Full-depth precast concrete deck panels can be used for rapid replacement of deteriorated bridge decks.1,2 These panels offer many advantages over cast-in-place concrete decks, including reduced commuter delays. To be durable, panel-to-panel connections must be carefully designed to avoid cracking, leaking, and spalling at the connection and limit subsequent deterioration of prestressing or reinforcing steel in the connection and supporting girders.3 One solution to reduce the probability of cracking and the consequent durability problems is to add a compressive force in the deck panels with posttensioning.

In typical precast concrete deck panel construction, the panels are set in place, the connection grout or concrete is placed, and the posttensioning applied. Subsequently, the horizontal shear connection between the panels and girders is made, which creates the composite system. Over time, creep, shrinkage, relaxation, and stress redistribution in the composite system, reduce the initial precompression. In addition, traffic loads will induce stresses in the panels. Relatively little posttensioning is needed on simply supported spans because the live loads induce compressive stresses in the deck. On continuous-span bridges, live loads induce tensile stresses in the deck over interior supports. If the posttensioning force is not great enough, the joints may crack and result in durability problems.

This paper uses the age-adjusted effective modulus method to determine the initial compressive stress to avoid cracking and leaking in precast, posttensioned concrete bridge deck panels.

For simple-span bridges, an initial precompression of 200 to 300 psi (1.4 to 2.1 MPa) is adequate; in three-span bridges, 200 to 750 psi (1.4 and 5.2 MPa) of initial prestress is needed.

Two-span systems are the most critical, with some bridges requiring close to 1000 psi (6.90 MPa) of initial prestress.

For continuous span bridges, accelerated construction schedules necessitate substantial increases in initial compressive stress.
Place the panels in position on the girders and adjust their elevation with leveling bolts or other method.

Form and cast panel-to-panel transverse connections.

Stress the panel system end to end with longitudinal posttensioning.

Create composite behavior by forming and casting the haunch and horizontal shear connector blockout pockets with grout or concrete.

The required posttensioning has not been standardized. The American Association of State Highway and Transportation Officials’ AASHTO LRFD Bridge Design Specifications recommends a minimum average effective prestress of 250 psi (1.72 MPa) for posttensioned precast concrete decks. Others have shown that this is not sufficient, particularly on continuous bridges.\(^5,6\) One finite element study suggested 450 psi (3.10 MPa) for all continuous bridges and 200 psi (1.38 MPa) for simple-span bridges.\(^7\) The required initial prestress to keep connections from cracking is dependent on the girder size, girder spacing, and other factors. A simplified method of analysis is needed to determine the required initial prestress to ensure durability after all losses and stress redistributions.

This paper presents a method to determine the changes in stresses within a composite system comprising posttensioned precast concrete deck panels, a cast-in-place concrete haunch, and steel girders. An age-adjusted effective modulus method, with no slip at the slab-steel girder interface, was used to evaluate time-dependent stress redistribution in composite systems. Steel girders do not shrink and their creep is negligible compared with concrete girders; thus they provide the worst-case restraint for a composite system. Based on the experimental research presented in part 1 of this paper,\(^8\) an allowable tensile stress across connections of \(3\sqrt{f_c'}\) (where \(f_c'\) is the smaller of the connection grout or panel concrete compressive strength in psi) (0.25 \(\sqrt{f_c'}\) in MPa) was proposed to keep the joints from leaking. A set of 22 in-service bridges with steel girders was then analyzed to determine the initial stress needed during a hypothetical bridge redecking with deck panels. The results give an indication of the stress that is needed to ensure the durability of the deck panels.

### Time-dependent behavior of full-depth deck panels

The typical sequence of construction for posttensioned full-depth precast concrete deck panels is as follows:

1. Place the panels in position on the girders and adjust their elevation with leveling bolts or other method.
2. Form and cast panel-to-panel transverse connections.
3. Stress the panel system end to end with longitudinal posttensioning.
4. Create composite behavior by forming and casting the haunch and horizontal shear connector blockout pockets with grout or concrete.

The deck panels begin with an initial longitudinal prestress. Unrestrained creep and shrinkage reduce this prestress in the interval between stressing and creating the composite system. The composite deck continues to shrink and creep due to the initial prestress. However, the girders restrain the shortening. Because the top of the girder and the bottom of the deck must strain compatibly and plane sections must remain plane, the girder imposes a tensile force on the deck while the deck imposes an equal and opposite compressive force on the girder. In a simply supported system, this results in the composite girder deflecting downward. The restraining force the girder imposes on the deck reduces the precompression in the deck. This could result in the deck losing a considerable amount of its original prestress force. Figure 1 illustrates this concept.

