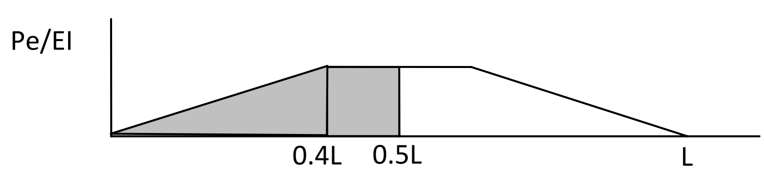


Figure 14. M/EI Diagram for Harped Strands

Prestress Applied to Composite Cross-Section

A brief study was performed to determine if the prestress force should be applied to the composite cross-section as recommended by the PCA method. The prestress was the only loading condition considered in this study, and shrinkage was ignored. The work of Newhouse (Newhouse 2005) contains a design example which was used to compare the PCA method and the separate sections method. The parameters are presented in Table 2.

Table 2. Design Parameters from Newhouse

Beam Size	PCBT-45
Beam compressive strength	8 ksi
Span length	100 ft
Modulus of the beam	4578 ksi
Moment of inertia of the beam	207,300 in^4
Distance from beam centroid to the bottom	22.23 in
Area of the beam	746.7 in^2
Deck width	6 ft
Thickness of the deck	7.5 in
Haunch thickness	1.5 in
Composite moment of inertia	432700 in^4
Creep Coefficients (Beam and Deck)	2.0

The updated AASHTO LRFD models for creep, shrinkage, and prestress loss are used in the time-dependent calculations. The separate sections method gives a rotation of 16,705,478/EI. The rotation using the PCA method when the prestressing force is applied to the bare beam is 9,921,428/EI, while the rotation when the PCA method is used applying the prestressing force to the composite beam gives a rotation of 16,943,831/EI. Newhouse (2005) provides a more in-depth discussion of the calculations using the separate sections method.

Notice that there is substantially more difference when the prestress is applied only to the beam cross-section. No further study was undertaken because limited results showed very clearly that applying the prestress to the composite section is more correct.

Set-Up and Results

Several trials with a variety of parameters were run to determine if the PCA method and the separate sections method gave similar results. It was out of the scope of this document to analyze all combinations of parameters that could be associated with this problem, so a few were selected that would provide an overall representation of results. The parameters that were used include three types of PCBT beams (45, 61, and 77), a deck width of 6 ft and 9 ft, and a long and medium span length for each case. The suggested number of prestressing strands for each trial was obtained from the VDOT preliminary design guide (VDOT 2006).

Comparison of Stresses

The prestressing force was applied to the composite cross-section. For each trial the stresses were calculated using the PCA method and the separate sections method. The calculated stress results showed that there was less than 5% difference in the calculation of stress at the bottom and top of the deck and at the bottom and top of the beam for all cases where the creep coefficient of the beam and the deck were equal. However, there was a significant difference when the creep coefficients were not equal. In general, the greatest percent difference occurred in the top of the deck (about 20% to 25%), and the smallest difference occurred in the bottom of the beam (5% or less). The PCA method conservatively over-predicted the stress in the bottom of the beam and deck in nearly all cases, but was not always conservative when predicting the stress in the top of the beam and deck.

As mentioned earlier, the PCA method is a simplified approach that assumes the creep coefficients of the beam and the deck are the same and equal to the creep coefficient of the beam. The results from this study provide reassurance that using the creep coefficient of the beam was sufficient if the creep coefficients of the beam and the deck were the same. However, although the PCA method was almost always conservative, results did vary if the creep coefficients were different. Since it seems that the creep coefficient of the beam did not provide highly accurate results, an investigation was undertaken to determine if a modification to the creep coefficient could be made in the PCA method that would more accurately model stresses.

A Better Creep Coefficient

This set of parametric studies addresses if the beam creep coefficient can be modified so that use in the PCA method would provide more accurate results in composite systems. It was quickly determined that an average or a weighted average of the beam and deck creep coefficients was not the best solution. Using one of these methods increases the creep coefficient used in the PCA method because the deck creep coefficient is always greater than or equal to the beam creep coefficient because the deck concrete is younger than beam concrete. The creep coefficients that give results closest to those found using the separate sections method were less than the beam creep coefficients.

A parametric study was completed to determine the best creep coefficient to compute stresses through a cross-section using the previously described trial cases, including beam and deck creep coefficients of 2.00 and 2.00, 1.75 and 2.25, 1.50 and 2.50, 1.25 and 2.75, and 1.00 and 3.00 respectively. Once again, the effects of shrinkage were not included. First, the more exact stresses were found at several locations through the depth of the composite cross-section using the separate sections method and the specified creep coefficients. Then, the stresses were found using the PCA method and a creep coefficient varying from 2.00 to 0.30 with 0.05 increments. The "best fit" PCA creep coefficient, ϕ, was the one that minimized the percent difference for the stresses along the depth of the composite cross-section when compared to those obtained using the separate sections method.

For example, consider a PCBT 45 beam, with 6 ft beam spacing, a span length of 60 ft, and 16 prestressing strands. Stresses were calculated using the separate sections method with numerous values of the beam and deck creep coefficients, and are presented in Table 3. The stresses were then calculated using the PCA method with varying creep coefficients. For the above example, the PCA stresses are shown in Table 4.

Table 3. Sample Stresses (ksi) Using the Separate Section Method

ϕg	ϕd	Top Deck	Bottom Deck	Top Beam	Bottom Beam
2.0	2.0	0.072	0.119	-0.256	0.655
1.75	2.25	0.067	0.103	-0.281	0.647
1.5	2.5	0.061	0.089	-0.306	0.638
1.25	2.75	0.053	0.074	-0.333	0.628
1.0	3.0	0.045	0.060	-0.362	0.618

After the stresses were computed using the separate sections method and PCA method, they were compared. Table 5 presents the percent difference of stresses with the beam and deck creep coefficients of 2.00.

The percent difference that is shown in Table 5 is the difference in stresses calculated using the PCA and separate sections methods. As shown in Table 5 by the highlighted region, the creep coefficient recommended for use by the PCA method is 2.00 because the creep coefficient of the beam is 2.00. However, it is observed in Table 5 that a creep coefficient of about 1.80 should be used in the PCA method to produce results closest to those obtained using the more accurate separate sections method. Note that the difference may be due to the aging coefficient that must be assumed in the separate sections method. A typical value of 0.8 for the aging coefficient was used in this example.

Likewise, consider beam and deck creep coefficients of 1.75 and 2.25, respectively. Table 6 illustrates the associated percent difference. The creep coefficient used for the PCA method is 1.75 because the PCA method specifies that the creep coefficient of the beam be used in calculations. However, 1.35 is a more accurate estimation of a creep coefficient that would give similar results in stress to those obtained using the separate sections method. This more accurate creep coefficient, ϕ, will be regarded as the "best fit ϕ" for the remainder of this document.

This process of determining a creep coefficient that can be used in the PCA method to produce stresses closest to those from the separate sections method was repeated for creep coefficients of the beam and deck of 1.50 and 2.50, 1.25 and 2.75, and 1.00 and 3.00, respectively. Then, this process was repeated for different beam sizes, span lengths, and beam spacings, and the results are shown in Table 7.

Table 4. Sample Stresses (ksi) Using the PCA Method

PCA ϕ	Top Deck	Bottom Deck	Top Beam	Bottom Beam
2.00	0.075	0.123	-0.247	0.658
1.95	0.074	0.122	-0.249	0.657
1.90	0.074	0.121	-0.251	0.656
1.85	0.073	0.120	-0.253	0.655
1.80	0.072	0.119	-0.256	0.655
1.75	0.071	0.117	-0.258	0.654
1.70	0.071	0.116	-0.261	0.653
1.65	0.070	0.115	-0.263	0.652
1.60	0.069	0.113	-0.266	0.651
1.55	0.068	0.112	-0.269	0.650
1.50	0.067	0.110	-0.272	0.649
1.45	0.066	0.109	-0.275	0.648
1.40	0.065	0.107	-0.278	0.647
1.35	0.064	0.105	-0.282	0.645
1.30	0.063	0.103	-0.286	0.644
1.25	0.062	0.101	-0.290	0.643
1.20	0.060	0.099	-0.294	0.641
1.15	0.059	0.097	-0.298	0.640
1.10	0.058	0.095	-0.302	0.638
1.05	0.056	0.092	-0.307	0.636
1.00	0.055	0.090	-0.312	0.635
0.95	0.053	0.087	-0.318	0.633
0.90	0.051	0.084	-0.323	0.631
0.85	0.050	0.081	-0.329	0.629
0.80	0.048	0.078	-0.335	0.627
0.75	0.046	0.075	-0.341	0.624
0.70	0.044	0.072	-0.348	0.622
0.65	0.041	0.068	-0.355	0.619
0.60	0.039	0.064	-0.363	0.617
0.55	0.037	0.060	-0.371	0.614
0.50	0.034	0.056	-0.379	0.611
0.45	0.031	0.052	-0.388	0.608
0.40	0.029	0.047	-0.397	0.605
0.35	0.026	0.042	-0.406	0.601
0.30	0.022	0.037	-0.416	0.598

Table 5. Percent Difference of Stresses for a Beam and Deck ϕ = 2.00

ϕg	Top Deck	Bottom Deck	Top Beam	Bottom Beam
2.00	-4	-3	4	0
1.95	-3	-3	3	0
1.90	-3	-2	2	0
1.85	-1	-1	1	0
1.80	0	0	0	0
1.75	1	2	-1	0
1.70	1	3	-2	0
1.65	3	3	-3	0
1.60	4	5	-4	1
1.55	6	6	-5	1
1.50	7	8	-6	1
1.45	8	8	-7	1
1.40	10	10	-9	1
1.35	11	12	-10	2
1.30	13	13	-12	2
1.25	14	15	-13	2
1.20	17	17	-15	2
1.15	18	18	-16	2
1.10	19	20	-18	3
1.05	22	23	-20	3
1.00	24	24	-22	3
0.95	26	27	-24	3
0.90	29	29	-26	4
0.85	31	32	-29	4
0.80	33	34	-31	4
0.75	36	37	-33	5
0.70	39	39	-36	5
0.65	43	43	-39	5
0.60	46	46	-42	6
0.55	49	50	-45	6
0.50	53	53	-48	7
0.45	57	56	-52	7
0.40	60	61	-55	8
0.35	64	65	-59	8
0.30	69	69	-63	9

Table 6. Percent Difference of Stresses for a Beam and Deck ϕ of 1.75 and 2.25

PCA ϕ	Top Deck	Bottom Deck	Top Beam	Bottom Beam
2.00	-12	-19	12	-2
1.95	-10	-18	11	-2
1.90	-10	-17	11	-1
1.85	-9	-17	10	-1
1.80	-7	-16	9	-1
1.75	-6	-14	8	-1
1.70	-6	-13	7	-1
1.65	-4	-12	6	-1
1.60	-3	-10	5	-1
1.55	-1	-9	4	0
1.50	0	-7	3	0
1.45	1	-6	2	0
1.40	3	-4	1	0
1.35	4	-2	0	0
1.30	6	0	-2	0
1.25	7	2	-3	1
1.20	10	4	-5	1
1.15	12	6	-6	1
1.10	13	8	-7	1
1.05	16	11	-9	2
1.00	18	13	-11	2
0.95	21	16	-13	2
0.90	24	18	-15	2
0.85	25	21	-17	3
0.80	28	24	-19	3
0.75	31	27	-21	4
0.70	34	30	-24	4
0.65	39	34	-26	4
0.60	42	38	-29	5
0.55	45	42	-32	5
0.50	49	46	-35	6
0.45	54	50	-38	6
0.40	57	54	-41	6
0.35	61	59	-44	7
0.30	67	64	-48	8

Table 7. Comparison of PCA Phi to Best Fit ϕ

Beam, Span, Length	ϕg, ϕd	Difference of ϕg & ϕd	PCA ϕ	Best Fit ϕ	Difference in ϕ, PCA & Best Fit	% Difference of ϕ, PCA & Best Fit
PCBT45,S6,L90	2.00,2.00	0.0	2.00	1.80	0.20	11
	1.75,2.25	0.5	1.75	1.40	0.35	25
	1.50,2.50	1.0	1.50	1.05	0.45	43
	1.25,2.75	1.5	1.25	0.83	0.42	51
	1.00,3.00	2.0	1.00	0.60	0.40	67
PCBT45,S6,L60	2.00,2.00	0.0	2.00	1.80	0.20	11
	1.75,2.25	0.5	1.75	1.35	0.40	30
	1.50,2.50	1.0	1.50	1.05	0.45	43
	1.25,2.75	1.5	1.25	0.80	0.45	56
	1.00,3.00	2.0	1.00	0.60	0.40	67
PCBT45,S9,L60	2.00,2.00	0.0	2.00	1.80	0.20	11
	1.75,2.25	0.5	1.75	1.43	0.32	22
	1.50,2.50	1.0	1.50	1.10	0.40	36
	1.25,2.75	1.5	1.25	0.85	0.40	47
	1.00,3.00	2.0	1.00	0.62	0.38	61
PCBT61,S6,L120	2.00,2.00	0.0	2.00	1.80	0.20	11
	1.75,2.25	0.5	1.75	1.35	0.40	30
	1.50,2.50	1.0	1.50	1.03	0.47	46
	1.25,2.75	1.5	1.25	0.80	0.45	56
	1.00,3.00	2.0	1.00	0.60	0.40	67
PCBT77,S6,L130	2.00,2.00	0.0	2.00	1.80	0.20	11
	1.75,2.25	0.5	1.75	1.35	0.40	30
	1.50,2.50	1.0	1.50	1.00	0.50	50
	1.25,2.75	1.5	1.25	0.75	0.50	67
	1.00,3.00	2.0	1.00	0.55	0.45	82
Average	2.00,2.00	0.0	2.00	1.80	0.20	11
	1.75,2.25	0.5	1.75	1.38	0.37	27
	1.50,2.50	1.0	1.50	1.05	0.45	44
	1.25,2.75	1.5	1.25	0.81	0.44	55
	1.00,1.00	2.0	1.00	0.59	0.41	69

It can be seen that there is similarity in the percent difference between the PCA ϕ and the Best Fit ϕ among the cases analyzed in Table 7. Graphically, the results are represented in Figure 15. This figure shows that the percent difference increases as the creep coefficients of the beam and the deck become more different. This suggests that the PCA method would accurately predict the stresses in a cross-section as long as the creep coefficients are similar. More research in this area should be done to determine if an appropriate adjustment factor could increase the accuracy of the PCA method.

