RECOMMENDATIONS FOR LONGITUDINAL POST-TENSIONING IN FULL-DEPTH PRECAST CONCRETE BRIDGE DECK PANELS

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ABSTRACT

Full-depth precast concrete panels offer an efficient alternative to traditional cast-in-place concrete for replacement or new construction of bridge decks. Research has shown that longitudinal post-tensioning keeps the precast bridge deck in compression and prevents problems such as leaking, cracking, spalling, and subsequent rusting on the supporting beams at the transverse panel joints. The development of guidelines for levels of post-tensioning applicable to a variety of bridge types is necessary so designers may easily implement precast concrete panels in bridge deck construction or rehabilitation. A study was undertaken to determine the initial level of post-tensioning required in various precast concrete bridge deck panel systems in order to maintain compression in the transverse panel joints until the end of each bridge's service life. These recommendations were determined based on the results of parametric studies which investigated the behavior of bridges with precast concrete decks supported by both steel and prestressed concrete girders in single spans as well as two and three continuous spans. The age-adjusted effective modulus method was used to account for the ongoing effects of creep and shrinkage in concrete. This paper presents the resulting recommendations for initial levels of post-tensioning for various bridge systems based on the trends observed in the parametric studies.

Keywords: Precast Deck Panels, Post-tensioning, Prestress Loss

INTRODUCTION

Full-depth precast concrete panels can be used to rapidly replace deteriorated bridge decks, or construct new decks. There are many advantages to using full-depth precast deck panels, but to ensure their long term durability, the details must be carefully designed. This paper focuses on the design of the transverse panel-to-panel joint. A common joint type is the post-tensioned, grouted, narrow female-female keyed joint. Research has shown that longitudinal post-tensioning keeps the precast bridge deck in compression and prevents problems such as leaking, cracking, spalling, and subsequent rusting on the supporting beams at the transverse panel joints. Due to the creep and shrinkage of the deck panels, and the restraint of these strains by the supporting girders, the deck panels lose compression over time. The research presented in this paper was undertaken to determine the level of initial prestress required in bridge deck panels to ensure that over time, and subsequent loss of prestress, the joints remain in compression.

FULL-DEPTH DECK PANEL SYSTEMS

Figure 1 is a schematic of a bridge deck panel system. In the construction process, first, the bridge girders are erected on their supports. Next, the precast concrete panels are placed on top of the girders along the bridge, and leveling bolts are used to adjust the panels to their final elevations. After the panels are in place, the transverse panel joints are filled with grout, and the entire bridge deck is longitudinally post-tensioned to seal and compress the joints. If shear connectors are installed after the precast deck is in place, this step is completed next. Formwork to contain the haunch is assembled, and a non-shrink, high strength grout is used to fill the haunch and the open blockouts in the panels. Following the grouting process, waterproofing membranes and overlays may be added to the deck surface to enhance its appearance, rideability, and durability.

TIME DEPENDENT EFFECTS IN CONCRETE

In concrete structures, creep and shrinkage cause strains to gradually develop. Usually the concrete contains prestressed and/or non-prestressed steel, so the development of strains in the cross section over time causes stresses to be induced in every element of the cross section, including the concrete itself as well as any steel that is present (Dilger, 1982). In many cases, the stresses and deformations resulting from this continuous redistribution of forces can influence the structure as much as the dead and live loads which are applied to it. Therefore, it is very important to consider the long-term effects of creep and shrinkage in concrete, as well as relaxation of prestressing steel, in the design and analysis of concrete elements. In this research, the effects which these ongoing changes have on the post-tensioning and the corresponding level of compression it applies to the precast concrete bridge deck panels are of particular interest and importance.

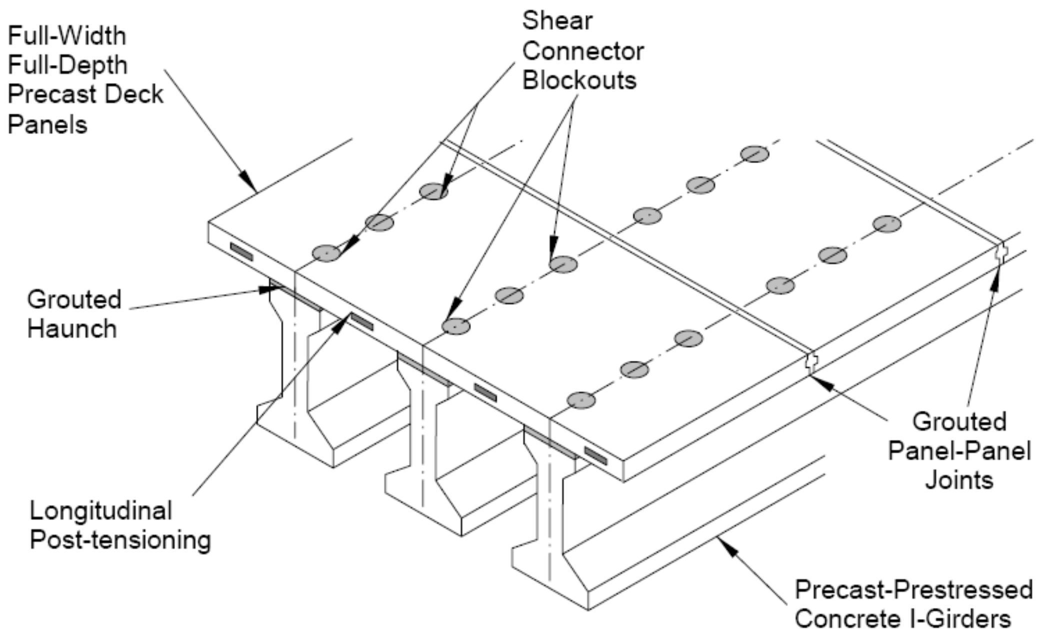

Figure 1. Precast Concrete Bridge Deck Panel System

RESEARCH OBJECTIVES AND SCOPE

The primary objective of this research is to recommend the initial level of post-tensioning required in various precast concrete bridge deck panel systems in order to maintain compression in the transverse panel joints until the end of each bridge's service life. The predictions of stress changes over time are made by performing time-dependent analyses which consider the redistribution of forces in bridge systems caused by creep and shrinkage in concrete and relaxation of prestressing steel. The age-adjusted effective modulus method is used to account for the ongoing effects of creep and shrinkage in concrete.

To achieve this goal, parametric studies were conducted on a variety of bridge configurations including both steel and prestressed concrete girders in simple span as well as two and three span continuous bridges.

BACKGROUND ON PRECAST CONCRETE BRIDGE DECK PANEL SYSTEMS

Issa has presented several papers related to performance of bridge deck panels (1995a, 1995b, 1998, 2000). The comprehensive research program included field surveys of existing bridges, finite element analysis and full-scale testing. Based on his field studies, Issa concluded that proper performance of the transverse joints was critical to the overall durability of the system (Issa, 1995b). The cases where inadequate performance of the system was observed were attributed to several possible factors, including the absence of longitudinal post-tensioning, the horizontal shear connection type, the panel-to-panel joint configuration, and the construction methods and materials used. As a result, one of the researchers' major recommendations was to longitudinally post-tension precast concrete bridge deck panels "to secure the tightness of the joints, to keep the joint in compression, and to guard against leakage" (Issa, 1995a).

Following their literature review, questionnaire surveys, and field investigations on precast concrete panels used in bridge deck construction and rehabilitation (1995), Issa et al. presented a third study in 1998 regarding the finite element modeling and analysis of such a system. Issa et al. suggested a minimum initial deck post-tensioning level of 200 psi for a simple-span bridge which was modeled. This level of prestress should keep the transverse joints in compression and also account for the time-dependent effects of creep and shrinkage in the concrete (1998). Based on a continuous bridge structure, Issa et al. recommended a minimum initial deck post-tensioning level of 450 psi to ensure compression in the deck at the critical interior support locations of continuous systems (1998).

In 2000, Issa, et al. performed an experimental study to investigate the behavior of full-depth precast concrete panels used in bridge decks. One of their main goals was to examine the amount of longitudinal post-tensioning needed to keep the transverse panel joints in compression. To accomplish this, they tested three different two-span continuous bridge models with precast concrete deck panels supported by steel girders. The first bridge model contained a bridge deck with no longitudinal post-tensioning, whereas the second and third models incorporated initial post-tensioning levels of 208 and 380 psi, respectively, into their deck systems. The most important overall conclusion from the experimental research was that "the longitudinal post-tensioning was effective in delaying crack initiation" (Issa et al., 2000).

METHODS AND MODELING

This research was initiated to determine if the previously recommended levels of initial prestress, which were based on a limited number of bridge configurations, were adequate for a wider variety of systems. To accomplish this, numerous models of different bridge systems were developed using the software Mathcad. Each model bridge cross-section consisted of either steel or prestressed concrete girders, a 1 in. haunch, and a deck made out of full-depth precast concrete panels. The level of post-tensioning applied to the precast concrete bridge deck in each model was varied until the transverse panel joints were observed to be in compression at the assumed end of each bridge's service life. This section describes the procedures used to develop the models in Mathcad, the determination of the girder types and other bridge details used for the parametric studies, and the implementation of the models in the parametric studies themselves.