In a continuous system, the shortening of the deck causes the whole system to deflect downward. Because this movement is prevented at the interior supports, the interior support reactions increase and the exterior reactions decrease. This results in the development of negative restraint moments, which cause additional tensile stresses in the deck. This effect also reduces the prestress in the deck.

Finally, in continuous systems, live loads also cause negative moments over the piers, which cause tensile stresses in
the deck. The final stress in the deck panel is then the sum of the initial precompression, the loss of precompression from redistribution, the tension from restraint moments, and the tension from live loads. Based on the experimental study presented in the first paper, for optimum performance, this total stress should be kept below $3\sqrt{f_c}$ in psi ($0.25\sqrt{f_c}$ in MPa), where $f_c$ is the smaller of the concrete and connection filler material specified 28-day compressive strength. The challenge is to develop a relatively straightforward method to calculate the time-dependent changes in stress in the system.

The method for estimating prestress losses presented in AASHTO LRFD specifications is not applicable to precast concrete deck panels. The method was derived specifically for pretensioned beams with cast-in-place concrete composite decks and is not appropriate for deck panels.

### Method of analysis

This section describes a time-dependent analysis using an age-adjusted effective modulus method, which can estimate the effects of creep, shrinkage, and relaxation on the change in compressive stress in the deck panels. A system of equations was written to satisfy equilibrium, compatibility, and the constitutive relationships for each time interval under consideration. For multispans, it was also necessary to consider the effect of continuity and live loads, especially at the interior supports where they cause tensile stress in the deck.

Other researchers have examined similar stress redistributions in composite systems. Dezi et al. proposed a two-step age-adjusted effective modulus method with flexible shear connections. The result from the analysis showed good comparison with the general step-by-step method previously proposed by the authors for practical application. Bradford showed that slip at the slab-girder interface is insignificant compared with time-dependent deformation; therefore slip is neglected in this study.

For this analysis, the deck life was divided into two time intervals, with day 0 being the day the panels were cast:

- **Construction phase**: from time of posttensioning the deck to the start of composite action between the girders and deck panels
- **Service phase**: from start of composite action to end of service life

The system of equations was written to model precast concrete deck behavior, and the equations were solved simultaneously. Tensile force, elongation, and sagging moment were considered positive while compressive force, shortening, and hogging moment were considered negative.

### Equations for time-dependent analysis

The construction phase was considered to be from the time of posttensioning the deck to the start of composite action. The haunch was assumed to be cast immediately after the deck panels were posttensioned, and composite action was assumed to begin shortly thereafter. The prestressing force along the length of the tendon was calculated by determining the immediate losses due to friction, wobble, and seating. For a straight tendon in a deck panel, only wobble reduces the tendon force over the length of the tendon during stressing. Seating loss reduces the force along the entire tendon for short tendon lengths and over some part of the tendon for longer tendon lengths. Although the tendon force varies along the length of the bridge, for the time-dependent analysis, the average tendon force was used as a starting point.

During the interval between stressing and start of composite action, creep and shrinkage reduce the tension in the prestressing strand, thereby reducing the compressive force in the deck concrete. Four equations are used to solve for the loss of precompression. Equation (1) is the equation of internal equilibrium; the change in strand force $\Delta N_{ptd}$ must be equal and opposite to the change in axial force in the deck $\Delta N_d$. Assuming the tendon is grouted and perfect bond exists, the change in strand strain $\Delta \varepsilon_{ptd}$ is equal to the change in deck strain $\Delta \varepsilon_d$ at the level of strand (Eq. [2]). In Eq. (3), the change in deck strain $\Delta \varepsilon_d$ is the sum of the creep strain associated with the initial strain in the deck, the elastic and creep strain associated with the change in force in the deck, and the deck shrinkage strain $\varepsilon_{shd}$. In Eq. (4), the numerator represents the difference between the total change in axial force in the prestressing strand and the change in axial force due to strand relaxation. The change in axial force due to relaxation is subtracted because it is not associated with a change in strain.

#### Equilibrium

$$\Delta N_d + \Delta N_{ptd} = 0$$  \hspace{1cm} (1)

#### Compatibility

$$\Delta \varepsilon_d = \Delta \varepsilon_{ptd}$$  \hspace{1cm} (2)

#### Constitutive relationship

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

where

$N_{do}$ = initial force at centroid of deck

$A_d$ = cross-sectional area of the effective deck
creep and shrinkage, causing a redistribution of forces to the steel girders; creep in the girders was neglected. A system of 12 equations is written to solve for 12 unknowns (Eq. [2] and [6] through [16]). Figure 2 illustrates the unknown changes in forces, moments, strains, and curvature.