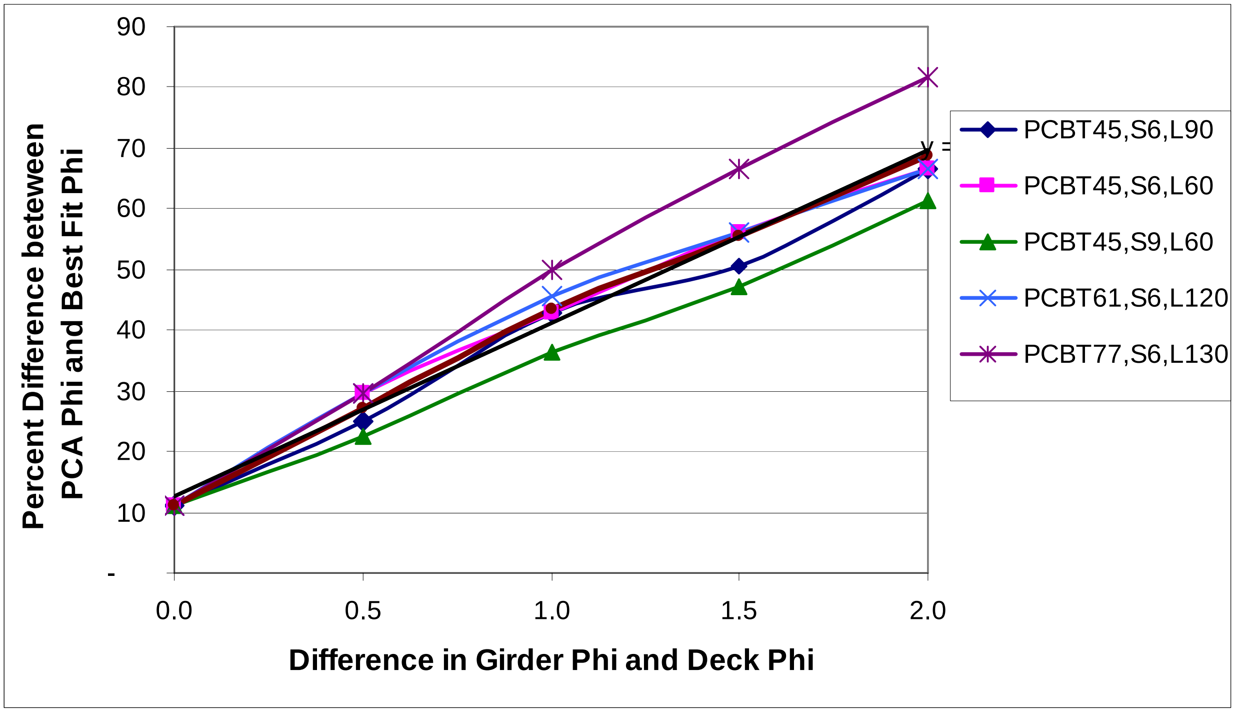


Figure 15. Percent Difference between PCA ϕ and the Best Fit ϕ

Conclusions from the Study of the PCA Method

It is important to note that the PCA method is conservative. In all cases, the beam creep coefficient used in the PCA method was larger than the best fit coefficient. For a particular case, the best fit creep coefficient is the one that can be used in the PCA method to predict stresses closest to those obtained from the separate sections method. This means that the PCA method predicts a greater redistribution of stresses and moments than what is actually occurring in the cross-section. Recognizing this, it was concluded that the PCA method can be used in the calculations of time-dependent moments for the remainder of this document.

Models for Creep and Shrinkage

Although, qualitatively, time-dependent effects in prestressed concrete are fairly well understood, many different models exist to quantitatively calculate their influences. VDOT uses the AASHTO LRFD specifications for design. Therefore, this study used the AASHTO creep, shrinkage, and prestress loss models with the PCA method to determine the time-dependent moments in the diaphragm. One exception was that the expression "$\phi/(1+\mu\phi)$" was used instead of "$(1-e^{-\phi})$". These two factors are generally equivalent, but $\phi/(1+\mu\phi)$, which is the expression used with the age adjusted effective modulus method (AAEM), is consistent with the prestress loss method presented in AASHTO, which is also an AAEM. To show that these two factors are equivalent, consider typical values of 2.0 for ϕ and 0.7 for μ:

$$1 - e^{-\phi} = 1 - e^{-2} = 0.865$$

$$\frac{\phi}{1 + \mu\phi} = \frac{2}{1 + 0.7 \cdot 2} = 0.833$$

In addition, some additional consideration of the time-dependent moment that is applied to the continuity diaphragm is in order. AASHTO states that the time-dependent moment must be considered if it is positive because it produces a more critical result, but it should be ignored if it is negative. This illustrates that there is much uncertainty in modeling the time-dependent effects in concrete, so AASHTO chooses the conservative option of ignoring beneficial effects but considering harmful ones.

AASHTO Creep Model

According to AASHTO LRFD Bridge Design Specifications (2007) Section 5.4.2.3.2, the creep coefficient can be calculated by using the following equations (note that the AASHTO notation for creep coefficient, ψ, is shown in the equations rather than the more typically used notation of ϕ):

$$\psi(\mathrm{t}, \mathrm{t_i}) = 1.9 k_{vs} k_{hc} k_f k_{td} t_i^{-0.118} \tag{28}$$

$$k_{vs} = 1.45 - 0.13(V/S) \geq 0 \tag{29}$$

$$k_{hc} = 1.56 - 0.008H \tag{30}$$

$$k_f = \frac{5}{1 + f'_{ci}} \tag{31}$$

$$k_{td} = \frac{t}{61 - 4f'_{ci} + t} \tag{32}$$

where

$\psi(t,t_i)$ = creep of a member at a given time, t, due to a load applied at an initial time, ti
H = relative humidity (%)
k_{vs} = factor for the effect of the volume-to-surface ratio of the component
k_f = factor for the effect of the concrete strength
k_{hc} = humidity factor for creep
k_{td} = time development factor
t = maturity of concrete (days)
t_i = age of concrete when the initial load is applied (days)
V/S = volume-to-surface area ratio (in)
f'_{ci} = specified compressive strength of concrete at time of prestressing (ksi).

Note that for this model one day of accelerated steam curing is considered equal to 7 days of regular curing. The above calculated factors can be used to predict the creep that will occur from any time to any other time.

AASHTO Shrinkage Model

Section 5.4.2.3.3 of the *AASHTO LRFD Bridge Design Specifications* (2007) defines shrinkage as follows:

$$\varepsilon_{sh} = -k_{vs} k_{hs} k_f k_{td} \, 0.48x10^{-3} \quad (33)$$

$$k_{hs} = 2.00 - 0.014H \quad (34)$$

where

ε_{sh} = shrinkage of a member
k_{hs} = humidity factor for shrinkage.

All other variable are defined in the same manner as for the creep model.

AASHTO Prestress Loss Model

The *AASHTO LRFD Bridge Design Specifications,* Section 5.9.5.4.1, defines the loss in prestress due to time-dependent changes by the following equation:

$$\Delta f_{pLT} = (\Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR1})_{id} + (\Delta f_{pSD} + \Delta f_{pCD} + \Delta f_{pR2} - \Delta f_{pSS})_{df} \quad (35)$$

where

Δf_{pLT} = change in prestressing steel stress due to time-dependent loss
Δf_{pSR} = prestress loss due to shrinkage of beam from transfer to deck placement
Δf_{pCR} = prestress loss due to creep of beam from transfer to deck placement
Δf_{pR1} = prestress loss due to relaxation of strands from transfer to deck placement
Δf_{pSD} = prestress loss due to shrinkage of beam from deck placement to final time
Δf_{pCD} = prestress loss due to creep of beam from deck placement to final time
Δf_{pR2} = prestress loss due to relaxation of strands from deck placement to final time
Δf_{pSS} = prestress loss due to shrinkage of deck from deck placement to final time.

The AASHTO creep and shrinkage models and prestress loss model were used in conjunction with the PCA method to determine the time dependent moments in continuity diaphragms.

Live Load Moments

QConBridge

QConBridge (WSDOT 2005) is a Windows-based software program that was developed by the Washington State DOT to perform live load analyses. It was used in this research to compute the maximum negative moments in continuity diaphragms for two-span and three-span continuous bridges under live load events. For simplification, only systems with equal span lengths were analyzed.

QConBidge uses the AASHTO LRFD HL93 live load model. In particular, this document uses the dual tandem truck train and the lane load to determine the maximum negative moment for each case, which will occur over the interior support. For use in the MathCAD spreadsheet, the greatest negative moments in equal span systems were determined for span lengths from 20 ft to 160 ft in 5 ft increments. All values were added to an input table in the MathCAD spreadsheet for later use in the analysis of the continuity diaphragms.

It is important to note that QConBridge calculates the maximum moments occurring in the structure per lane loaded. However, analysis and design of bridges require that a critical moment per beam be found. So, distribution factors were calculated and used to adjust the moment per lane into a moment per beam. Spacing, deck thickness, and length requirements were checked to make sure that AASHTO recommended distribution factors could be used.

Thermal Effects

Changes in the average temperature of the structure can occur throughout the entire cross-section of the bridge. This will result in translational distortions if the bridge is free to expand and contract, while stresses are introduced if the superstructure is restrained. Also, temperature variations can exist through the depth of the cross-section of the bridge. Temperature gradients will cause rotational distortions if the bridge is unrestrained, and bending moments in bridges that are continuous. It is important to be able to model and analyze the effects of temperature gradients on a continuous concrete structure because the bending moments that are introduced must be included in the total moment that develops in the continuity diaphragm.

Background

Heat can be transferred by radiation, convection, and conduction. Usually the contributions from convection and conduction are so small that a single coefficient will consider these effects (Imbsen et al. 1985). Heat transfer by radiation occurs when the structure is exposed to sunlight, and often results in noticeable thermal changes.

Temperature gradients occur because the top and bottom of a member are exposed to a change in temperature and absorb heat rapidly, while the middle portion is predominately insulated from these effects due to the relatively non-conductive nature of concrete. A positive thermal gradient is one in which the deck is warmer than the beam, and a negative thermal gradient is one in which the deck is cooler than the beam. A maximum positive thermal gradient would occur on a sunny and warm day after a few days of cool overcast weather. Conversely, a negative temperature gradient may be applicable on a cool day following warm weather.

Thermal gradients in structures were often not considered before 1980. Then, even after the importance of accounting for temperature changes was universally recognized, there were many different opinions on how to quantify them to best predict the responses. Although thermal effects were known to introduce some additional strain in the cross-section of a bridge, there were not many reported cases where temperature caused significant damage (Imbsen et al. 1985). Researchers now attribute this to the slightly conservative nature of the design process.

Also, the effects of temperature were often largely overlooked, even in situations where temperature was cited as being a likely reason for cracking, because it was known that creep and shrinkage also contributed to the problem.

Engineers realized that bridge design in the United States could be simplified somewhat if a standard temperature gradient could be agreed upon and modified to account for geographically specific thermal conditions. In 1978, the British Standard BS 5400 published results defining a negative thermal gradient. This gradient was adopted by the 1985 NCHRP Report 276 as the standard negative thermal gradient for use in the United States. In 1983, a study was published by Potgieter and Gamble that outlined a positive thermal gradient to be used in the design of typical US bridges (Potgieter et al. 1983). This gradient was based on data gathered from weather stations around the country. This was the positive thermal gradient that was adopted by the 1985 NCHRP Report 276 (Imbsen et al. 1985) as the standard for use.

NCHRP Report 276 was very important because at the time it was published, states considered the effects of axial length changes to be the most severe temperature effect, and no state included temperature gradient effects in its typical design codes. In the NCHRP report, field tests and meteorological data were compiled to divide the United States into four zones for solar radiation (Imbsen et al. 1985). This temperature gradient was later modified and introduced into the *AASHTO LRFD Bridge Design Specifications.*

Although a standard temperature gradient is used in continuity diaphragm calculations today, thermal effects are still being researched. A series of experimental tests were completed in 2005 as part of NCHRP Project 12-53 (Dimmerling et al. 2005) to examine the performance of diaphragm reinforcement common in the United States. It was concluded that daily temperature changes affect the end reactions in a continuous system by as much as 25%. This finding reinforced the importance of including thermal effects in the design of continuity diaphragms.

AASHTO LRFD Specifications

The *AASHTO LRFD Bridge Design Specifications,* Section 3.12.3, outlines the current temperature gradient that should be used to calculate thermal effects that occur through a cross-section of a bridge system. The standard temperature gradient is from Figure 3.12.3-2 in the AASHTO specifications and is shown in Figure 16.

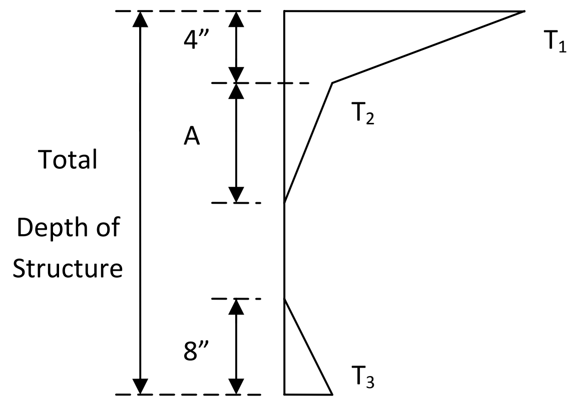


Figure 16. Positive Temperature Gradient through Cross-Section

The United States is divided into four zones based on climate. Based on Figure 3.12.3-1 in the AASHTO specifications, Virginia is in Zone 3. Table 3.12.3-1 in the AASHTO specifications shows that the temperatures associated with Zone 3 are:

$T_1 = 41°F$
$T_2 = 11°F$.

Section 3.12.3 in AASHTO states that T_3 should be taken as 0°F unless a site study indicates otherwise, and the maximum value that can be used for T_3 is 5°F. For the analysis in this project, T_3 will be taken to be 0°F since no other data is available. Section 3.12.3 also defines the value of the dimension A in Figure 16 as 12.0 in for concrete superstructures that have a depth of 16 in or more. All of the PCBT beams considered in this study fall into this category.

Thermal Moment

The thermal effects on a concrete bridge must be taken into account (Imbsen, et al. 1985). The method suggested in the AASHTO LRFD Bridge Design Specifications states that the equation to calculate the axial force in a fully restrained system due to thermal effects is:

$$P = \int_Y E \cdot \alpha \cdot T(Y) \cdot b(Y)dy = \int_Y \sigma(Y) \cdot b(Y)dy \tag{36}$$

where

E = elastic modulus
α = coefficient of thermal expansion
T(Y) = temperature at depth Y
b(Y) = net section width at height Y
σ(Y) = longitudinal stress at a fiber located a distance Y from the center of gravity of the cross-section
P = restraining axial force.