MATERIAL PROPERTIES

Steel and Prestressed Concrete Girders

The steel girders were either rolled shapes or plate girders, with a modulus of elasticity of 29,000 ksi. The cross-sections with prestressed concrete girders included either Virginia PCBT girders or AASHTO standard girders, each with a 28 day compressive strength of 7000 psi and an aging coefficient of 0.7. The prestressing strands were all ½ in. diameter, Grade 270 low relaxation strands, with a modulus of elasticity of 28,500 ksi. Creep and shrinkage properties were calculated using AASHTO LRFD (2007) models.

Precast Concrete Panels and Haunch

The precast concrete panels making up the bridge deck in each model had a 28 day compressive strength of 5000 psi. The steel girder bridges had 8.5 in. thick precast decks, while the prestressed concrete girder bridges had 8 in. thick precast decks. Each bridge model also contained a 1 in. thick haunch separating the top of each girder from the bottom of the precast deck. The 28 day compressive strength of the haunch was assumed to be equal to that of the precast deck. The deck and the haunch were both assigned an aging coefficient of 0.7, and the deck post-tensioning strands were all ½ in. diameter, Grade 270 low relaxation strands. Creep and shrinkage properties were calculated using AASHTO LRFD (2007) models.

It should be noted that creep and shrinkage are highly variable and difficult to predict. This research used a recently developed model for creep and shrinkage, which was shown to be relatively accurate in the prediction of prestress losses in pretensioned girders (Tadros et al. 2003). The panels were assumed to be relatively low strength and young in age, which will result in high creep and shrinkage. This in turn should result in conservative predictions of loss of compression in the deck over time.

MODEL DEVELOPMENT

The primary steps in the development of the Mathcad models included denoting the time intervals to be analyzed for each type of bridge, and determining the equations to calculate the redistribution of stresses due to long-term creep, shrinkage, and steel relaxation corresponding to each of these time intervals. For the multiple span bridges, it was also necessary to consider the effects of continuity and live loads, particularly at the interior supports. The procedures used to develop each type of bridge model are discussed in this section.

Construction Time Intervals

The time-dependent analyses performed in each Mathcad model were separated into the major time intervals existing throughout the construction and service life of a bridge with a deck composed of precast concrete panels. For the bridges with precast concrete deck panels supported by steel girders, the two time intervals containing stress redistributions were denoted as:

1. D/SG 1 – Time of post-tensioning the deck to the start of composite action between the deck and girders
2. D/SG 2 – Start of composite action between deck and girders to the end of the bridge's service life, which was estimated as 10,000 days.

While these two phases also applied to the bridges with precast concrete deck panels supported by prestressed concrete girders, an additional phase was necessary to account for the time-dependent effects occurring in the prestressed concrete girder. The three time intervals for the precast deck panel/prestressed concrete girder system were denoted as:

1. D/CG 1 – Time of transfer of prestress to the concrete girder to the start of composite action between the girders and deck
2. D/CG 2 – Time of post-tensioning the deck to the start of composite action between the deck and girders
3. D/CG 3 – Start of composite action between deck and girders to the end of the bridge's service life, which was estimated as 10,000 days.

Table 1 indicates the construction time intervals on which the time-dependent analyses performed in the various bridge models were based. For the prestressed concrete girder bridges, it was assumed that the concrete girders and deck panels were both cast at the same time, so the three concrete girder bridge intervals are relative to this particular time of girder and panel casting. Composite action was assumed to occur instantaneously at 60 days in both the steel and prestressed concrete girder bridges.

Table 1. Construction Time Intervals for Bridge Models

Time Interval	Start Time (days)	End Time (days)
D/SG 1	55	60
D/SG 2	60	10000
D/CG 1	1	60
D/CG 2	55	60
D/CG 3	60	10000

Equations for Time-Dependent Analysis

Once the appropriate time intervals were established, it was necessary to write systems of equations to model the behavior and solve for the changes occurring in a given bridge in each of the time intervals listed above. Whereas long-term prestress losses only had to be considered in the decks of the bridges with steel girders, the bridges containing prestressed concrete girders presented a more complicated situation, with time-dependent effects occurring in both the concrete girders and the concrete deck. An age adjusted effective modulus formulation was used to write the constitutive equations for the concrete deck, haunch and concrete girders. Tensile stresses and lengthening strains were defined as positive, while compressive stresses and shortening strains were considered negative. In addition, compression or shortening at the top of a member indicated positive moment and positive curvature. The systems of equations used to solve for the changes occurring in each bridge system over time are presented in this section. Since many of the variables used in each type of model appear multiple times in different equations, all quantities are defined in the **NOTATION** section at the end of this paper.

Bridges with Steel Girders

The first time interval for the steel girder bridges, D/SG 1, includes the changes in forces and strains occurring from the time that the deck is post-tensioned to the start of composite action between the concrete deck and the steel girders. During this time, creep, shrinkage, and steel relaxation simultaneously cause the force in the post-tensioning to become less tensile, and the corresponding force in the deck concrete to become less compressive. In addition, compressive shrinkage strains occur in the deck concrete, which results in shortening of the post-tensioning steel as well. These changes are modeled by the following four equations:

<u>Equilibrium</u>

$$\Delta N_d + \Delta N_{ptd} = 0 \tag{1}$$

<u>Compatibility</u>

$$\Delta \varepsilon_d = \Delta \varepsilon_{ptd} \tag{2}$$

<u>Constitutive</u>

$$\Delta \varepsilon_d = \frac{N_{do}}{A_d E_d}\phi_d + \frac{\Delta N_d}{A_d E_d}(1 + \mu_d \phi_d) + \varepsilon_{shd} \tag{3}$$

$$\Delta \varepsilon_{ptd} = \frac{\Delta N_{ptd} - \Delta f_{pR} A_{ptd}}{A_{ptd} E_{ptd}} \tag{4}$$

where all variables are defined in the notation section.

Equation 1 defines the equilibrium requirement that the change in the compressive axial force in the deck concrete must be equal and opposite to the corresponding change in the tensile axial force in the post-tensioning strands. Equation 2 establishes compatibility between the changes in strain in the deck concrete and the post-tensioning steel. Equations 3 and 4 are the constitutive relationships between the changes in strains and forces in the deck concrete and the post-tensioning tendons. The three terms in equation 3 represent the creep associated with the initial strain in the deck, the elastic strain and creep strain components due to the change in force in the deck during D/SG 1, and the shrinkage strain in the deck concrete during D/SG 1. The two quantities in the numerator of equation 4 represent the total change in axial force in the deck post-tensioning strands, and the change in force due to relaxation of the post-tensioning steel which is subtracted out since it has no corresponding change in strain. The quantities ϕ_d, ε_{shd}, and Δf_{pR} in equations 3 and 4 represent the deck creep coefficient, deck shrinkage strain, and post-tensioning strand relaxation corresponding to the D/SG 1 time interval only.

The second time interval for the steel girder bridges, D/SG 2, includes the changes in forces, moments, strains, and curvature from the start of composite action between deck and girders to the end of the bridge's service life, which was estimated as 10,000 days. In the composite cross-sections of the steel girder bridges, the concrete deck and haunch undergo creep and shrinkage while the steel girder resists these forces. The corresponding changes in forces, moments, strains and curvature for the D/SG 2 time interval in the steel girder bridges are modeled by the following equations:

Equilibrium

$$\Delta N_d + \Delta N_h + \Delta N_g + \Delta N_{ptd} = 0 \tag{5}$$

$$\Delta M_d + \Delta M_h + \Delta M_g + \Delta N_h * a + \Delta N_g * b = 0 \tag{6}$$

Compatibility

$$\Delta \varepsilon_d = \Delta \varepsilon_{ptd} \tag{7}$$

$$\Delta \varepsilon_d = \Delta \varepsilon_h - \Delta \chi * a \tag{8}$$

$$\Delta \varepsilon_d = \Delta \varepsilon_g - \Delta \chi * b \tag{9}$$

Constitutive

$$\Delta \varepsilon_d = \frac{N_{doc}}{A_d E_d}\phi_d + \frac{\Delta N_d}{A_d E_d}(1 + \mu_d \phi_d) + \varepsilon_{shd} \tag{10}$$

$$\Delta \varepsilon_g = \frac{\Delta N_g}{A_g E_g} \tag{11}$$

$$\Delta \varepsilon_h = \frac{\Delta N_h}{A_h E_h}(1 + \mu_h \phi_h) + \varepsilon_{shh} \tag{12}$$

$$\Delta \varepsilon_{ptd} = \frac{\Delta N_{ptd} - \Delta f_{pR} A_{ptd}}{A_{ptd} E_{ptd}} \tag{13}$$

$$\Delta \chi = \frac{\Delta M_d}{I_d E_d}(1 + \mu_d \phi_d) \tag{14}$$

$$\Delta \chi = \frac{\Delta M_h}{I_h E_h}(1 + \mu_h \phi_h) \tag{15}$$

$$\Delta \chi = \frac{\Delta M_g}{I_g E_g} \tag{16}$$

where all variables are defined in the notation section.