Likewise, the moment caused by the thermal forces in a fully restrained system is also defined in NCHRP 276 and is as follows:

$$M = \int_Y E \cdot \alpha \cdot T(Y) \cdot b(Y) \cdot Ydy = \int_Y \sigma(Y) \cdot b(Y) \cdot Ydy \qquad (37)$$

Figure 17 illustrates the AASHTO LRFD Zone 3 thermal gradient being applied to the PCBT beam.

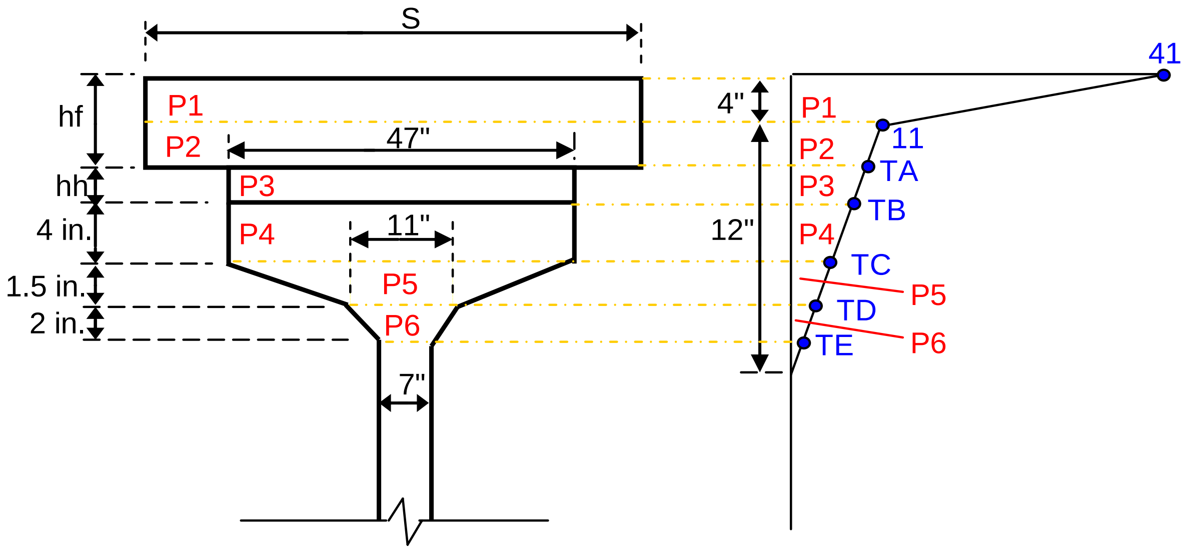


Figure 17. Thermal Forces in PCBT Beams

Note that the top and bottom sections of all PCBT beams are the same, so the difference between beam sizes is in web heights. Therefore, the top part of the cross-section shown in Figure 17 is valid for all PCBT beam sizes. The negative thermal gradient is not considered because it causes beneficial negative moments over interior supports. The design example in Appendix C of Koch (2008) presents sample calculations of forces P1 through P6 and for temperatures TA through TE.

In addition, notice that assumptions were made in the model regarding the thickness of the deck and the haunch. Calculations were based on the assumption that the thickness of the deck is at least 4 in, so the temperature of 11°F is acting in the deck. This is a valid assumption because the VDOT standard specification design aid states that the thickness of the deck should range between 7.5 in and 8.5 in (VDOT 2005). In addition, the depth of the deck and haunch together must be at least 8.5 in, so that the temperature gradient reaches 0°F at or before the point labeled "TE" in Figure 17. This is a valid assumption because the height of the haunch can be assumed to be at least 1 in in all cases.

It is important to note that the restraint moment at the interior support is 1.5 times the moment that acts on the end of a beam (Figure 18).

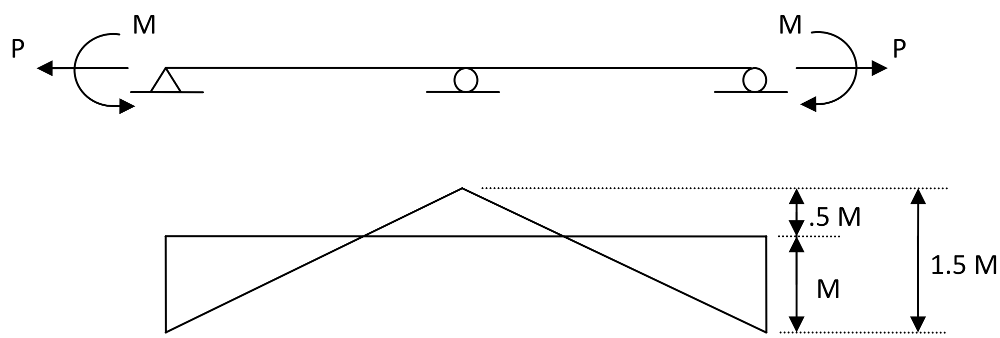


Figure 18. Thermal Restraint Moments

Dead Load on Composite System

VDOT designs are based on a specified composite dead load, which is added to a bridge after it is made continuous and composite. The dead load varies from 0.27 k/ft for a 6 ft beam spacing, to 0.32 k/ft for a 10 ft beam spacing. This dead load includes such things as an overlay and barrier rails. The diaphragm moment resulting from this uniformly distributed moment was calculated with a simple elastic analysis of a two-span beam.

Analytical Study

Once the methods needed to compute each of the contributing moments in the diaphragms were established, a parametric study was undertaken. A wide variety of combinations of beam size, beam spacing, beam strength and span length were analyzed as described in this section.

Cases Analyzed

For each PCBT beam size, the following cases were analyzed:

- 6 ksi compressive strength, average span length, and narrowest beam spacing
- 6 ksi compressive strength, long span length, and narrowest beam spacing
- 8 ksi compressive strength, average span length, and narrowest beam spacing
- 8 ksi compressive strength, long span length, and narrowest beam spacing
- 6 ksi compressive strength, average span length, and widest beam spacing
- 6 ksi compressive strength, long span length, and widest beam spacing
- 8 ksi compressive strength, average span length, and widest beam spacing
- 8 ksi compressive strength, long span length, and widest beam spacing.

These cases were selected because they are considered to be the extreme cases, so they provide bounds for the typical design parameters. The "long span length" was the greatest span length according to the VDOT design aid for a particular beam with a specified compressive strength and beam spacing, and the "average span length" was considered to be about 20 to 25 ft shorter than the greatest span length. Likewise, the "narrowest beam spacing" was considered to be the smallest beam spacing available on the VDOT design aid for a particular beam size, beam spacing, and compressive strength, while the “widest beam spacing” was the largest beam spacing available.

MathCAD Spreadsheet

The following is a brief description of the calculations that are performed in the MathCAD spreadsheet presented in Appendix C of Koch (2008):

Input:

Beam and deck properties
Span length
Strand pattern (including number of strands harped)
Live loads from QConBridge

Intermediate Calculations:

Calculate transformed cross-sectional properties
Check allowable stresses at transfer
Calculate creep and shrinkage coefficients
    From initial time to deck placement
    From deck placement to end of service
Calculate prestress loss
    From initial time to deck placement
    From deck placement to end of service
Check stresses at deck placement
Calculate composite cross-sectional properties
Calculate cracking moment of diaphragm

Time-dependent Calculations:

Calculate dead load creep rotation
Find the moment to restrain the dead load creep rotation
Calculate the end rotation due to prestress
Find the moment to restrain prestress rotations
Subtract effects of rotation from transfer to deck placement and calculate reduced restraint moment
Calculate the diaphragm restraint moment due to loss of prestress
Calculate differential shrinkage restraint moment
Find the total time-dependent moment in the diaphragm

Final Calculations:

Check stresses at end of service
Calculate the thermal restraint moment
Calculate the nominal moment capacity of the diaphragm

Check if the AASHTO requirements are satisfied
Check the flexural strength of the diaphragm.

RESULTS

Beams Older than 90 days

The objective of this part of the research project was to determine how many prestressing strands or additional mild reinforcing bars must be extended into diaphragms so that the factored nominal strength is greater than 1.2 times the cracking moment for beams older than 90 days. A certain amount of strength is obtained from the detail developed by Newhouse, so the additional strength, if required, must come from the prestressing strands or the additional mild steel.

Consider, for example, the PCBT-45 shown in Figure 19. Notice that the design strength is greater than 1.2 times the cracking moment for all cases. So, for this beam size, no additional reinforcement is required.

It is important to note the slight variations in the nominal moment and cracking moment for the PCBT-45 beam in Figure 19. The span length and the beam spacing affect the effective width of the deck ("b_e" in Figure 5). Therefore, the effective deck width is the only variable that changes in the calculation of the nominal moment and cracking moment of the continuity diaphragm. Figures similar to Figure 19, which present results for other beams, can be found in Appendix B.

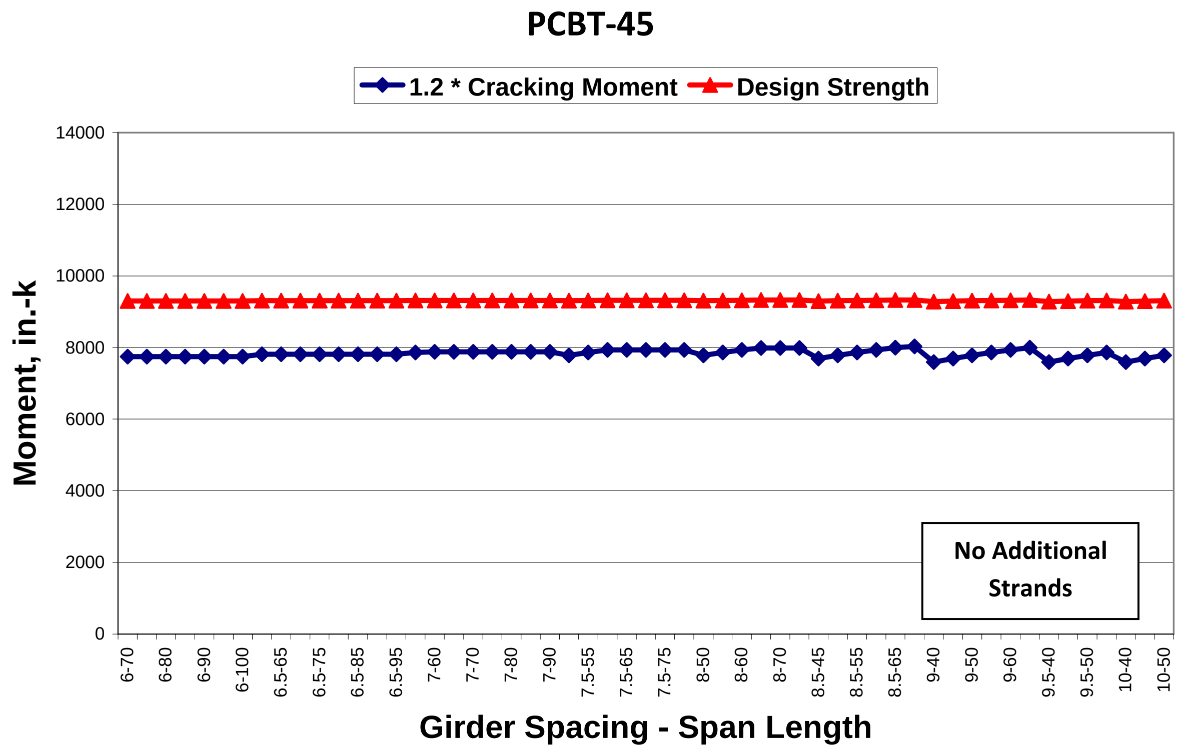


Figure 19. Cracking Moment and Design Strength for PCBT-45 Beam

This process was repeated for each PCBT beam for a variety of span lengths and beam spacings. The following cases were investigated:

- For the PCBT-29: 10 cases with a beam spacing varying from 6 ft to 8 ft and a span length varying from 40 ft to 60 ft
- For the PCBT-37: 29 cases with a beam spacing varying from 6 ft to 10 ft and a span length varying from 40 ft to 80 ft
- For the PCBT-45: 55 cases with a beam spacing varying from 6 ft to 10 ft and a span length varying from 40 ft to 100 ft
- For the PCBT-53: 68 cases with a beam spacing varying from 6 ft to 10 ft and a span length varying from 40 ft to 115 ft
- For the PCBT-61: 70 cases with a beam spacing varying from 6 ft to 10 ft and a span length varying from 50 ft to 125 ft
- For the PCBT-69: 66 cases with a beam spacing varying from 6 ft to 10 ft and a span length varying from 60 ft to 135 ft
- For the PCBT-77: 60 cases with a beam spacing varying from 6 ft to 10 ft and a span length varying from 80 ft to 145 ft
- For the PCBT-85: 60 cases with a beam spacing varying from 6 ft to 10 ft and a span length varying from 95 ft to 150 ft
- For the PCBT-93: 59 cases with a beam spacing varying from 6 ft to 10 ft and a span length varying from 100 ft to 160 ft

Required Number of Additional Strands in Diaphragm

For cases for which bars alone were not adequate, the numbers of strands were adjusted until the design strength was at least 1.2 times the cracking moment for all cases. Table 8 presents the results.

Table 8. Bent Strands Required and Recommended for PCBT Beams

Beam	No. of Bent Strands Required	No. of Bent Strands Recommended
PCBT-29	0	0
PCBT-37	0	0
PCBT-45	0	0
PCBT-53	0	0
PCBT-61	0	0
PCBT-69	1	0
PCBT-77	1	2
PCBT-85	2	2
PCBT-93	2	2

Note that the PCBT-69 beam has nearly sufficient moment capacity beyond the cracking moment without any bent strands. In fact, the design strength was 1.17 times the cracking moment, instead of the needed 1.2, for the worst case analyzed for this beam without bent strands. Therefore, it is recommended that no additional strands be used because it is highly likely that this detail is adequate without extended strands, but the decision to add strands should be left to the discretion of the designer. The extended strand detail is shown in Figure 20.

There is another option if extending prestressing strands into the diaphragm is not desirable. Additional mild steel could be used for the continuity diaphragm reinforcement. Using five, rather than four, No. 6 bars bent at a 180° angle would provide adequate capacity for the diaphragm for all cases. One standard could be used for all sizes, or the larger number of bars could be used only for the beams larger than PCBT-69. This detail is presented in Figure 21.