Equations 5 and 6 define the equilibrium requirements for the changes in forces and moments in the composite system. Equation 7 was discussed previously, and equations 8 and 9 establish additional strain compatibility relationships based on the assumption that plane sections remain plane throughout the composite cross section. The terms in equations 10 and 13 are similar to those discussed for the D/SG 1 time interval, and the quantity N_{doc} in equation 10 is the force in the deck at the beginning of the composite phase. Equation 11 represents the change in strain in the steel girder which undergoes no creep or shrinkage. Equations 14 and 15 each describe the change in curvature based on the elastic and creep-producing changes in moment in the concrete deck and haunch, respectively. Equation 16 also represents the change in curvature, but in terms of the elastic change in moment in the steel girder. The quantities ϕ_d, ϕ_h, ε_{shd}, ε_{shh}, and Δf_{pR} in the above equations represent the creep coefficients, shrinkage strains, and steel strand relaxation corresponding to the D/SG 2 time interval only. Figure 2 illustrates the initial force present and the changes occurring throughout the D/SG 2 phase defined by equations 5 through 16. While all of the varying quantities are represented as positive in the figure, the appropriate sign conventions were accounted for in the corresponding equations.

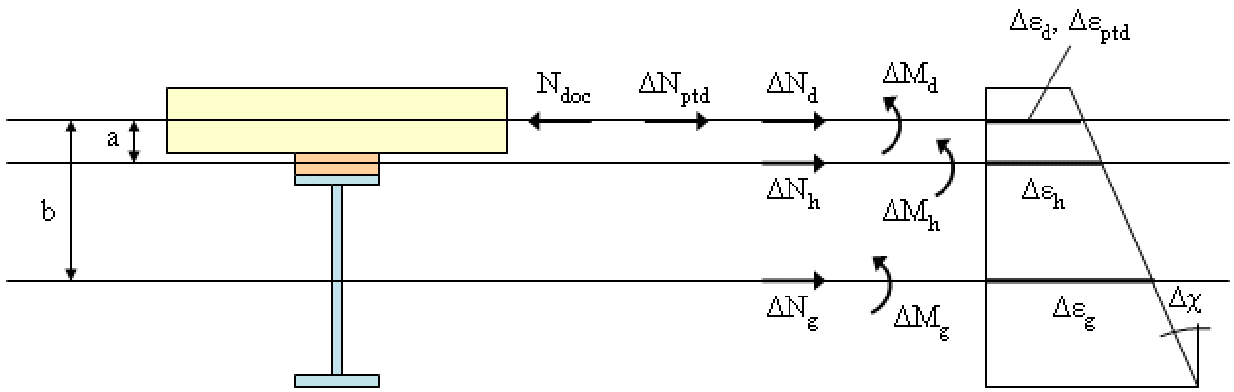

Figure 2. Initial Force and Changes Occurring in the D/SG 2 Phase

Bridges with Prestressed Concrete Girders

The first phase listed above for the prestressed concrete girder bridges, D/CG 1, includes the changes in forces, moments, strains, and curvature occurring from the time that prestress is transferred to the girder to the time that the girder becomes composite with the deck. Since steel girders are not affected by long-term prestress losses, this phase was not needed in the analysis of the steel girder bridges. The changes occurring in the prestressed concrete girder from transfer of prestress to composite action with the deck are modeled by the following equations:

Equilibrium

$$\Delta N_g + \Delta N_{psg} = 0 \tag{17}$$

$$\Delta M_g + \Delta N_{psg} * e_g = 0 \tag{18}$$

Compatibility

$$\Delta\varepsilon_g = \Delta\varepsilon_{psg} - \Delta\chi * e_g \tag{19}$$

Constitutive

$$\Delta\varepsilon_g = \frac{N_{go}}{A_{gn}E_g}\phi_g + \frac{\Delta N_g}{A_{gn}E_g}(1+\mu_g\phi_g) + \varepsilon_{shg} \tag{20}$$

$$\Delta\chi = \frac{M_{go}}{E_g I_{gn}}\phi_g + \frac{\Delta M_g}{E_g I_{gn}}(1+\mu_g\phi_g) \tag{21}$$

$$\Delta\varepsilon_{psg} = \frac{\Delta N_{psg} - \Delta f_{pR} A_{psg}}{A_{psg}E_{psg}} \tag{22}$$

where all variables are defined in the notation section.

While the format and purpose of equations 17 through 22 are similar to those explained for the steel girder time intervals above, these equations now account for the prestressing force and time-dependent effects occurring in the concrete girder. The quantities N_{go} and M_{go} indicate the initial force and moment due to the prestress and self weight in the girder, and the variables ϕ_g, ε_{shg}, and Δf_{pR} represent the girder creep coefficient, girder shrinkage strain, and prestressing strand relaxation corresponding to the D/CG 1 time interval only. Figure 3 illustrates the initial force and moment as well as the changes occurring in the D/CG 1 phase defined by equations 17 through 22. While all of the varying quantities are represented as positive in the figure, the appropriate sign conventions were accounted for in the corresponding equations.

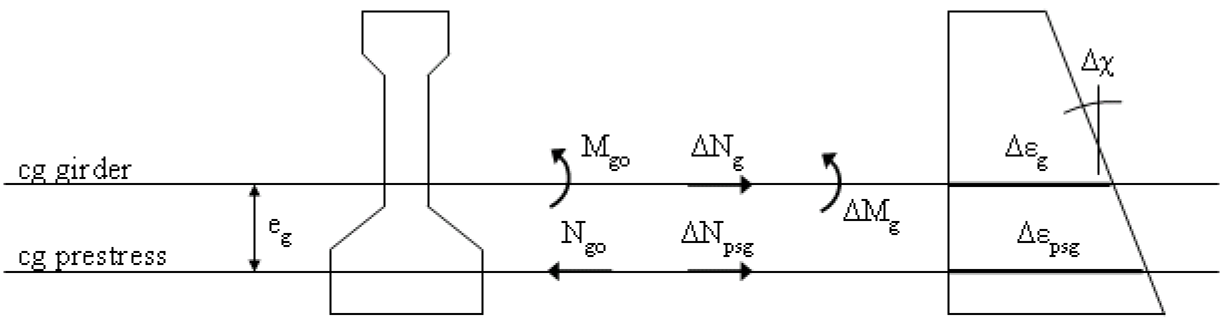

Figure 3. Initial Effects and Changes Occurring in the D/CG 1 Phase

The second phase listed above for the prestressed concrete girder bridges, D/CG 2, is identical to the first time interval for the steel girder bridges (D/SG 1). This is true because only the concrete deck panels are affected by the changes occurring during this time interval, which spans from the time of deck post-tensioning to the beginning of composite action between the deck and girders. Therefore, the same equations (1 to 4) presented for calculating changes during the D/SG 1 phase apply for calculating changes during the D/CG 2 phase.

The third time interval for the prestressed concrete girder bridge models, D/CG 3, is similar to the second time interval for the steel girder bridges (D/SG 2). For both types of girders, this phase begins with the start of composite action between the deck and girders and concludes at the end of the bridge's service life. In the composite cross-sections of the prestressed concrete girder bridges, however, the concrete deck, haunch, and girder each experience the effects of creep and shrinkage at different rates, making the time-dependent redistribution of forces and moments much more complex than in the steel girder bridges. The equations to calculate changes in forces, moments, strains and curvature for the D/CG 3 time interval in the prestressed concrete girder bridges can be found in Bowers (2007).

SIMPLE SPAN MODELS

Mathcad models were developed to determine changes in stresses and strains in the composite system based on the previously described methods. The Mathcad sheets and all variables are presented in Bowers (2007). It is important to note that any section properties or stresses calculated in the following procedures are located at midspan. In the simple span prestressed concrete girder models, midspan was the critical location for potential tensile stresses in the deck if upward camber of the girder dominated the curvature of the span. In the simple span steel girder models, midspan was also used as the location for calculating the stresses throughout the deck, which are constant along a span under uniform curvature.

Bridges with Steel Girders

The basic steps to formulate the simple span steel girder bridge models included:

1. Define and/or calculate all material properties, section properties, and time intervals.
2. Program Mathcad routines to calculate creep coefficients and shrinkage values based on the AASHTO 2007 Specification equations.
3. Perform calculations for phase D/SG 1, deck post-tensioning to composite action:
    a. Compute the average stress in the post-tensioning tendons immediately after jacking, considering instantaneous losses due to anchor seating.
    b. Use the AASHTO equation to find the relaxation in the tendons over the time interval.
    c. Apply the creep and shrinkage routines programmed in step 2 to calculate the creep coefficient and shrinkage strain in the concrete deck during the time interval.
    d. Insert equations 1 to 4 into matrices and use matrix algebra to solve for the unknown changes in forces and strains.
4. Perform calculations for phase D/SG 2, composite action to end of bridge service life:
    a. Apply the creep and shrinkage routines programmed in step 2 to calculate the creep coefficient and shrinkage strain in the concrete deck during the time interval.
    b. Update the initial axial force N_{do} in the deck to account for the change in force in the deck from D/SG 1, and use the new quantity N_{doc} for the calculations in the interval D/SG 2.
    c. Apply the AASHTO equation to find the relaxation in the post-tensioning strands over the time interval.
    d. Insert equations 5 to 16 into matrices and use matrix algebra to solve for the unknown changes in forces, moments, strains, and curvature.
    e. Calculate and plot the final stresses throughout the composite cross section.

Bridges with Prestressed Concrete Girders

The basic steps to formulate the simple span prestressed concrete girder bridge models were very similar to those used for the steel girder bridge models, except for the addition of the D/CG 1 time interval to account for the concrete in the girder. The approach for formulating these models is described Bowers (2007).

CONTINUOUS SPAN MODELS

All of the two and three span continuous bridge models begin with the simple span procedures described above. After this process is used to find the stresses in the concrete deck in a simple span case, each Mathcad model continues with additional calculations to account for the time-dependent effects in either two or three continuous spans. It is important to note that the most critical location in the continuous models was assumed to be at the interior support(s), where the highest values of tension in the concrete should occur at the top of the deck due to negative bending caused by live loads and stress redistributions.

The first new step introduced in the two and three-span continuous bridge models involved using the force method to calculate the stresses induced by the time-dependent effects and continuity at the interior support(s). This procedure was very similar for the two and three-span continuous models. The procedure required removal of the interior support and calculation of the resulting downward displacement due to the change in curvature calculated in the time dependent analysis ($\Delta\chi$) which is uniform along the length for steel girders. Then the force required to return the support to zero displacement is calculated. Because this is a force which develops slowly over time, the age-adjusted transformed moment of inertia of the composite cross section is used for the calculation. With this force, the moment and stresses at the interior support can be calculated.

After finding the critical deck stress due to time-dependent effects, the next new requirement for a continuous system was to account for the component of stress in the deck due to live loads. The live loads on each bridge created negative moments and subsequent tensile stresses at the interior support(s). These negative moments were found using QConBridge, a software package created by the Washington State Department of Transportation (Brice, 2005).

Bridges with Steel Girders

After the simple span analysis, the additional steps necessary to analyze two or three continuous spans with steel girders follow. Refer to Bowers (2007) for an example continuous steel girder bridge model in Mathcad.

1. Calculate the regular and age-adjusted transformed section properties for the composite section including the haunch.
2. Calculate the stresses induced by continuity at the interior support(s) using the force method.

3. Use QConBridge to determine the negative moment at the interior support(s) due to live loads on the bridge, and calculate the corresponding tensile stress.
4. Multiply the stress due to live loads by the appropriate distribution factor. The stress due to live loads was also multiplied by a factor of 0.8, which is for the Service III "load combination relating only to tension in prestressed concrete superstructures with the objective of crack control" (AASHTO, 2004). Although the original intention of this factor was for controlling cracking in the tensile region at the bottom of prestressed concrete girders in positive bending, for this research it was similarly assumed to apply to tension at the top of the concrete in a negative moment region of a bridge deck.
5. Find the final stress in the deck by summing three quantities: the stress at the top of the deck after the simple span analysis, the stress generated at the interior support(s) due to continuity and time-dependent effects, and the factored stress due to live loads.

Bridges with Prestressed Concrete Girders

The procedure for analysis of the continuous spans with prestressed concrete girders was significantly more complicated than that for the steel girders. Unlike the uniform change in curvature assumed to exist along the full length of a composite span with steel beams, a span with composite concrete girders does not exhibit a constant change in curvature along its length because of the varying centroid of prestress in the girders and the time-dependent effects involved in the system. In this case, the time-dependent behavior is complicated by the typically unequal ages of the girder and deck concretes as well as the effects of continuity. Therefore, a sectional analysis was performed for these models, and the change in curvature during the composite time interval (D/CG 3) was calculated at several locations along each span. These locations included the ends, the ¼ and ¾ points and midspan in each span. The change in curvature at each location during the interval D/CG 3 was then used to calculate the component of stress at the continuous supports due to continuity using the force method described above. Refer to Bowers for additional details regarding the calculations indicated, as well as two examples of continuous prestressed concrete girder bridge models in Mathcad.

PARAMETRIC STUDIES

After developing each type of bridge model in Mathcad, these models were employed to investigate the response of different bridge layouts to various amounts of post-tensioning in their precast concrete decks. The primary goal was to look for trends in the behavior of similar bridges so that simple design recommendations regarding levels of post-tensioning for bridge decks could be made.

Selection of Steel Girders

Span lengths of 60 ft, 90 ft, and 120 ft with girder spacings of 6 ft and 9 ft were selected for evaluation in the steel girder bridge parametric studies. Tables 2 and 3 provide the details of the different steel girders used in the simple and continuous span parametric studies.

Selection of Prestressed Concrete Girders

Due to the larger availability of PCBT and AASHTO girder design aids and the additional complexity inherent in the time-dependent analysis of bridges with prestressed concrete girders, a greater number of cross sections with concrete girders was analyzed in the parametric studies. Three different sizes of each type of concrete girder were selected, and a ‘short’ and a ‘long’ span length for both 6 and 9 ft girder spacings were designed for each type of girder. An attempt was made to maintain consistent span length to girder depth ratios for each set of similar span lengths and girder spacings for each girder type.

Table 2. Steel Girders used in Parametric Studies

Girder Spacing (ft)	Span Length (ft)	Steel Girder Depth (in)	W or Plate Girder Section
6	60	24	W24x103
6	90	36	W36x160
6	120	48	PL 1, d=48
9	60	24	W24x146
9	90	36	W36x232
9	120	48	PL 2, d=50

Table 3. Plate Girder Dimensions

Plate Girder	Total Depth (in)	t_f (in)	b_f (in)	t_w (in)	d_w (in)
PL 1	48	1.125	14	0.75	45.75
PL 2	50	1.375	16	0.875	47.25

The bridges with PCBT, or Prestressed Concrete Bulb-T, girders were designed using the Virginia standard bulb-T details and preliminary design tables. The PCBT-37, PCBT-61, and PCBT-85 girders (with respective depths of 37, 61, and 85 in.) were chosen, and the required number of prestressing strands for each combination of span length and girder spacing was determined using the preliminary design tables. The bridges with AASHTO girders were designed using the AASHTO I-Beam details and design charts provided in the PCI Bridge Design Manual (2005). The AASHTO Type II, Type IV, and Type VI girders with respective depths of 36, 54, and 72 in. were selected, and the required number of prestressing strands for each combination of span length and girder spacing was determined using the preliminary design charts. Table 4 shows the details of the different prestressed concrete girders used in the simple and continuous span parametric studies.

Method for Conducting Parametric Studies

After all of the models were created and the steel and prestressed concrete girder bridges were designed, the amount of initial compression in the deck of each bridge model was varied by changing the number of post-tensioning strands. This process was started at an initial compression stress of about 100 or 200 psi in the deck, and as the stress was increased by increments of either 100 or 200 psi, the resulting stress in the deck panel joints at the end of each time-dependent analysis was recorded. The number of post-tensioning strands in the deck was increased until the results showed that the deck panel joints remained in compression at the end of the bridge service life.

Table 4. Prestressed Concrete Girders used in Parametric Studies

Girder Type	Girder Spacing (ft)	Span Length (ft)	L/d	No. of ½ in. Dia. Strands
PCBT-37	6	40 75	13.0 24.3	14 28
	9	40 -	13.0 -	14 -
PCBT-61	6	65 125	12.8 24.6	16 50
	9	50 85	9.8 16.7	18 28
PCBT-85	6	85 150	12.0 21.2	20 50
	9	70 125	9.9 17.6	22 44
AASHTO Type II (d = 36 in.)	6	45 70	15.0 23.3	8 28
	9	35 55	11.7 18.3	8 24
AASHTO Type IV (d = 54 in.)	6	75 120	16.7 26.7	16 54
	9	65 100	14.4 22.2	18 50
AASHTO Type VI (d = 72 in.)	6	100 160	16.7 26.7	22 76
	9	100 140	16.7 23.3	30 76

RESULTS AND ANALYSIS

SIMPLE SPAN MODELS

Bridges with Steel Girders

The first set of parametric studies was performed for simple span bridges with steel girders and a precast concrete deck. The precast concrete decks in each of the six different steel girder bridges were post-tensioned to stresses ranging from 100 to 400 psi in increments of approximately 100 psi. Figure 4 illustrates typical distributions of stress and strain obtained throughout the steel girder bridge cross sections at the end of service. The values shown in Figure 4 correspond with the results for the simple span W24x103 model initially post-tensioned to -200 psi, which is also provided as an example Mathcad model in Bowers (2007).