A final option is presented in Figure 22. In this case, the original four hairpin bars are left unchanged, but two additional No. 5 L-shaped bars are extended from the upper part of the bottom flange. These two L-shaped bars provide the additional capacity required for the beams larger than PCBT-69.

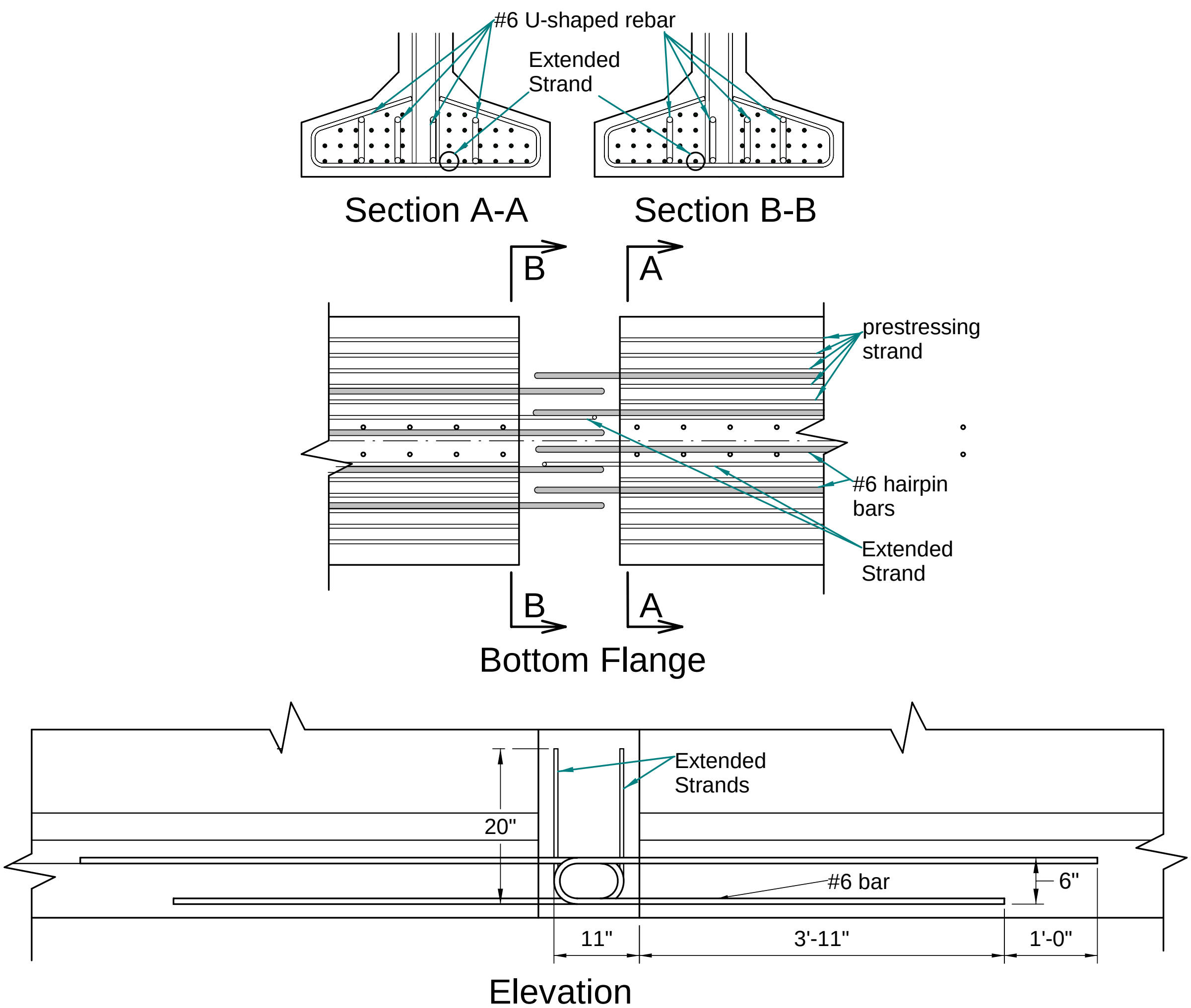


Figure 20. Modified Detail with Two Extended Prestressing Strands

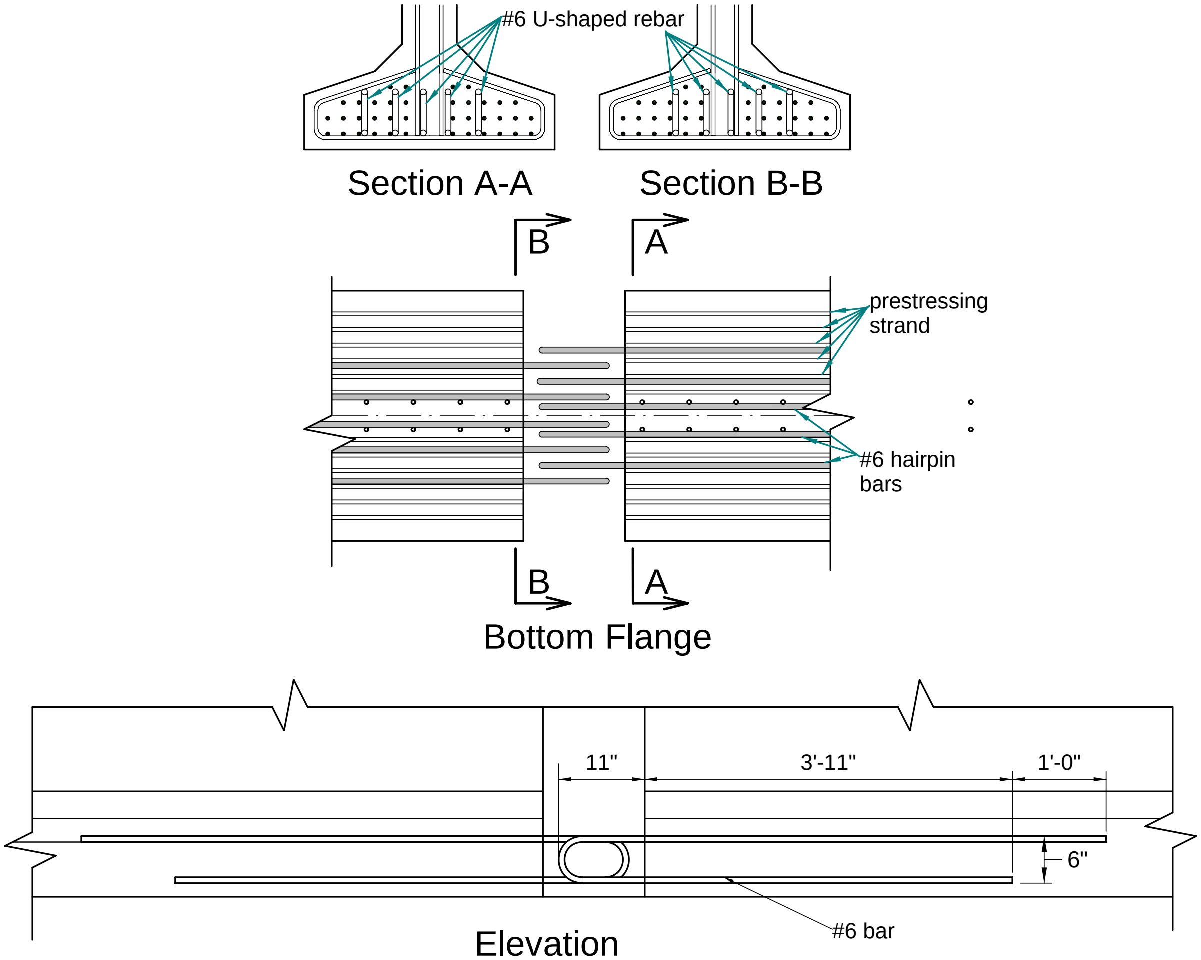


Figure 21. Detail with Five No. 6 Hairpin Bars in Each Beam End

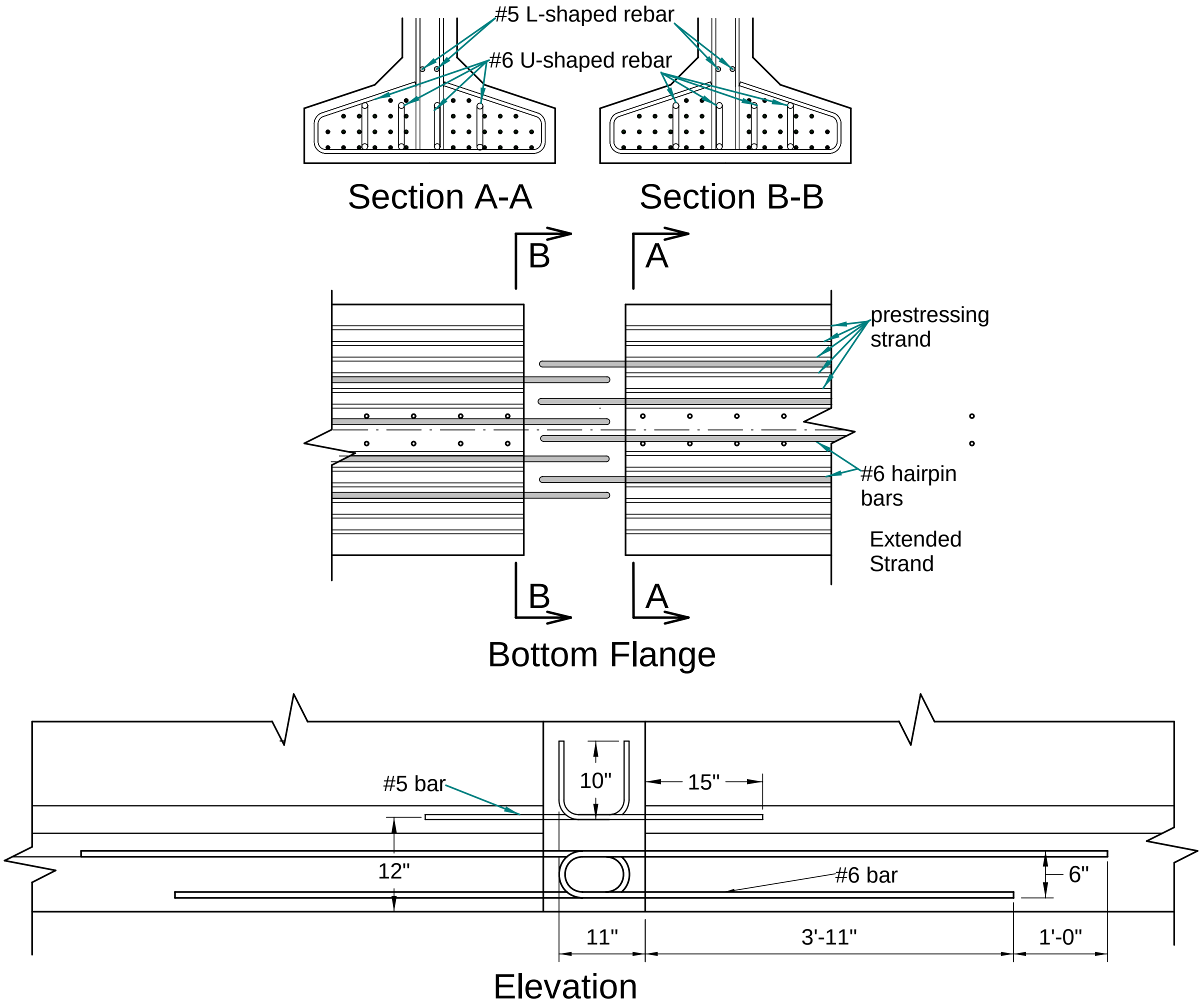


Figure 22. Modified Detail with L-shaped No. 5 Bars High in Bottom Bulb

Beams Younger than 90 days

A comprehensive MathCAD spreadsheet was developed to perform all calculations. See Appendix C of Koch (2008) for the spreadsheet of a two span system. The three-span system is shown in Appendix D of Koch. The highlighted fields in the Appendices represent input values that can be adjusted to represent any situation. Results from runs of the MathCAD sheet were recorded in an EXCEL spreadsheet for further analysis. The objective was to determine how many days each PCBT beam must be stored so that positive time-dependent moments will not develop in the continuity diaphragm.

Interpreting Results

Figure 23 is a plot of the individual components of the restraint moment in the diaphragm for an average analysis case. These components include the time-dependent, dead load on composite, live load, and thermal restraint moments. The example presented in Figure 23 is a PCBT-61 beam with 8 ksi compressive strength concrete, a wide beam spacing (10 ft), and a relatively short span length (60 ft) for the given beam size. As specified by AASHTO, the modified total does not include the contribution of the time-dependent restraint moment if it is negative, but does include it if it is positive. Figure 23 illustrates how the time dependent restraint moments change depending on the number of days the beams are stored before they are made continuous and composite. For beams which are very young at the time of continuity, prestress creep dominates and positive moments develop in the diaphragm. For beams which are older at the time continuity is created, differential shrinkage dominates and negative moments develop in the diaphragm. For other cases that were analyzed in this study, the magnitudes of the moments in Figure 23 are different, but the general patterns are similar.