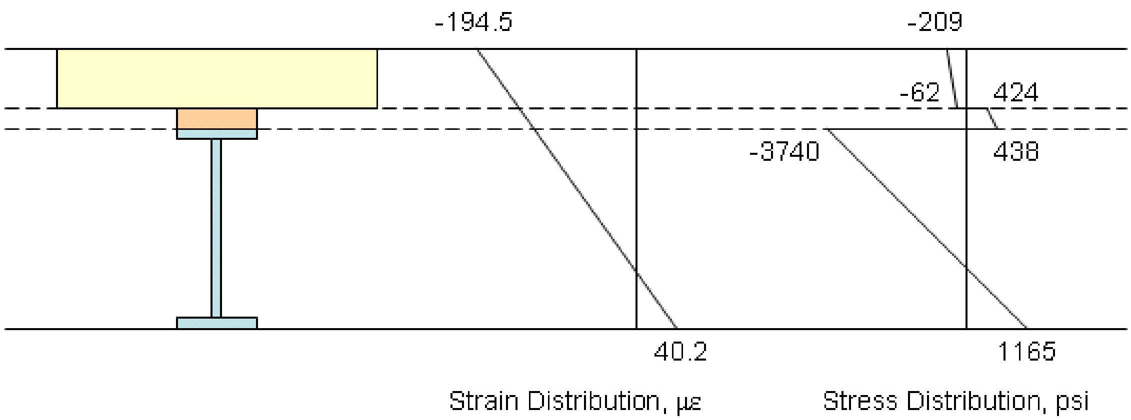

Figure 4. Typical Distributions of Stress and Strain for Steel Girder Bridge Models at End of Service Life

For each bridge model and different amount of initial post-tensioning in the deck, the final stresses at the top, middle, and bottom of the concrete deck at midspan after accounting for the time-dependent effects in the concrete were tabulated (Bowers, 2007). Although these calculations were performed at midspan, the changes in stress throughout the depth of the concrete deck are constant along the length of the simple span which experiences uniform changes in curvature. For the simple span steel girder bridges, it was expected that the worst location for potential tensile stresses in the concrete would be along the bottom of the deck throughout the span, since this is where the steel girder provides the greatest restraint of creep and shrinkage in the concrete deck. These predictions were verified by the results, which showed that in each of the 24 parametric studies, the compressive stresses were highest at the top of the bridge deck, and the stresses became less compressive or even somewhat tensile from the top of the deck to the bottom of the deck.

In order to maintain compression throughout the depth of the concrete deck at midspan, at least 200 psi of initial post-tensioning in the precast panels was required. The initial compressive stress of 200 psi resulted in a minimum of 39-62 psi residual compression in the precast panel joints in each of the six models. Larger amounts of initial post-tensioning were needed to obtain greater amounts of residual compression in the concrete deck. Figure 5 illustrates the relationship between the most tensile final stress (located at the bottom of the concrete deck in each case) and the span length at each of the four levels of initial post-tensioning.

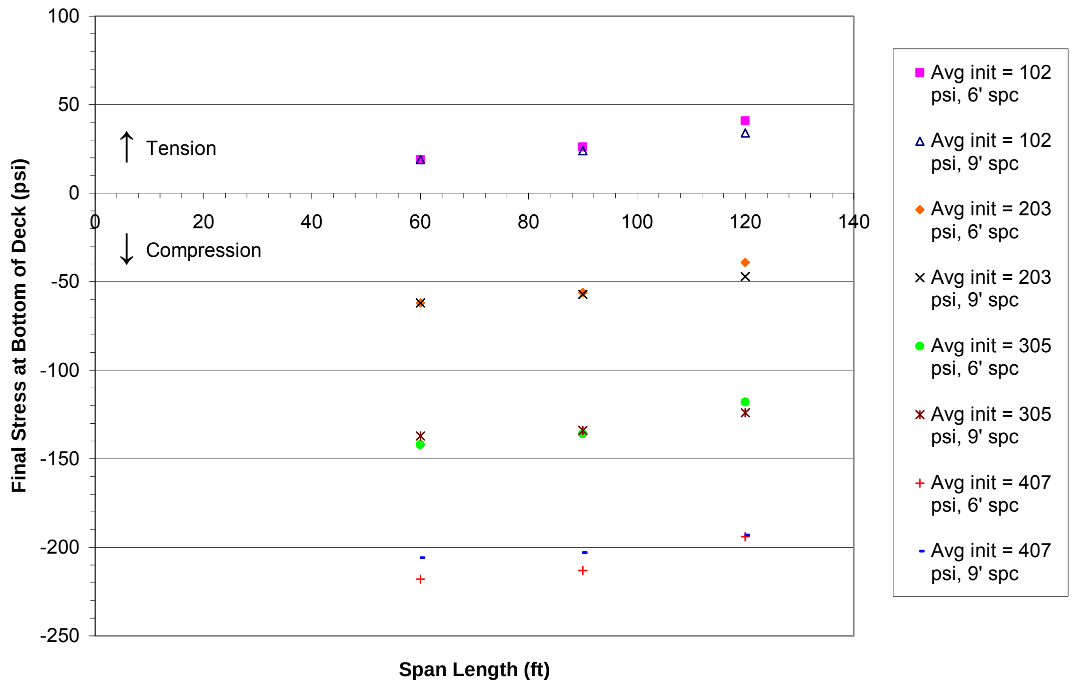

Figure 5. Final Deck Stress vs. Span Length for Simple Span Steel Girder Models

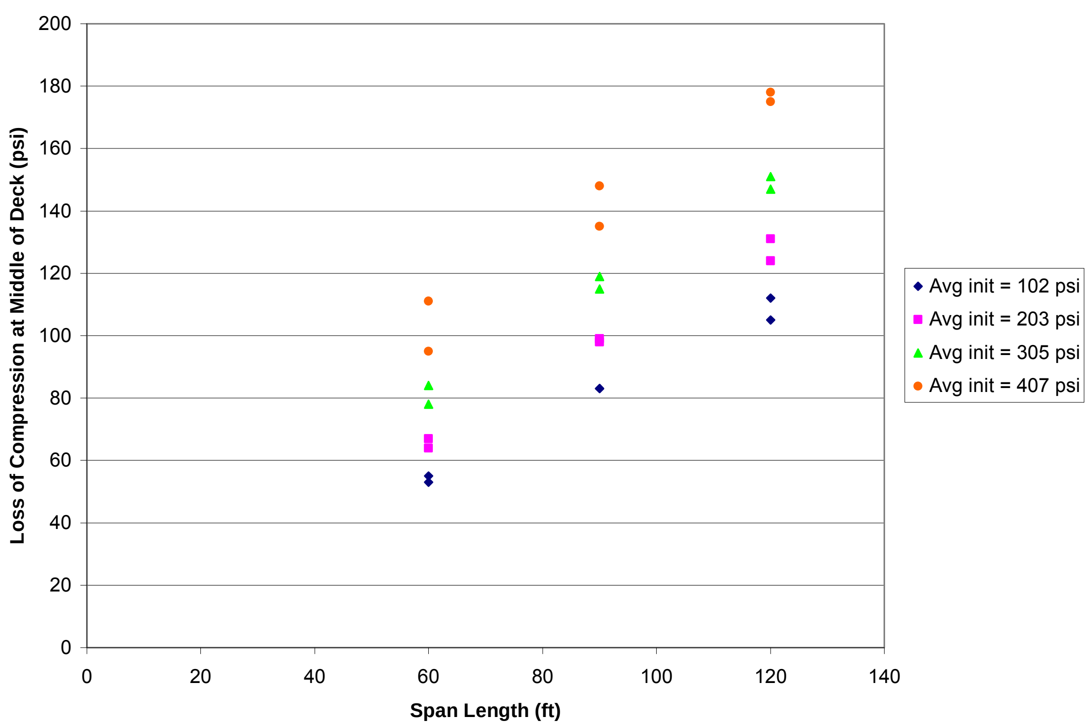

Figure 6. Net Loss of Compression at Middle of Deck for Simple Span Steel Girder Models

Figure 5 indicates a linear relationship between the span length and the critical deck stress at the end of service for the simple span steel girder bridges. The final stresses were not significantly affected by the girder spacing (6 ft or 9 ft). The figure also shows that at each different level of initial post-tensioning, the net loss of compression in the deck increases with span length. These losses are better illustrated in Figure 6.

The behavior illustrated in Figures 5 and 6 for the simple span steel girder bridges is logical. While the steel girder sizes increase proportionally with span length, the size of the effective deck cross sections at 6 ft. and 9 ft. girder spacings stay the same. Therefore, as the deck becomes less stiff relative to the girder with increasing girder sizes, the steel girder restrains the creep and shrinkage of the deck concrete more, causing it to experience greater losses of compression.

Bridges with Prestressed Concrete Girders

The second set of parametric studies was performed for simple span bridges with prestressed concrete girders and a precast concrete deck. The precast decks in each of the 23 different prestressed concrete girder bridges were post-tensioned to stresses ranging from 100-330 psi in increments of approximately 100 psi. Tabulate results for PCBT and AASHTO girders are given in Bowers (2007).

Due to the greater complexity of a prestressed concrete girder composite cross section, predicting the time-dependent behavior of these simple span bridges was much less straightforward than for the steel girder bridges. In the prestressed concrete girder models, the negative moment due to the upward camber of the girder counteracted the positive moments caused by the girder and deck self weights. The behavior of each system was further complicated by the time-dependent losses occurring at different rates in the girder and deck concretes of different ages. Therefore, the results of the prestressed concrete girder bridge parametric studies were much more dependent on the specific dimensions and characteristics of each model than in the steel girder bridges.

PCBT Girder Bridge Analyses

In all of the 33 simple span PCBT girder parametric studies, the entire depth of the bridge deck remained in compression at the end of the bridge service life. In each of these models except one, the compressive stresses in the bridge deck at the end of service were highest at the bottom of the deck, and became less compressive from the bottom to the top of the deck. In order to maintain compression throughout the depth of the concrete deck at midspan, at least 100 psi of initial post-tensioning in the precast panels was required. The initial compressive stress of 100 psi resulted in minimum residual compressive stresses ranging from 7 psi to 260 psi in the precast panel joints in the PCBT girder models. Figure 7 illustrates the relationship between the final stress at the middle of the concrete deck and the span length at the three different levels of initial post-tensioning in the simple span PCBT girder models.

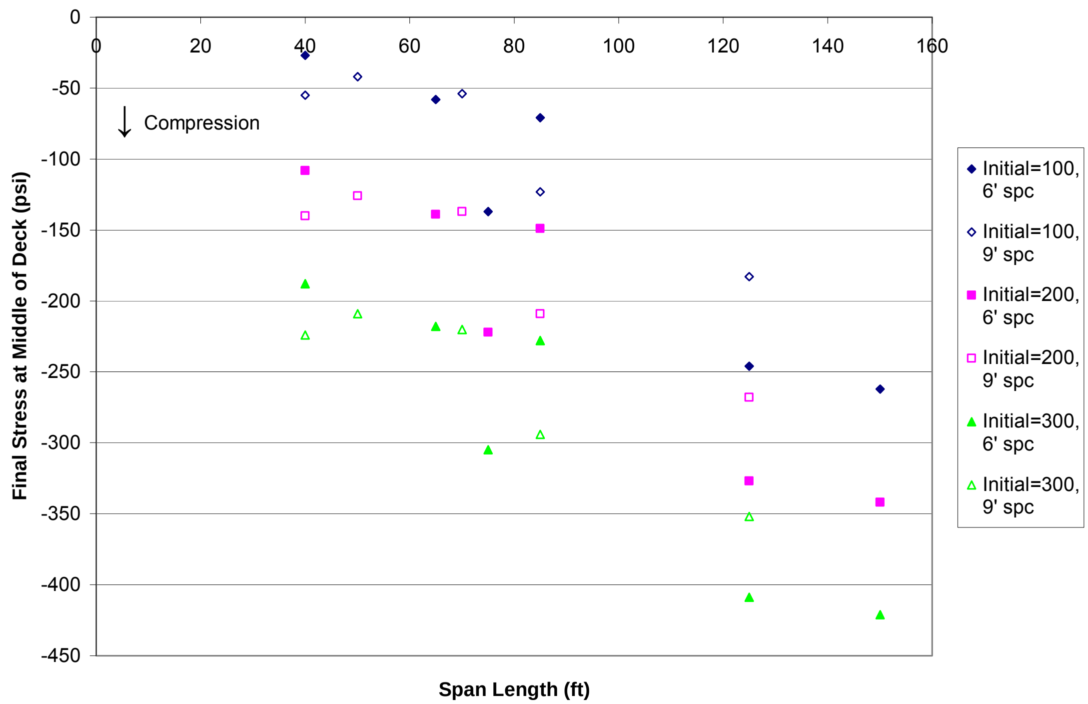

Figure 7. Final Deck Stress vs. Span Length for Simple Span PCBT Girder Models

Figure 7 shows a general trend of increasing residual compression in the concrete deck with increasing span length at each level of initial post-tensioning for both the 6 and 9 ft girder spacings. This behavior is the opposite of the trend observed in the simple span steel girder models, which showed decreasing residual compression in the concrete deck with increasing span length at each level of initial post-tensioning. While most of the concrete bridge decks in the simple span PCBT girder models experienced a net loss of compression from the time of post-tensioning to the end of service but still remained in compression, a few of the models with longer span lengths underwent an overall gain in compression during this time. Whereas the steel girders do not creep and shrink, the initial compression present in the concrete girders probably plays a role in helping the concrete girder bridge decks to lose a smaller amount of compression, or even gain some compression, by the end of service life. Figure 8 shows the net change in compressive stress at the middle of the deck for the simple span PCBT girder models.

AASHTO Girder Bridge Analyses

In the 36 simple span AASHTO girder parametric studies, the bridge decks again all remained in compression throughout their depths at the end of service life of each bridge model. In order to maintain compression throughout the depth of the concrete deck at midspan, at least 100 psi of initial post-tensioning in the precast panels was required. The initial compressive stress of 100 psi resulted in minimum residual compressive stresses ranging from 85 psi to 321 psi in the precast deck panels in the AASHTO girder models.

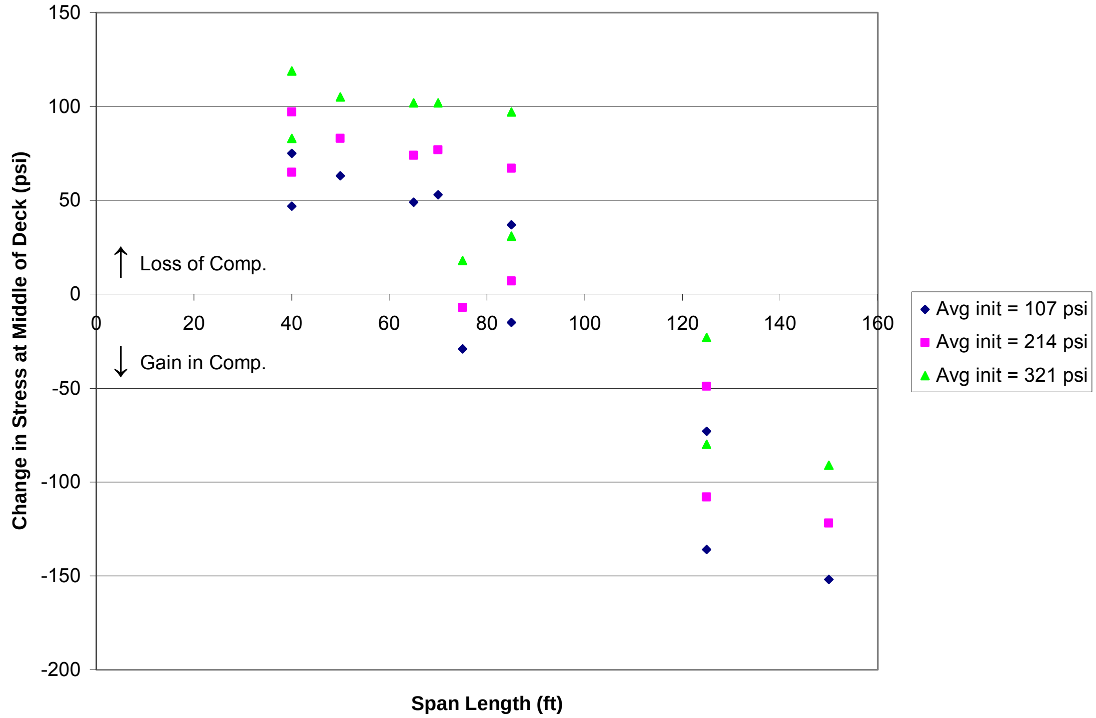

Figure 8 Net Change in Compression at Middle of Deck for PCBT Girder Models

CONTINUOUS SPAN MODELS

Bridges with Steel Girders

The two and three span continuous models with steel girders exemplified behavior similar to the simple span steel girder models, but included much additional tension in the concrete deck due to negative moments caused by live loads and the restraint of downward deflection at the piers. Tabulated results of the parametric studies for the two and three span continuous steel girder bridge models are presented in Bowers (2007). The tables give the final stresses at the top of the concrete deck both with and without the tension due to live loads for each level of initial post-tensioning applied. All stresses given for the two and three span continuous bridges are located at the interior support(s), which was assumed to be the critical location because of the maximum negative moments created there by live loads and restraint moments. Results provided in these tables for the two-span systems are illustrated graphically in Figures 9 and 10.

Figures 9 and 10 show the much larger losses of compression generated in the concrete decks of the two and three span continuous steel girder bridges are than the losses which occurred in the simple span steel girder models. After comparing the respective two and three span graphs with and without the stress due to live loads, it is clear that the live loads contribute a significant portion of the tensile stress present in the concrete deck at the interior supports.