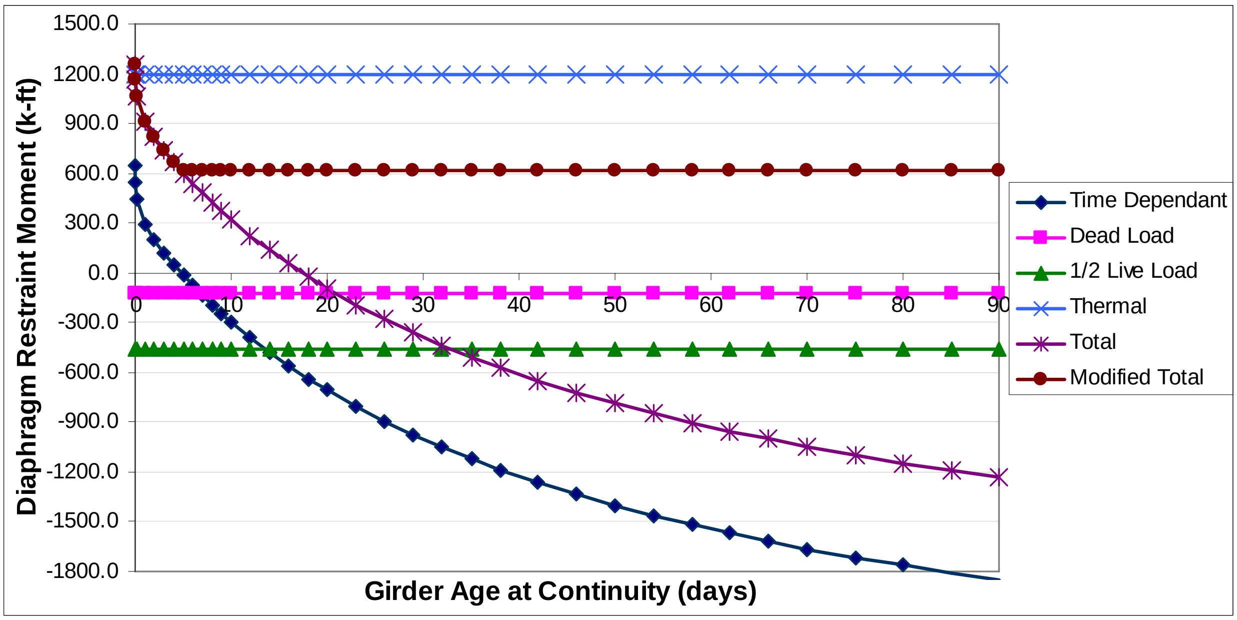


Figure 23. Continuity Diaphragm Restraint Moments

Note the magnitude of the positive thermal restraint moment compared to the dead load and live load restraint moment. The positive thermal moment is greater than the sum of the composite dead load and half of the life load. Also notice that that the modified total moment, which does not include the contribution of the time-dependent moment when it is negative, is never negative for this example. This means that with the AASHTO requirement that helpful time dependent moments be ignored, the total moment will always be positive, even for an infinite number of storage days. Therefore, the beams must be stored 90 days. When the helpful (negative) contribution of the time-dependent moment is considered, the minimum number of storage days for this example is found to be 18 days. This is because the total time-dependent moment will just become negative for this storage duration.

All Results

The previously discussed process of determining the minimum number of storage days was repeated for all of the different cases. They are shown in Tables 9 and 10. The highlighted cases in Tables 9 and 10 are those that failed the AASHTO requirement, which excludes the contribution of the time-dependent moment if it is negative. Therefore, the highlighted cases would need to be stored 90 days since the total moment in the diaphragm will never be negative (because the effects due to negative time-dependent moments must be ignored). There are many of these cases.

Table 9. Experimental Results, PCBT-29 to PCBT 53

Beam Size (in)	Beam f'_c (ksi)	Span Len. (ft)	Deck Space (ft)	Beam Age 2-Span (day)
29	6	Mid (40)	6	45
29	6	Long (45)	6	46
29	8	Mid (40)	6	19
29	8	Long (60)	6	16
37	6	Mid (40)	6	52
37	6	Long (60)	6	30
37	8	Mid (55)	6	13
37	8	Long (80)	6	17
37	6	Mid (40)	7.5	48
37	6	Long (45)	7.5	51
37	8	Mid (40)	7.5	26
37	8	Long (60)	7.5	20
45	6	Mid (60)	6	31
45	6	Long (85)	6	30
45	8	Mid (75)	6	7
45	8	Long (100)	6	20
45	6	Mid (40)	9	62
45	6	Long (45)	9	53
45	8	Mid (40)	9	32
45	8	Long (65)	9	21
53	6	Mid (80)	6	24
53	6	Long (105)	6	40
53	8	Mid (90)	6	6
53	8	Long (115)	6	18
53	6	Mid (40)	10	66
53	6	Long (45)	10	55
53	8	Mid (50)	10	25
53	8	Long (75)	10	20

Table 10. Experimental Results, PCBT-61 to PCBT 93

Beam Size (in)	Beam f'_c (ksi)	Span Len. (ft)	Deck Space (ft)	Beam Age 2-Span (day)
61	6	Mid (95)	6	22
61	6	Long (120)	6	36
61	8	Mid (100)	6	6
61	8	Long (125)	6	13
61	6	Mid (40)	10	67
61	6	Long (60)	10	43
61	8	Mid (60)	10	18
61	8	Long (85)	10	16
69	6	Mid (105)	6	16
69	6	Long (130)	6	37
69	8	Mid (110)	6	4
69	8	Long (150)	6	5
69	6	Mid (60)	10	46
69	6	Long (85)	10	37
69	8	Mid (75)	10	16
69	8	Long (100)	10	16
77	6	Mid (110)	6	15
77	6	Long (135)	6	29
77	8	Mid (115)	6	2
77	8	Long (140)	6	9
77	6	Mid (65)	10	43
77	6	Long (90)	10	35
77	8	Mid (90)	10	14
77	8	Long (115)	10	18
85	6	Mid (120)	6	14
85	6	Long (145)	6	27
85	8	Mid (125)	6	1
85	8	Long (150)	6	7
85	6	Mid (85)	10	34
85	6	Long (110)	10	38
85	8	Mid (105)	10	13
85	8	Long (130)	10	17
93	6	Mid (130)	6	10
93	6	Long (155)	6	24
93	8	Mid (130)	6	1
93	8	Long (155)	6	6
93	6	Mid (95)	10	37
93	6	Long (120)	10	36
93	8	Mid (115)	10	12
93	8	Long (140)	10	15

General Trends

There are general trends among the results. It is important to recognize and consider these patterns so that continuity diaphragm restraint moments can be better understood.

Changes in Length

No comprehensive conclusions can be drawn to compare the change that occurs in the total restraint moment due to changes in the length of the member. In some cases, moving from an average span length to a long span length required a longer storage period for the beams, and in others a shorter storage period resulted. It is difficult to directly determine the effects of adjusting the span length on the total diaphragm restraint moment, because the span length affects all of the individual components of the total restraint moment in different ways. However, the following observations can be noted as span lengths increase for a given number of storage days:

- The thermal moment will not change
- The live load moment will become more negative
- The dead load moment will become more negative
- The time-dependent moment could become either more positive or negative depending on the magnitudes of the changes in the following components:
  The moment to restrain the dead load creep rotation will become more negative.
  The moment to restrain prestress creep rotations will become more positive.
  The moment to restrain the creep of prestress losses will become more positive.
  The differential shrinkage restraint moment will not change.

Changes in Compressive Strength

Analysis of the results showed that there was a decrease in the minimum number of storage days when the beam compressive strength was increased from 6 ksi to 8 ksi. The average decrease in the minimum storage days was 65.5%. The largest reduction occurred in the cases consisting of the medium span lengths with the closer beam spacing, which was an average decrease of 80.6% for the cases tested. The smallest reduction occurred in the cases consisting of the long span lengths with the wide beam spacing, which was a decrease of 59.2%.

The decrease in the minimum number of storage days for beams as compressive strength increases is due to several factors. First of all, as the compressive strength increases, there are changes in the transformed area and in the centroid of the transformed area. This will have numerous effects on the calculation of total restraint moment. Also, the ultimate creep and shrinkage will decrease as the concrete strength increases. Therefore, there is a decrease in the minimum number of days that a beam needs to be stored so that the remaining time-dependent effects will not cause positive moments in the diaphragm.

Changes in Beam Spacing

The results of the analyzed cases show that there was a 137% average increase in the number of days that the beams needed to be stored so that a positive moment would not develop in the diaphragm when the beam spacing increases from the close to the wide spacing. A larger compressive strength results in a greater degree of change when the beam spacing is adjusted. Also, a more average span length results in greater changes as well. Hence, there is only a moderate (24%) increase in the minimum number of storage days when the beam spacing is changed from close to wide for the cases consisting of the 6 ksi compressive strength with the long span lengths

The beam spacing has several effects on the continuity diaphragm restraint moments. First of all, the dead load moment of the structure in the continuity diaphragm will become more negative as the deck width increases. Likewise, the live load moment in the continuity diaphragm will also become more negative as the deck width increases because the beam distribution factors will increase. Also, differential shrinkage between the deck and the beams increases because a larger volume of the deck results in a greater shrinkage force in the deck. Most importantly, the thermal gradient that was used in this analysis has a very large component that is applied to the top of the deck. So, increasing the width of the deck substantially increases the positive thermal restraint moment in the continuity diaphragm that needs to be counteracted by the other moments.

Two-Spans vs. Three-Spans

Although it was previously assumed that two-span cases are critical, three-span systems must be further explored because time-dependent factors are interdependent. Appendix D of Koch (2008) shows a comparison of the calculated terms that change between the two-span and three-span systems. When moving from a two-span case to a three-span case, the composite dead load restraint moment becomes less negative, the live load restraint moment becomes less negative, the thermal restraint moment becomes less positive, the non-composite dead load restraint moment becomes less negative, the prestress restraint moment becomes less positive, and the differential shrinkage moment becomes less negative. Several tests were run on about half of the PCBT beams to compare the two-span and the three span-cases. The results are shown in Table 11.

The two-span systems are critical for all cases, as shown in Table 11. This means that the PCBT beams need to be stored for fewer days before establishing continuity so that a negative diaphragm moment will result for three-span systems. Therefore, it is a valid assumption that the two-span cases are critical and are the only ones that need to be computed. As in the previous tables, highlighted cells indicate cases for which beams must be stored for 90 days.

Table 11. Three-Span Systems vs. Two-Span Systems: Minimum Storage Duration

Beam Size (in)	Beam f'_c (ksi)	Span Len. (ft)	Deck Space (ft)	Beam Age 2-Span (day)	Beam Age 3-Span (day)
29	6	Mid (40)	6	45	39
29	6	Long (45)	6	46	40
29	8	Mid (40)	6	19	16
29	8	Long (60)	6	16	14
45	6	Mid (60)	6	31	28
45	6	Long (85)	6	30	23
45	8	Mid (75)	6	7	6
45	8	Long (100)	6	20	12
45	6	Mid (40)	9	62	55
45	6	Long (45)	9	53	47
45	8	Mid (40)	9	32	29
45	8	Long (65)	9	21	19
61	6	Mid (95)	6	22	19
61	6	Long (120)	6	36	23
61	8	Mid (100)	6	6	4
61	8	Long (125)	6	13	7
61	6	Mid (40)	10	67	61
61	6	Long (60)	10	43	39
61	8	Mid (60)	10	18	16
61	8	Long (85)	10	16	4
77	6	Mid (110)	6	15	12
77	6	Long (135)	6	29	20
77	8	Mid (115)	6	2	1
77	8	Long (140)	6	9	7
77	6	Mid (65)	10	43	39
77	6	Long (90)	10	35	31
77	8	Mid (90)	10	14	12
77	8	Long (115)	10	18	18
93	6	Mid (130)	6	10	7
93	6	Long (155)	6	24	16
93	8	Mid (130)	6	1	1
93	8	Long (155)	6	6	4
93	6	Mid (95)	10	37	33
93	6	Long (120)	10	36	32
93	8	Mid (115)	10	12	10
93	8	Long (140)	10	15	13

CONCLUSIONS

Several conclusions can be drawn from the research presented in this document that will assist in the design of continuity diaphragms for PCBT beams.

Beams Older than 90 Days

A study was undertaken to determine how many bent strands or mild reinforcing bars need to be extended into continuity diaphragms, in addition to Newhouse's standard detail (Figure 4), to provide sufficient moment capacity for PCBT beams. The applicable AASHTO LRFD article requires, for beams that are older than 90 days at the time continuity is established, that the factored nominal moment of the diaphragm be greater than or equal to 1.2 times the cracking moment. Several beam spacings and span lengths were considered for each beam size.

It was concluded that no additional strands are required for the PCBT-29, PCBT-37, PCBT-45, PCBT-53, and PCBT-61. For the PCBT-69, the design strength was 1.17 times the cracking moment for the worst case analyzed without bent strands, instead of the required 1.2. Therefore, it is highly likely that this detail is adequate without extended strands, but the decision to add strands should be left to the discretion of the designer. The PCBT-77, PCBT-85, and the PCBT-93 require additional reinforcement in the diaphragm. This can be provided with any of three details: two additional bent strands extended into the continuity diaphragm, one additional No. 6 hairpin bar, or two L-shaped No. 5 bars extending into the diaphragm from high in the bottom bulb. These details are presented in Figures 20, 21 and 22.

Beams Younger than 90 Days

Testing the PCA Method

The stresses throughout the cross-section of a composite bridge system were computed using the PCA method and then were compared to those found using the separate sections method (or the Trost-Menn method) to determine accuracy. The PCA method generally produced conservative estimates of all stresses, and the beam creep coefficient is always a conservative estimation of the creep coefficient that would give results similar to those obtained using the separate sections method. Also, this method is fairly accurate when the beam and the deck creep coefficients are the same, or similar. However, more error is introduced as the creep coefficients become more different from each other, but the method is still overall conservative. Therefore, it has been concluded that the PCA method adequately predicts the restraint moments that develop in continuity diaphragms due to time-dependent effects.

Check for Compliance with AASHTO Specifications

The AASHTO LRFD Bridge Design Specifications were used to analyze PCBT beams that are younger than 90 days when continuity is established. AASHTO states that if a beam is stored for less than 90 days before being erected, the calculated stress at the bottom of the continuity diaphragm for the combination of superimposed permanent load, settlement, creep, shrinkage, 50 percent live load and temperature gradient, if applicable, must be compressive. In addition, the time-dependent moment must be considered if it is positive because it produces a more critical result, but it should be ignored if it is negative. The goal of this aspect of the research was to determine the minimum number of days that PCBT beams need to be stored so that the AASHTO specifications are met.

If the designer is willing to perform a somewhat involved analysis, the number of beam storage days can be reduced significantly below 90 days about half of the time. For the other half of the cases, the total moment in the diaphragm will never become negative because the contributions of the negative time-dependent moment must be ignored. For all of these cases, the beams must be stored for 90 days.

If the negative time-dependent restraint moments are considered, the largest minimum number of storage days for beams with a compressive strength of 6 ksi is 67 days and the largest minimum number of storage days for beams with a compressive strength of 8 ksi is 32 days. Note that including the negative time-dependent restraint moments in analysis of beams with a compressive strength of 8 ksi cause a substantial reduction (32 days from 90 days) in the greatest minimum storage duration. In general, narrower beam spacing and higher concrete compressive strength results in shorter required storage duration.

A quick check can be done to see if beams must be stored 90 days. If the sum of the thermal, composite dead load, and half of the live load restraint moments is positive, then the beam must be stored 90 days. This is true because the only restraint moment that varies with storage duration is the time-dependent moment, which must be ignored if it is negative. If the sum of the thermal, composite dead load, and half of the live load restraint moments is negative, then it is very likely that the beam will be able to be stored for less than 90 days. In this case, it would be beneficial to calculate the time-dependent restraint moment and determine the minimum number of storage days that would result in a negative diaphragm restraint moment.