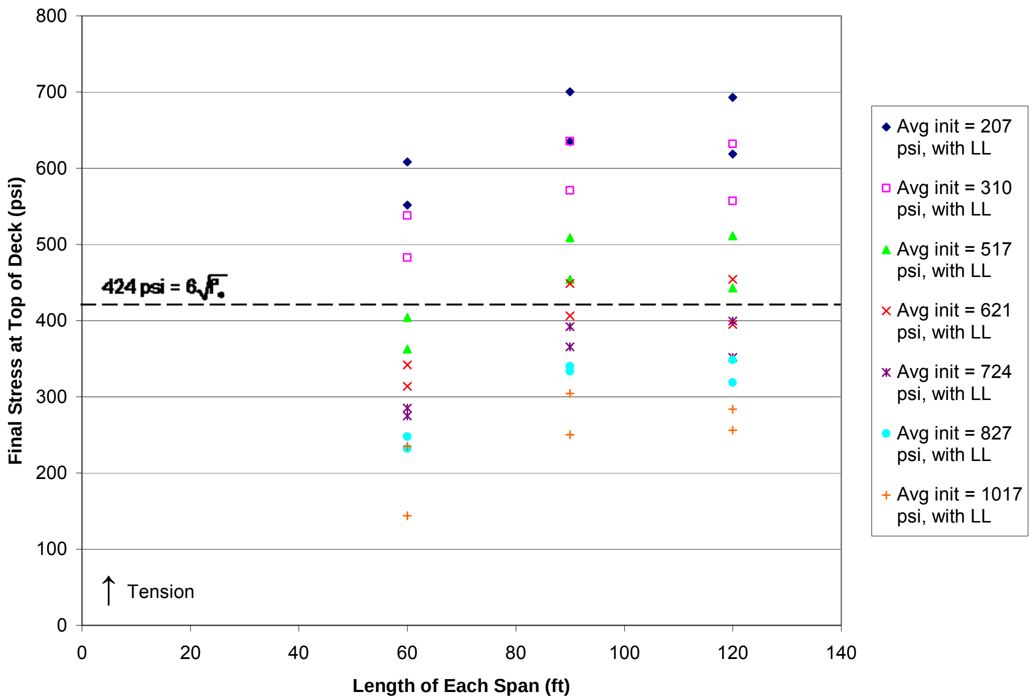

Figure 9. Final Stresses for Two-Span Continuous Steel Girder Bridges, Including Live Load

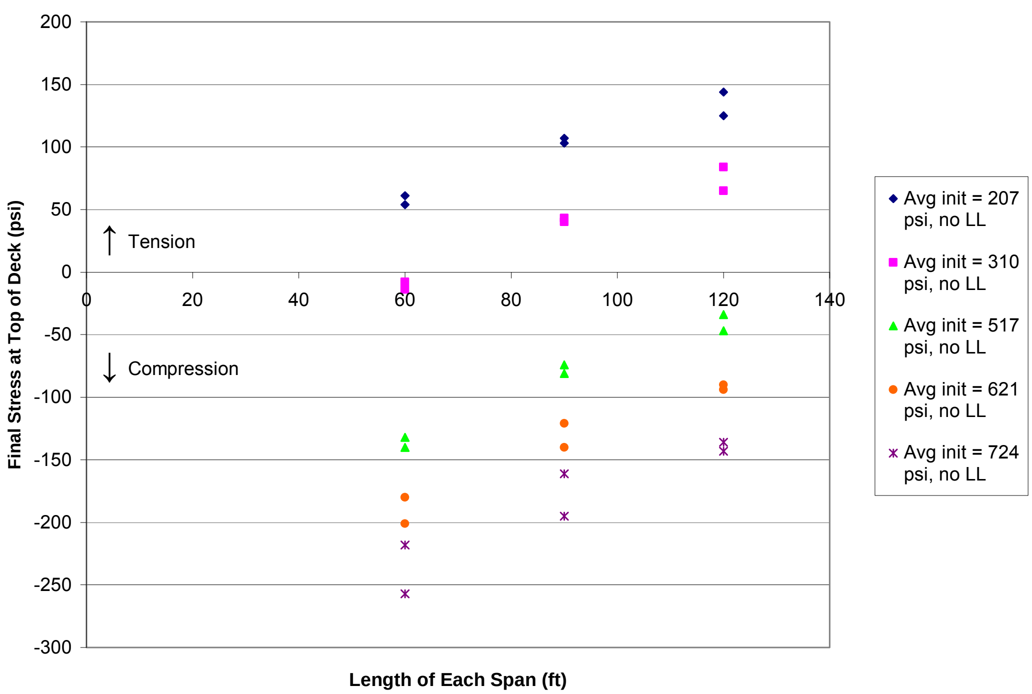

Figure 10. Final Stresses for Two-Span Continuous Steel Girder Bridges, Not Incl. Live Load

To provide reasonable recommendations for precast deck panel post-tensioning in the two and three span continuous steel girder models, the AASHTO LRFD limits regarding tensile stresses in concrete were incorporated. Table 5.9.4.2.2-1 in LRFD establishes a tension limit for the types of bridges considered in this research and "subjected to not worse than moderate corrosion conditions;" this limit is given in Equation 23:

$$\sigma_t = 0.19\sqrt{f'_c} \tag{23}$$

where:

f'_c is the concrete compressive strength in ksi.
Equation 23 is equivalent to $6\sqrt{f'_c}$ with f'_c in psi.

For the 5000 psi concrete panels used in this research, equation 23 produces a tensile stress limit of 425 psi. Based on the results illustrated in Figure 10 (the two span continuous system with live load), an initial compressive stress of about 620 psi must be provided in the precast concrete deck of a two span continuous steel girder bridge to prevent tensile stresses exceeding the limit 425 psi under time-dependent effects and live loads. For three span continuous steel girder bridges, an initial compressive stress of about 500 psi must be provided in the precast concrete deck to prevent tensile stresses exceeding the limit of 425 psi under time-dependent effects and live loads. These initial compressive stresses are provided by longitudinal post-tensioning in the precast concrete deck. In addition to keeping the maximum deck stresses below the tensile limit, these initial levels of post-tensioning also keep the deck in compression under permanent loads and loads induced from time dependent effects in the concrete.

Bridges with Prestressed Concrete Girders

The two and three span continuous models with prestressed concrete girders behaved differently than the simple span concrete girder models, but were also less affected by the live loads than the continuous steel girder bridges. Tables in Bowers (2007) show the results of the parametric studies for the two and three span continuous prestressed concrete girder bridge models, respectively. Selected results are summarized graphically in Figures 11 and 12. Additional figures which illustrate results for the two and three span continuous PCBT and AASHTO girder models are presented in Bowers.

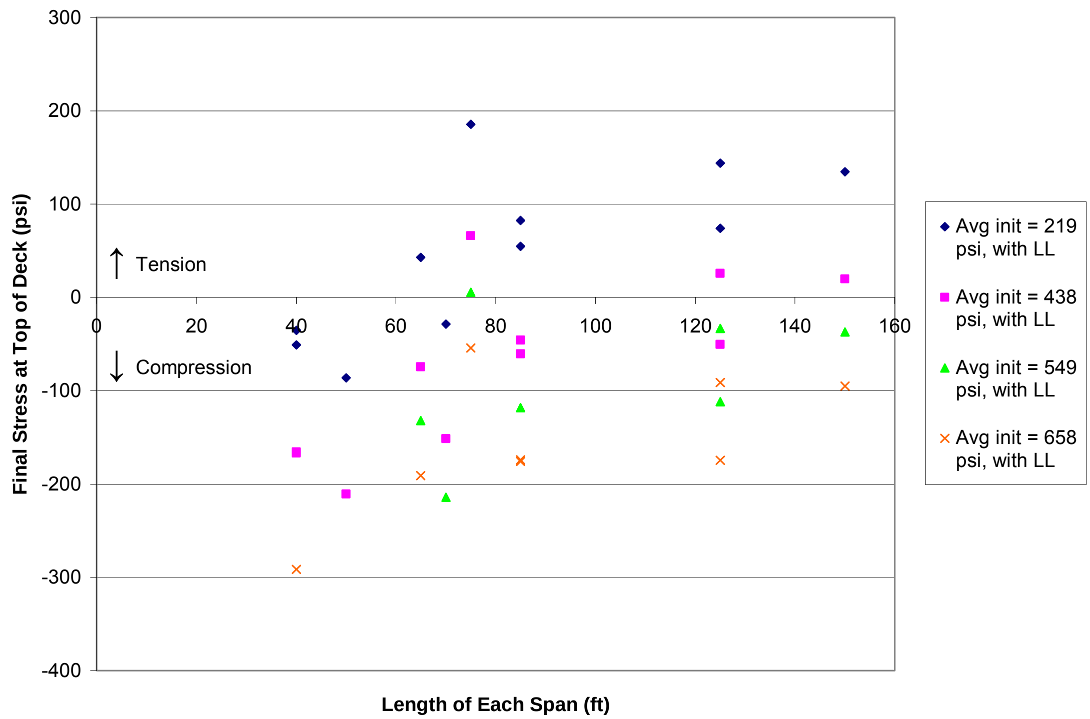

Figure 11. Final Stresses for Two-Span Continuous PCBT Girder Bridges, Incl. Live Load