It was found that if negative time-dependent moments are considered, 67 days is the longest that a PCBT beam needs to be stored for the cases analyzed. However, half of the cases must be stored for 90 days if the negative time-dependent effects are ignored. The factors that seemed to contribute to the necessary 90 day storage time were average span lengths, wide beam spacings, and lower compressive strengths.

If negative time-dependent moments are considered, there are general trends in the data. Analysis of the results showed that there was a decrease in the minimum number of storage days when the beam compressive strength was increased from 6 ksi to 8 ksi and when the beam spacing decreased from the widest spacing to the closest spacing. However, no definite conclusions could be drawn to compare the change that occurs in the total restraint moment due to changes in the length of the member.

Three-span systems were also considered, but the two-span systems were found to be critical for all of the cases in this study. In other words, the PCBT beams need to be stored for fewer days before establishing continuity (so that a negative diaphragm moment will result) if they are used in three-span systems rather than two-span systems. This is true if negative time-dependent moments are considered or if they are ignored.

The above discussions are based on the assumption that the designer would like to consider the bridge to behave as continuous under super-imposed dead loads and live loads. This can be advantageous and economical, because a continuous system carries loads more efficiently, and a savings can be realized in the number of strands required for positive mid-span moments. However, the designer has the option to consider the diaphragm to be ineffective, and design every span as if it were simply supported. In this case, it is recommended that the same detail be used, because it will be effective in controlling cracking, if cracks develop in the diaphragm. Cracking may become a maintenance issue over the life of the bridge, but the loss in stiffness, and hence the degree of continuity, will not be a problem. It is also wise to design the deck reinforcement as if the bridge were fully continuous for super-imposed dead loads and live loads. This will control cracking that may develop in the deck due to negative moments over the piers.

RECOMMENDATIONS

Recommendations for Diaphragm Design

1. *When a project's time frame allows it, VDOT's Structure & Bridge Division should require that beams in bridges made continuous for live load be stored for 90 days prior to erection.* If the beams are stored for 90 days, the bridge can be designed as fully continuous for super-imposed dead load and live load.

2. *When PCBT-29 through 69 beams are stored for 90 days, VDOT's Structure & Bridge Division should use the original detail recommended in Newhouse (Figure 4) for the positive moment reinforcement in the continuity diaphragm.*

3. *When PCBT -77 through 93 beams are stored for 90 days, VDOT's Structure & Bridge Division should use one of the details recommended in Figures 20, 21, or 22.*

4. *To perform a preliminary check to determine if a beam can be stored for less than 90 days, VDOT's Structure & Bridge Division engineers should calculate and sum the thermal restraint moment, half of live load moment, and moment from composite dead load. If this sum is positive, the beam must be stored 90 days. If the sum is negative, further calculations can be performed to determine minimum storage days.*

5. *VDOT Structure & Bridge Division engineers should use the prepared Mathcad sheet to determine number of storage days.*

6. *In the case where 90 day storage is not feasible, the previously noted details are still recommended for use. However, VDOT's Structure & Bridge Division should design the bridge for all loads as if it were a series of simple spans. The deck reinforcement should be designed as if the bridge were fully continuous for super-imposed dead loads and live loads.*

If this design method is used, cracks in the diaphragm, where the beams frame in, may need to be sealed to prevent maintenance problems.

RECOMMENDATIONS FOR FUTURE WORK

The results of this research produce additional questions. First of all, is there a more accurate creep coefficient that can be used in place of the beam creep coefficient when determining stresses in composite systems using the PCA method? This study showed that although the PCA method was almost always conservative, the beam creep coefficient was not particularly accurate. This is especially true if the beam and deck coefficient were not similar. More work is needed to determine if the beam creep coefficient can be simply adjusted for use in the PCA method to give more accurate results. Also, additional aging coefficients should be considered. This study only considers an aging coefficient of 0.8, which could have a significant impact on the appropriate creep coefficient that should be used in the PCA method.

Additionally, further research regarding the thermal restraint moment would be beneficial. Although a standard thermal gradient exists for Virginia, experimental testing could improve upon the existing design values. This research illustrated that the thermal restraint moment calculated using the AASHTO thermal gradient was dominant in the design of continuity diaphragm. Therefore, VDOT should consider evaluating a thermal gradient specifically for Virginia to assure that the AASHTO gradient is appropriate. Installing thermocouples into new bridge systems would allow researchers to see how temperature changes through the life of the structure due to environmental factors. Also, attaching strain gauges to a concrete beam that is part of a bridge would provide insight as to how temperature affects strain due to the surrounding temperatures.

Finally, additional design parameters could also be varied for PCBT beams that are less than 90 days old before they are made composite and continuous. Only different values for the span lengths, beam spacings, and beam compressive strengths were considered for the PCBT beams younger than 90 days. Other design parameters that could be varied to provide a more comprehensive understanding of the results include the age of beam at transfer of prestressing, haunch height, and deck concrete compressive strength. In addition, a sensitivity analysis could be performed on the aging coefficient to determine the magnitude of change that would result from adjusting this parameter.

COSTS AND BENEFITS ASSESSMENT

The use of precast prestressed bulb-T beams made continuous for live load has many benefits. As stated in the Introduction section, with a diaphragm between beam ends and the slab cast continuously over interior supports, costly and high-maintenance joints can be eliminated. With the elimination of the joints, and the reduction in long-term cambers afforded by the diaphragm, the ride quality of the deck is greatly improved. It is possible to reduce the number of prestressing strands required for the beams, because the live loads and composite dead loads are carried as if the beams were continuous over several supports. This results in lower mid-span moments, and hence fewer required strands. Another benefit to using diaphragms is that a continuous system has redundant load paths, so a continuous bridge is more likely to survive an extreme event without complete collapse.

This report presented the designer with four options in the design of multi-span bridges with PCBT beams:

1. Design each span as simply supported, provide no diaphragm, and provide a joint in the deck over each support.

2. Design each span as simply supported, provide a diaphragm using the recommended details, and provide negative moment reinforcement in the deck. This option has no storage time requirements.

3. Design the spans as simply supported for self-weight and deck weight, but fully continuous for super-imposed dead load and live load, store the beams for 90 days, use the recommended diaphragm details and provide negative moment reinforcement in the deck.

4. Design the spans as simply supported for self-weight and deck weight, but fully continuous for super-imposed dead load and live load, store the beams for less than 90 days as allowed by the results of a full time-dependent analysis, use the recommended diaphragm details and provide negative moment reinforcement in the deck.

Table 12 presents the costs specific to each solution.

Table 12. Cost Differentials for Various Solutions

Alternative	Cost Differentials
1 – Simple Span with Joints	joints, joint maintenance, sub-structure maintenance from leaking joints, prestressing as required for simple spans (more)
2 – Simple Span with Diaphragms	Diaphragms (concrete, reinforcement, formwork, construction time), additional reinforcement in beams, additional reinforcement in deck, diaphragm crack sealing if required, prestressing as required for simple spans (more)
3 – Continuous with 90 day storage	Diaphragms, (concrete, reinforcement, formwork, construction time), additional reinforcement in beams, additional reinforcement in deck, storage time, prestressing as required for continuous spans (less)
4 – Continuous with less than 90 days storage	Diaphragms, (concrete, reinforcement, formwork, construction time), additional reinforcement in beams, additional reinforcement in deck, added design costs for time-dependent analysis, prestressing as required for continuous spans (less)

ACKNOWLEDGMENTS

The authors gratefully acknowledge the assistance and guidance provided by Julius Volgyi and John Martin of VDOT's Structure & Bridge Division. Also acknowledged are the contributions of Rodney Davis. The opinions expressed in this document are those of the authors and not necessarily of the sponsor.

REFERENCES

American Association of State Highway and Transportation Officials. (2007). *AASHTO LRFD Bridge Design Specifications*. Fourth Edition with 2008 Interims. Washington, DC.

American Concrete Institute Manual of Concrete Practice. (2002). *Prediction of Creep, Shrinkage, and Temperature Effects in Concrete Structures.* ACI 209R-92. Farmington Hills, MI.

American Concrete Institute Manual of Concrete Practice. (2002). *Report on Factors Affecting Shrinkage and Creep of Hardened Concrete.* ACI 209R-05. Farmington Hills, MI.

Bazant, Z.P. (1975). Theory of Creep and Shrinkage in Concrete Structures: A Precis of Recent Developments. *Mechanics Today,* Vol. 2. American Academy of Mechanics, Pergamon, New York.

British Standards Institution. (1978). *Steel, Concrete and Composite Bridge, Part I, General Statement.* British Standard BS 5400. Crowthorne, Berkshire, U.K.

Dimmerling, A., Miller, R.A., Reid, C., Mirmiran, A., Hastak, M., and Baseheart, T.M. (2005). Connections Between Simply Supported Concrete Beams Made Continuous – Results of NCHRP Project 12-53. *Transportation Research Record: Journal of the Transportation Research Board, No. 1928.* Transportation Research Board of the National Academies, Washington, DC.

Freyermuth, C.L. (1969). Design of Continuous Highway Bridges with Precast, Prestressed Concrete Girders. *Journal of the Prestressed Concrete Institute,* Vol. 14, No 2.

Hognestad, E., Mattock, A.H., Karr, P.H. (1960). Composite Construction for Continuity. *Journal of the Prestressed Concrete Institute,* Vol. 5, No 1.

Imbsen, A., Vandershaf, D.E., Schamber, R.A., and Nutt, R.V. (1985). *Thermal Effects in Concrete Bridge Superstructures.* National Cooperative Highway Research Program Report 276. National Research Council, Washington, DC.

Koch, S. (2008). Prestressed PCBT Girders Made Continuous and Composite with a Cast-in-Place Deck and Diaphragm. Masters Thesis, Virginia Tech, Blacksburg, VA. http://scholar.lib.vt.edu/theses/available/etd-05022008-144755/.

MacGregor, J.G., and Wright, J.K. (2005). *Reinforced Concrete: Mechanics and Design,* 4th Edition. Pearson Prentice Hall, Upper Saddle River, NJ.

Mattock, A.H. (1961). Precast-Prestress Concrete Bridges: 5. Creep and Shrinkage Studies. *Development Department Bulletin D46.* Portland Cement Association, Research and Development Laboratories, Vol. 3, No 2. Stokie, IL.

Mattock, A.H., and Kaar, P.H. (1960). Continuous Precast-Prestressed Concrete Bridges. *Development Department Bulletin D43,* Vol. 2, No. 5. Portland Cement Association, Research and Development Laboratories, Stokie, IL.

Menn, C. (1986*). Prestressed Concrete Bridges,* translated by Gauvreau, P., originally published as Stahlbentonbruken, Springer-Berlag, Wein. Vienna, Austria.

Newhouse, C.D. (2005). Design and Behavior of Prestressed, Precast Girders Made Continuous: An Analytical and Experimental Study. Ph.D. Dissertation, Virginia Tech, Blacksburg.

Nilson, A.H. (1987). Design of Prestressed Concrete, 2nd ed. John Wiley and Sons, New York.

Oesterle, R.G., Glikin, J.D., and Larson, S.C. (1989). *Design of Precast Prestressed Girders Made Continuous.* National Cooperative Highway Research Program Report 322, National Research Council, Washington, DC.

Potgieter, I.C., and Gamble, W.L. (1983). Response of Highway Bridges to Nonlinear Temperature Distributions. Rep. No. FHWA/IL/UI-201, University of Illinois at Urbana-Champaign, IL.

Salmons, J.R. (1974). *End Connections of Pretensioned I-Beam Bridges.* Missouri Cooperative Highway Report 73-5C. Missouri State Highway Department, Jefferson City.

Virginia Department of Transportation. (2006). *Volume V: Part 2: Design Aids: Typical Details.* http://www.vdot.virginia.gov/business/bridge-v5p2.asp.

Washington State Department of Transportation. (2005). Bridge Engineering Software: QConBridges. http://www.wsdot.wa.gov/eesc/bridge/software/index.cfm?fuseaction=software_detail&software_id=48.

APPENDIX A
DESIGN OF CONTINUITY DIAPHRAGMS FOR BEAMS OLDER THAN 90 DAYS

These calculations: PCBT-77 with a beam spacing of 8 ft and a span length of 130 ft

Unit definition:

$k := 1000lb$

$kft := k\cdot ft$

$kin := k\cdot in$

$ksi := \frac{k}{in^2}$

$klf := \frac{k}{ft}$

$pcf := \frac{lb}{ft^3}$

$\mu\varepsilon := .000001$

$psi := \frac{lb}{in^2}$

Variables (PCBT Girder):

$s := 8ft$

$l := 130ft$

$fc := 4ksi$

$h_d := 8in$

$h_h := 1in$

$A_s := 3.52in^2$

$I_b := 788700in^4$

$A_b := 970.7in^2$

$h_b := 77in$

$y_b := 37.67in$

$strands := 1$

$A_{strand} := .153in^2$

Effective width:

$$b := \begin{pmatrix} .125\cdot l \\ 12\cdot h_d + 7in \\ s \end{pmatrix}$$

$b_{eff} := min(b)$

$b_{eff} = 96in$

Gross Area:

$$A_g := A_b + 47in \cdot h_h + b_{eff} \cdot h_d$$

$$A_g = 1.786 \times 10^3 \cdot in^2$$

Gross Centroid:

$$y_g := \frac{A_b \cdot y_b + 47in \cdot h_h \cdot \left(h_b + .5 \cdot h_h\right) + b_{eff} \cdot h_d \cdot \left(h_b + h_h + .5 \cdot h_d\right)}{A_g}$$

$$y_g = 57.784\,in$$

Gross Moment of Inertia:

$$I_g := I_b + A_b \cdot \left(y_g - y_b\right)^2 + \frac{47 \cdot in \cdot h_h^{\,3}}{12} + \left(47 \cdot in \cdot h_h\right) \cdot \left(h_b + .5 \cdot h_h - y_g\right)^2 \ldots$$
$$+ \frac{b_{eff} \cdot h_d^{\,3}}{12} + \left(b_{eff} \cdot h_d\right) \cdot \left(h_b + h_h + .5 \cdot h_d - y_g\right)^2$$

$$I_g = 1.654 \times 10^6 \cdot in^4$$

Modulus of Rupture:

$$f_r := .24 \cdot \left(\sqrt{\frac{fc}{ksi}}\right) \cdot ksi$$

$$f_r = 0.48\,ksi$$

Cracking Moment:

$$M_{cr} := \frac{I_g}{y_g} \cdot f_r$$

$$M_{cr} = 1.374 \times 10^4 \cdot kin$$

1.2 times the Cracking Moment:

$$1.2M_{cr} = 1.649 \times 10^4 \cdot kin$$

Depth of compression block:

$$a := \frac{A_s \cdot 60ksi + A_{strand} \cdot strands \cdot \frac{(30in - 8.25in)}{.163} \frac{k}{in^3}}{.85 \cdot fc \cdot b_{eff}}$$

$a = 0.71 \cdot in$

Effective depth of steel:

$d_s := h_b + h_h + h_d - 4.63in$

$d_s = 81.37 \cdot in$

Effective depth of prestress:

$d_{ps} := h_b + h_h + h_d - 2.25in$

$d_{ps} = 83.75 \cdot in$

Nominal Flexural Stength:

$$M_n := A_s \cdot 60ksi \cdot \left(d_s - \frac{a}{2} \right) + A_{strand} \cdot strands \cdot \frac{(30in - 8.25in)}{.163} \frac{k}{in^3} \cdot \left(d_{ps} - \frac{a}{2} \right)$$

$M_n = 1.881 \times 10^4 \cdot kin$

Design Strength:

$.9 \cdot M_n = 1.693 \times 10^4 \cdot kin$

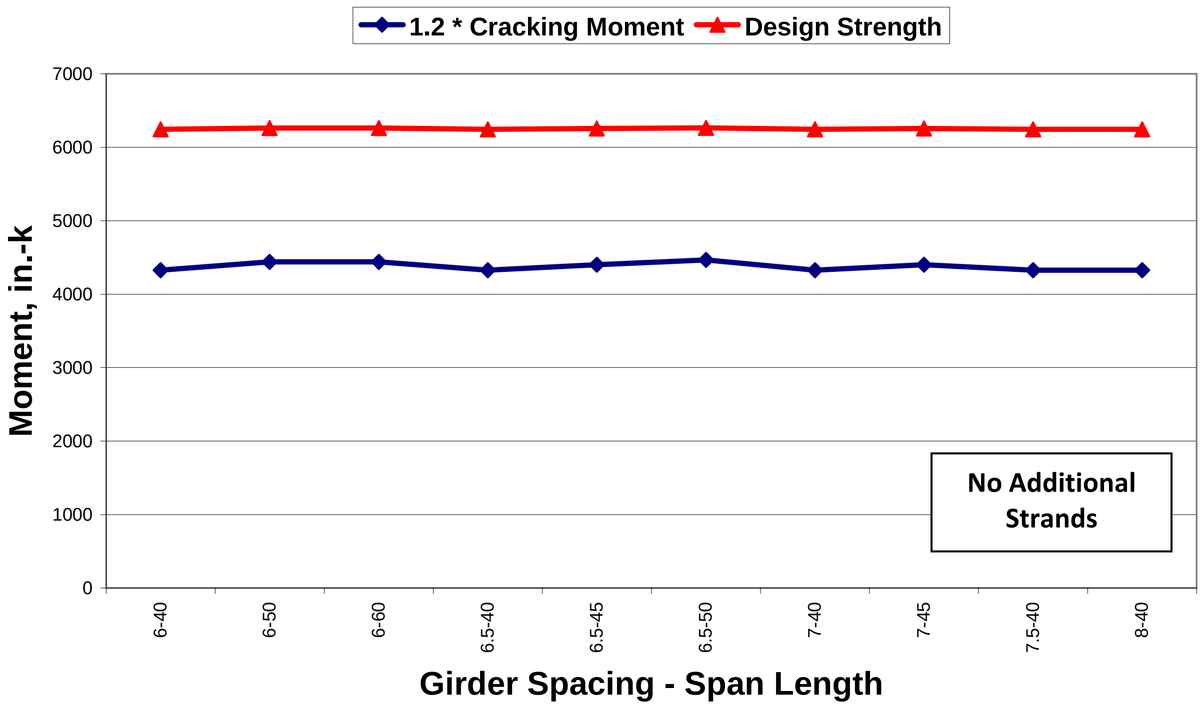
Is diaphragm OK?

$$check := \left| \begin{array}{l} check \leftarrow \text{"NOT OK"} \quad \text{if } M_{cr} > .9M_n \\ check \leftarrow \text{"OK"} \quad \text{if } M_{cr} < .9M_n \end{array} \right.$$

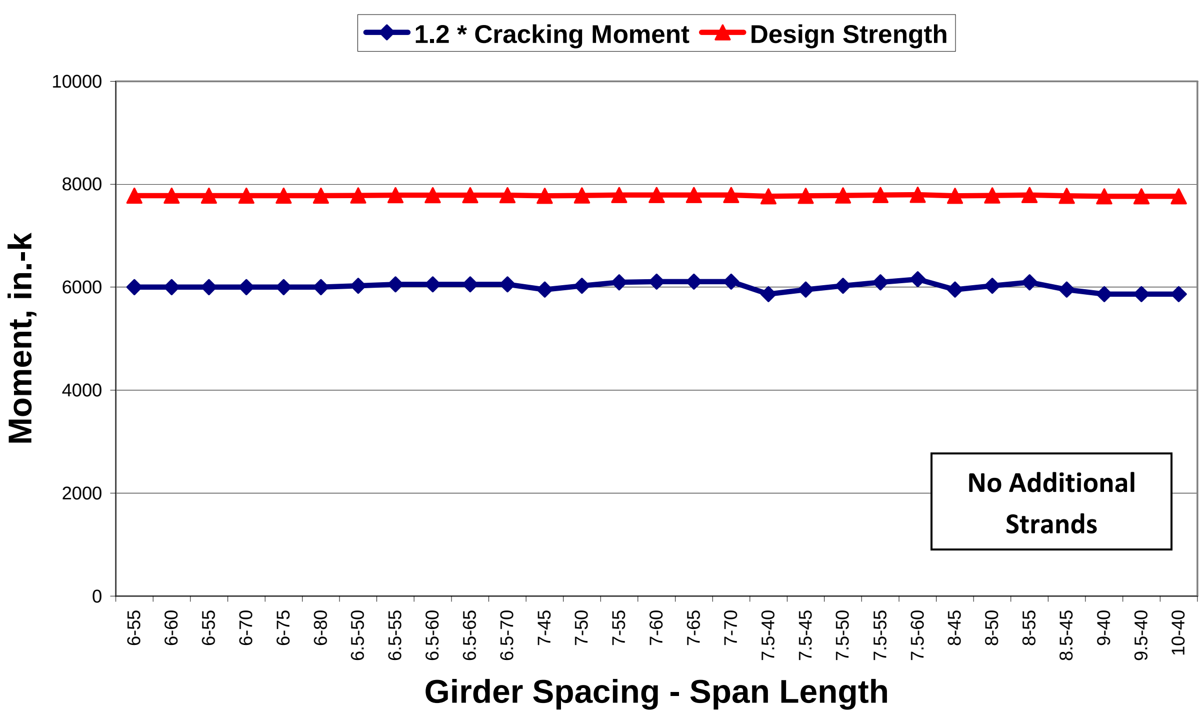
check = "OK"