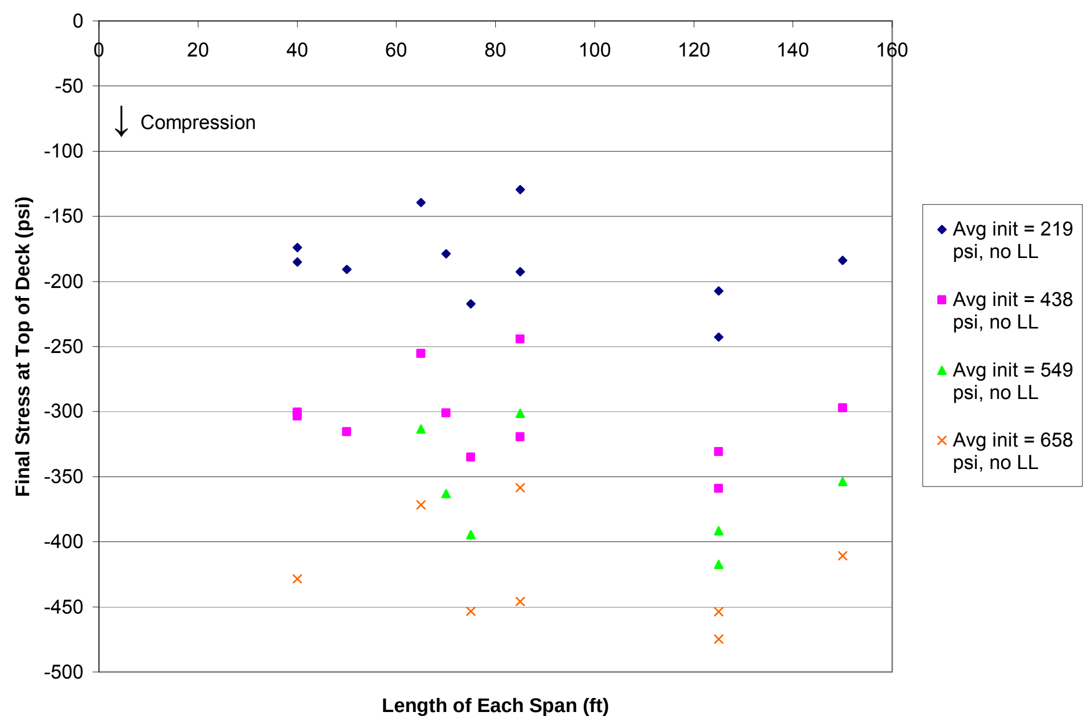

Figure 12. Final Stresses for Two-Span Continuous PCBT Girder Bridges, Not Incl. Live Load

Unlike the continuous steel girder models, the maximum tensile stress for all bridges with prestressed girders was less than 300 psi. This result was obtained from the smallest applied initial compressive stress of about 220 psi. In two and three span continuous bridges with PCBT or AASHTO girders, only 200 psi of initial compression is needed to keep the precast concrete deck stresses well under the tensile stress limit of 425 psi after time-dependent effects and live loads are considered. In addition to keeping the maximum deck stresses below the tensile limit, these initial levels of post-tensioning also keep the deck in compression under permanent loads and loads induced from time dependent effects in the concrete. This preservation of compression in the deck is depicted in Figure 12 which does not include live loads.

CONCLUSIONS AND RECOMMENDATIONS

SUMMARY

To facilitate the implementation of these full-depth precast bridge deck systems, designers need an easy, straightforward method or guidelines for determining the amount of longitudinal post-tensioning required in the bridge deck in order to keep the transverse joints in compression. Prior to this research, a handful of recommendations for longitudinal post-tensioning in precast bridge decks were presented, but these suggestions were based on the results of a limited number of laboratory tests or finite element model results. While previous models were limited to the use of steel girders, this research incorporates two different types of prestressed concrete girders as well as steel girders. In addition, the results of this research offer two different options for calculating the required amount of initial compression in a precast concrete bridge deck, which include 1) estimating the required initial compression from the general guidelines proposed, or 2) implementing the age-adjusted effective modulus method via the corresponding model developed and used to perform this research.

DESIGN RECOMMENDATIONS

In order to provide simple guidelines for use by designers, a single value of required initial post-tensioning was determined for each different type of bridge model investigated in this research. These guidelines are presented in Table 5.

As shown in Table 5, the recommendations for all of the three span continuous bridges were expanded to include three or more continuous spans. This modification was made based on the assumption that each additional span would theoretically provide more restraint against potential tension at the top of the deck over the interior supports, therefore reducing the successive amounts of initial compression required in the deck. This theory is already exemplified by the reduction from 650 psi to 500 psi of initial compression needed from two to three continuous steel girder spans.

Table 5. Recommended Values of Initial Post-Tensioning

Girder Type	Number of Spans	Required Initial P/T (psi)
Steel	1	200
	2	650
	3 or more	500
PCBT	1	200
	2	200
	3 or more	200
AASHTO	1	200
	2	200
	3 or more	200

As an alternative to the general guidelines provided in Table 5, the designer may also choose to implement the modeling procedure developed and used in this research to calculate a more specific initial compressive stress required in the precast deck of his or her bridge structure. This option may be productive when a given bridge cross section differs enough from the parametric studies performed in this research that the general guidelines provided here may be overly conservative or unconservative. As a second alternative to using the general guidelines provided, the designer may also be able to interpolate a more exact level of initial post-tensioning appropriate for his or her bridge configuration from the tables and graphs of results presented in this paper.

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NOTATION

a	=	distance between deck centroid and haunch centroid, in.
A_d	=	cross-sectional area of the effective deck, in^2
A_g	=	gross area of the girder, in^2
A_{gn}	=	net area of concrete girder, in^2
A_h	=	cross-sectional area of the haunch, in^2
A_{psg}	=	total area of prestressing strands in girder, in^2
A_{ptd}	=	total area of post-tensioning strands in deck, in^2
b	=	distance between deck centroid and girder centroid, in.
c	=	distance between deck centroid and centroid of girder prestressing strands, in.
e_g	=	eccentricity of prestressing strands in the concrete girder, in.
E_d	=	modulus of elasticity of the deck, ksi
E_g	=	modulus of elasticity of the girder, ksi
E_h	=	modulus of elasticity of the haunch, ksi
E_{psg}	=	modulus of elasticity of girder prestressing strands, ksi
E_{ptd}	=	modulus of elasticity of deck post-tensioning strands, ksi
I_d	=	moment of inertia of the effective deck, in^4
I_g	=	moment of inertia of the girder, in^4
I_{gn}	=	net moment of inertia of concrete girder, in^4
I_h	=	moment of inertia of the haunch, in^4
M_{go}	=	initial moment in concrete girder, kip-in
M_{goc}	=	initial moment in concrete girder for composite phase, kip-in
N_{do}	=	initial force at centroid of deck, kips
N_{doc}	=	initial force at centroid of deck for composite phase, kips
N_{go}	=	initial force at centroid of concrete girder, kips

N_{goc}	=	initial force at centroid of concrete girder for composite phase, kips
χ	=	constant curvature of span, strain/inch
Δf_{pR}	=	change in stress due to relaxation in girder or deck strands over a given time interval
ΔM_d	=	change in moment in deck
ΔM_g	=	change in moment in girder
ΔM_h	=	change in moment in haunch
ΔN_d	=	change in force at centroid of deck
ΔN_g	=	change in force at centroid of girder
ΔN_h	=	change in force at centroid of haunch
ΔN_{psg}	=	change in force at centroid of prestress in girder
ΔN_{ptd}	=	change in force at centroid of post-tensioning in deck
$\Delta \chi$	=	change in curvature
$\Delta \varepsilon_d$	=	change in strain at centroid of deck
$\Delta \varepsilon_g$	=	change in strain at centroid of girder
$\Delta \varepsilon_h$	=	change in strain at centroid of haunch
$\Delta \varepsilon_{psg}$	=	change in strain at centroid of prestress in girder
$\Delta \varepsilon_{ptd}$	=	change in strain in post-tensioning strands in deck
ε_{shd}	=	shrinkage strain in deck concrete over a given time interval
ε_{shg}	=	shrinkage strain in girder concrete over a given time interval
ε_{shh}	=	shrinkage strain in haunch concrete over a given time interval
μ_d	=	aging coefficient for the deck
μ_g	=	aging coefficient for the girder concrete
μ_h	=	aging coefficient for the haunch
ϕ_d	=	creep coefficient for deck concrete over a given time interval
ϕ_g	=	creep coefficient for girder concrete over a given time interval
ϕ_h	=	creep coefficient for haunch concrete over a given time interval