APPENDIX B
STRANDS FOR PCBT BEAMS OLDER THAN 90 DAYS

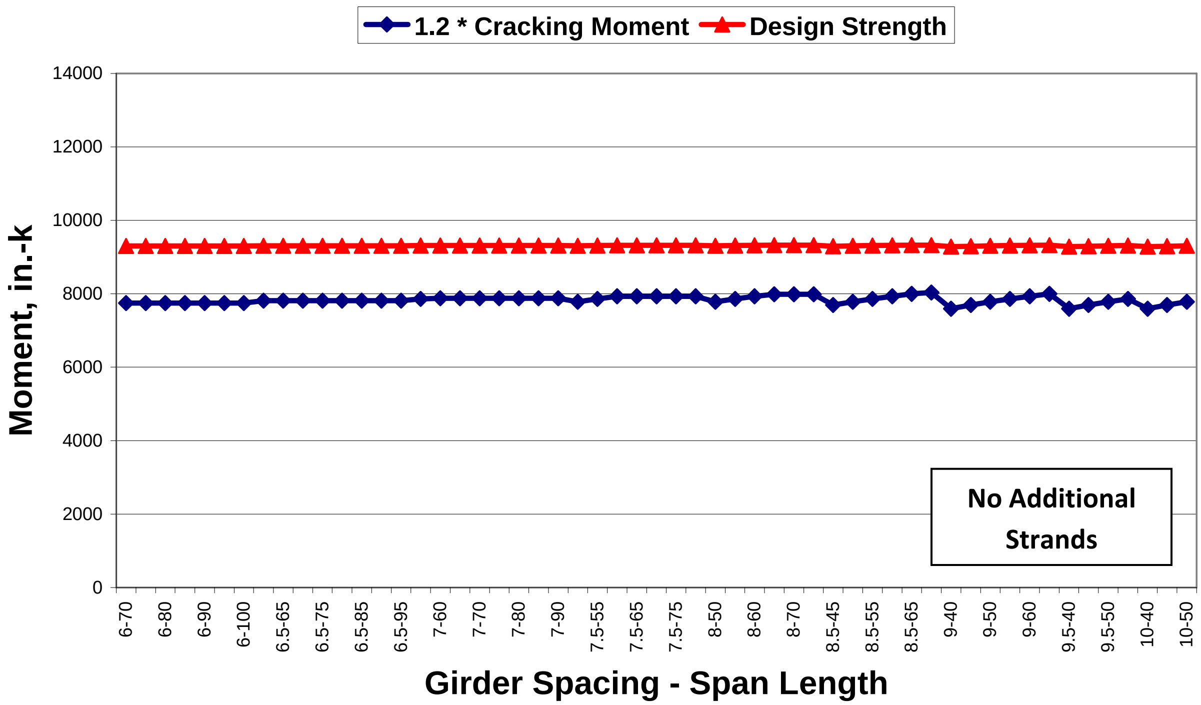
PCBT-29



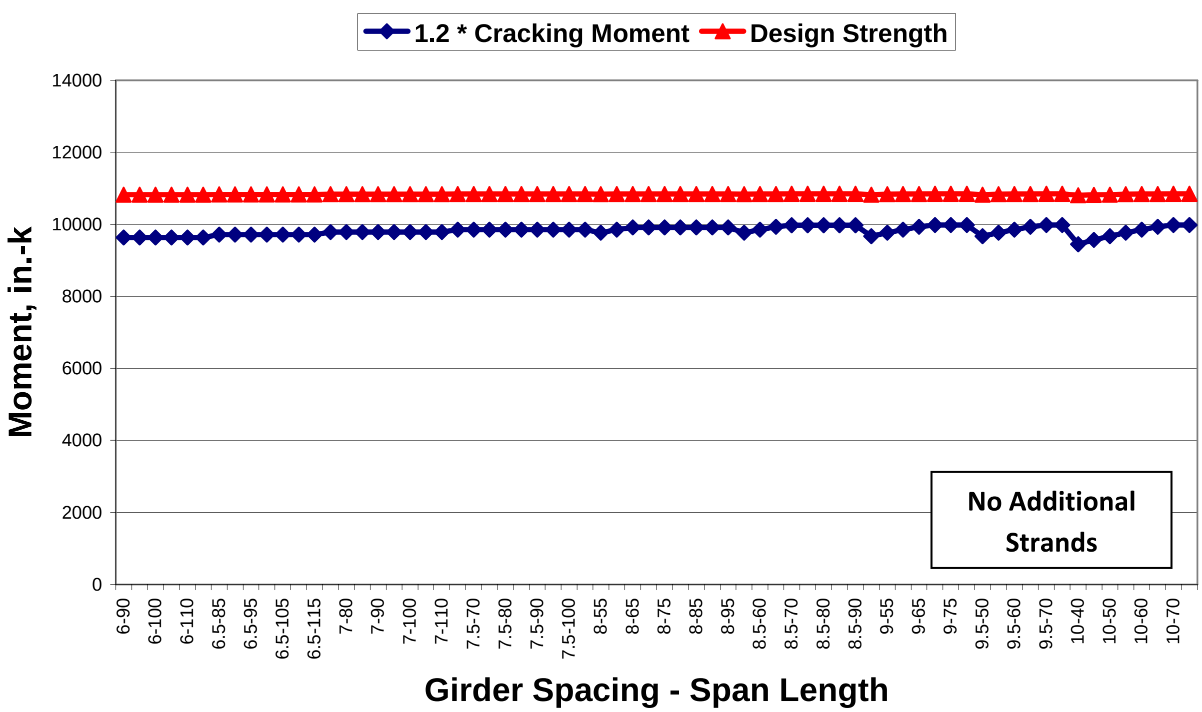
PCBT-37



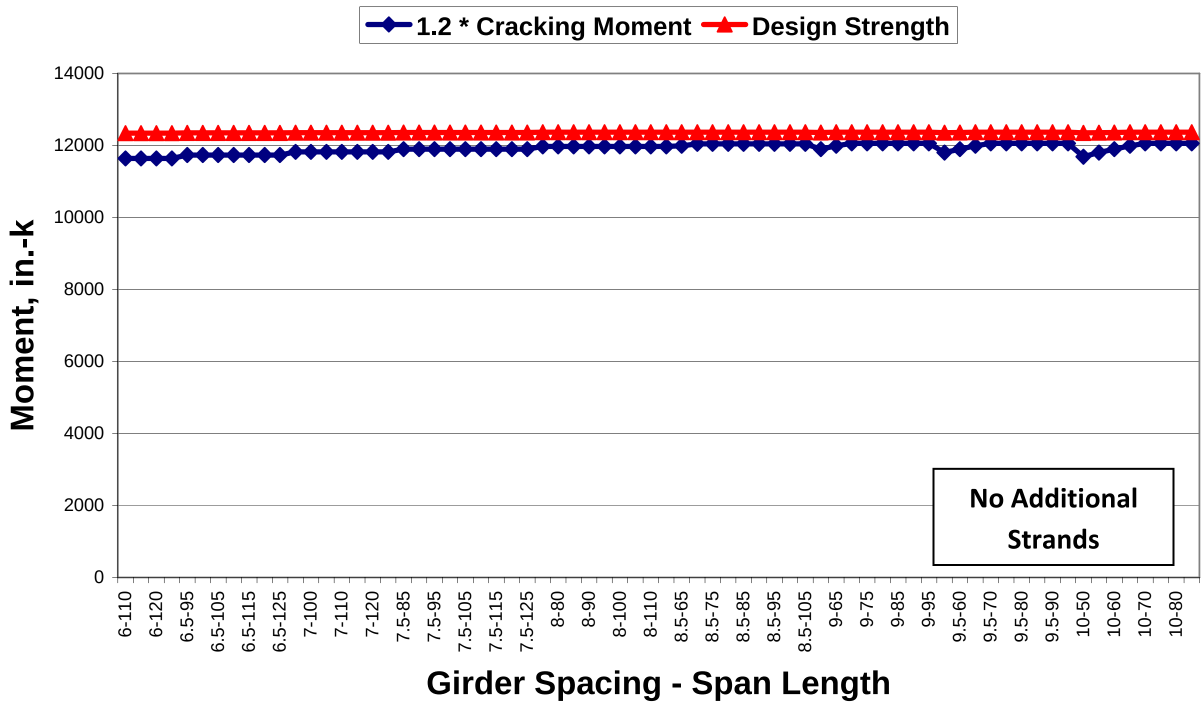
PCBT-45



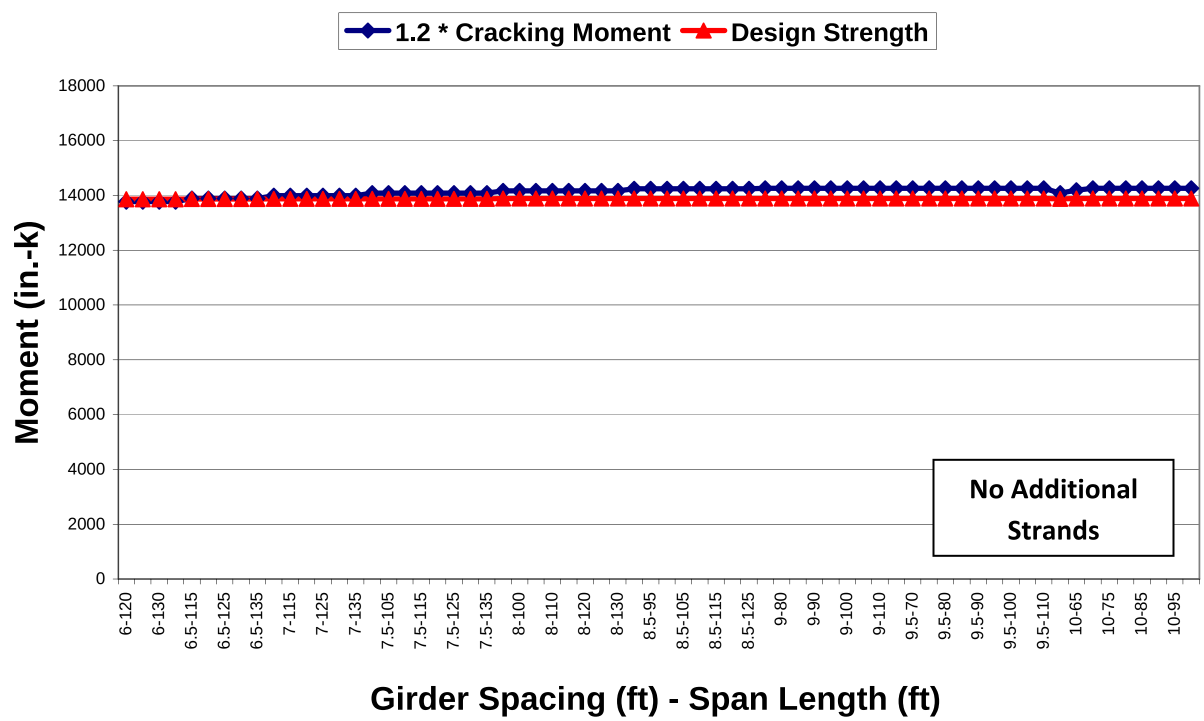
PCBT-53



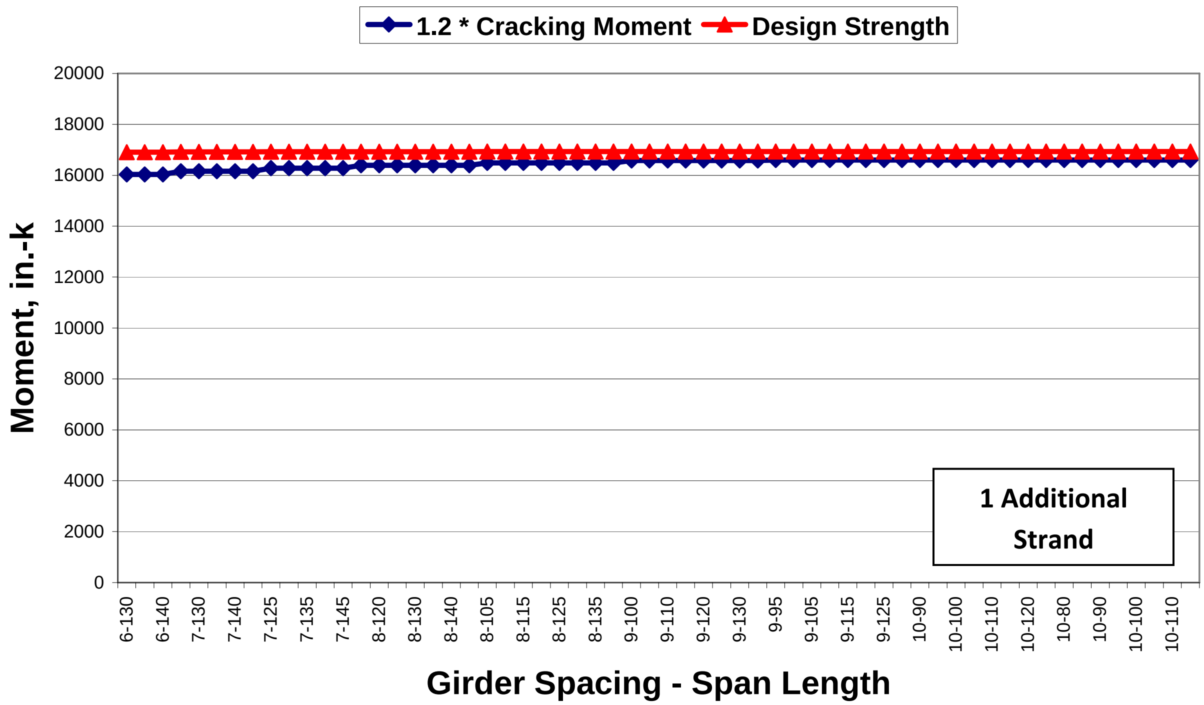
PCBT-61



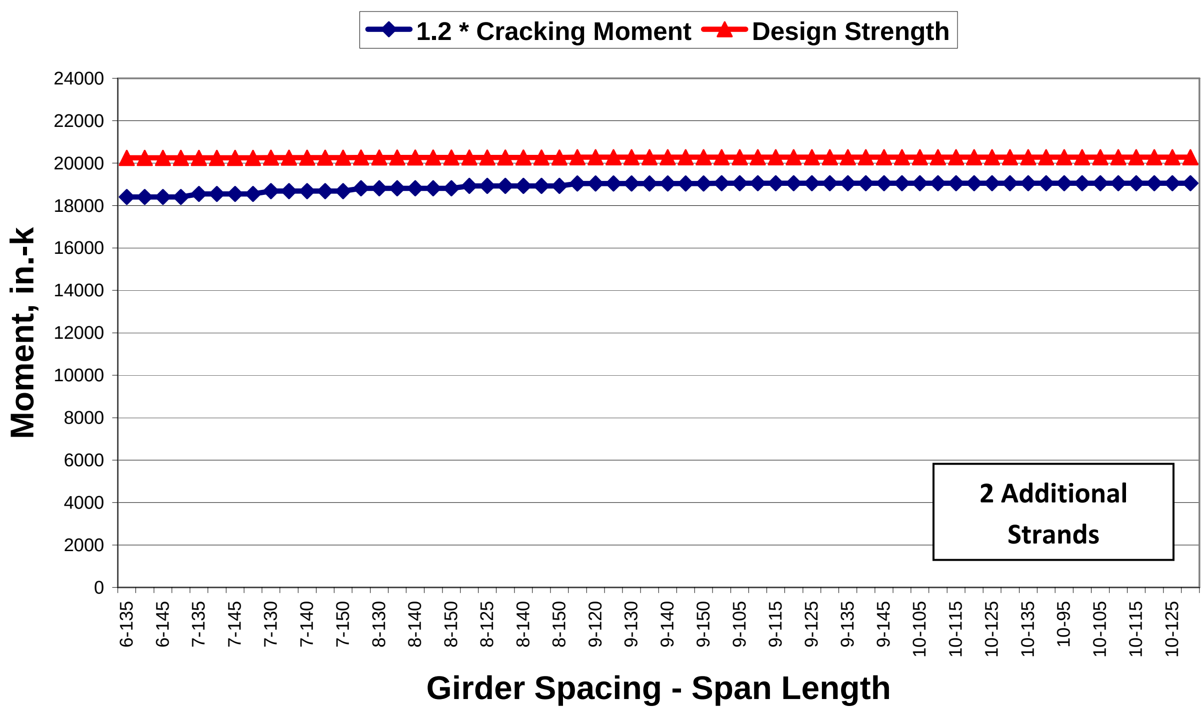
PCBT-69



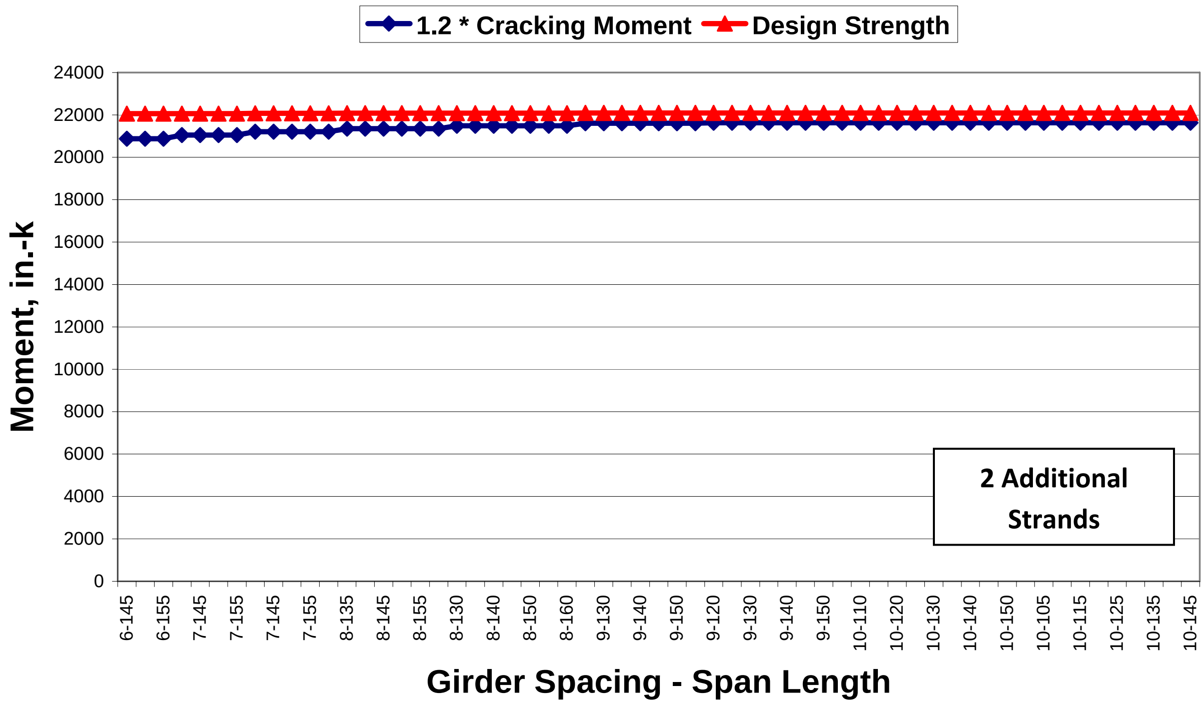
PCBT-77



PCBT-85

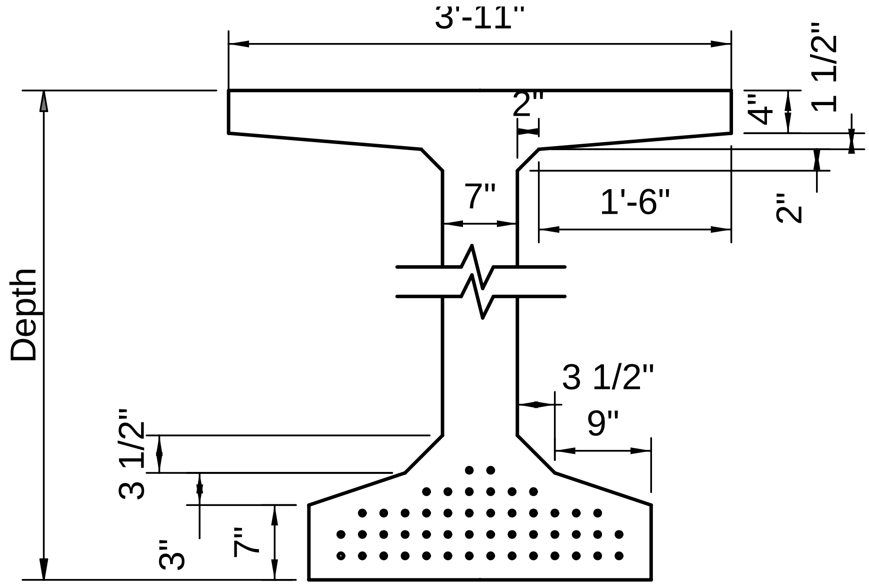


PCBT-93



1.2 * Cracking Moment
Design Strength
Moment, in.-k
24000
22000
20000
18000
16000
14000
12000
10000
8000
6000
4000
2000
0
6-145
6-155
7-145
7-155
7-145
7-155
8-135
8-145
8-155
8-130
8-140
8-150
8-160
9-130
9-140
9-150
9-120
9-130
9-140
9-150
10-110
10-120
10-130
10-140
10-150
10-105
10-115
10-125
10-135
10-145
Girder Spacing - Span Length
2 Additional Strands

APPENDIX C
PRESTRESSED CONCRETE BEAMS
STANDARD BULB-T DETAILS AND SECTION PROPERTIES



Beam Designation	Depth, in	Area, in^2	Centroid to Bottom, in	Moment of Inertia, in^4	Weight (@ 150 pcf), lb/ft
PCBT-29	29	643.7	14.66	66,800	661
PCBT-37	37	690.7	18.43	126,000	720
PCBT-45	45	746.7	2223	207,300	778
PCBT-53	53	802.7	26.06	312,400	836
PCBT-61	61	858.7	29.92	443,100	899
PCBT-69	69	914.7	33.79	601,300	953
PCBT-77	77	970.7	37.67	788,700	1013
PCBT-85	85	1026.7	41.57	1,007,200	1070
PCBT-93	93	1082.7	45.48	1,258,500	1128