Equilibrium

\[ \Delta N_d + \Delta N_h + \Delta N_g + \Delta N_{ptd} = 0 \]  

where

\[ \Delta N_d = \text{change in force at centroid of deck} \]
\[ \Delta N_h = \text{change in force at centroid of girder} \]
\[ \Delta N_g = \text{change in force at centroid of haunch} \]

These equations can be simplified into Eq. (5):

\[ \Delta N_{ptd} = \frac{N_{ptd} \phi_d + \Delta f_{pt} A_{ptd}}{A_{ptd} E_{ptd}} \]  

where

\[ \Delta f_{pt} = \text{change in stress due to relaxation of deck strands over a given time interval} \]
\[ A_{ptd} = \text{total area of posttensioning strands in the effective deck} \]
\[ E_{ptd} = \text{modulus of elasticity of the prestressing strand} \]

All three terms in the numerator will be negative (shortening) strains, so the change in prestrain force will also be negative, or a decrease in tension.

The service phase was considered to be from the start of composite action to the end of service life. In this interval, the precast concrete deck and haunch undergo creep and shrinkage, causing a redistribution of forces to the steel girders; creep in the girders was neglected. A system of 12 equations is written to solve for 12 unknowns (Eq. [2] and [6] through [16]). Figure 2 illustrates the unknown changes in forces, moments, strains, and curvature.

Equilibrium

\[ \Delta N_d + \Delta N_h + \Delta N_g + \Delta N_{ptd} = 0 \]  

where

\[ \Delta N_d = \text{change in force at centroid of deck} \]
\[ \Delta N_h = \text{change in force at centroid of girder} \]
\[ \Delta N_g = \text{change in force at centroid of haunch} \]

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Equilibrium

\[ \Delta N_d + \Delta N_h + \Delta N_g + \Delta N_{ptd} = 0 \]  

where

\[ \Delta N_d = \text{change in force at centroid of deck} \]
\[ \Delta N_h = \text{change in force at centroid of girder} \]
\[ \Delta N_g = \text{change in force at centroid of haunch} \]

These equations can be simplified into Eq. (5):

\[ \Delta N_{ptd} = \frac{N_{ptd} \phi_d + \Delta f_{pt} A_{ptd}}{A_{ptd} E_{ptd}} \]  

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\[ A_{ptd} = \text{total area of posttensioning strands in the effective deck} \]
\[ E_{ptd} = \text{modulus of elasticity of the prestressing strand} \]

All three terms in the numerator will be negative (shortening) strains, so the change in prestrain force will also be negative, or a decrease in tension.

The service phase was considered to be from the start of composite action to the end of service life. In this interval, the precast concrete deck and haunch undergo
where

\[ \Delta \epsilon_h = \text{change in strain at centroid of haunch} \]

\[ \Delta \chi = \text{change in curvature} \]

\[ \Delta \epsilon_d = \Delta \epsilon_g - b \Delta \chi \quad (9) \]

where

\[ \Delta \epsilon_g = \text{change in strain at centroid of girder} \]

Constitutive relationships

\[ \Delta \epsilon_d = \frac{N_{doc}}{A_d E_d} \phi_d + \frac{\Delta N_d}{A_d E_d} (1 + \mu_d \phi_d) + \epsilon_{shd} \quad (10) \]

where

\[ N_{doc} = \text{initial force at centroid of deck for composite phase} \]

\[ \Delta \epsilon_h = \frac{\Delta N_g}{A_h E_h} \quad (11) \]

where

\[ A_g = \text{cross-sectional area of girder} \]

\[ E_g = \text{modulus of elasticity of girder} \]

\[ \Delta \epsilon_h = \frac{\Delta N_h}{A_h E_h} (1 + \mu_h \phi_h) + \epsilon_{shh} \quad (12) \]

where

\[ A_h = \text{cross-sectional area of haunch} \]

\[ E_h = \text{modulus of elasticity of haunch} \]

\[ \mu_h = \text{aging coefficient for haunch} \]

\[ \phi_h = \text{creep coefficient for haunch concrete over a given time interval} \]

\[ \epsilon_{shh} = \text{shrinkage strain in haunch concrete over a given time interval} \]

\[ \Delta \epsilon_{pad} = \frac{\Delta N_{pad}}{A_{pad} E_{pad}} - \Delta f_{phk} A_{pad} \quad (13) \]

\[ \Delta \chi = \frac{\Delta M_d}{I_d E_d} (1 + \mu_d \phi_d) \quad (14) \]

where

\[ I_d = \text{moment of inertia of effective deck} \]

\[ I_h = \text{moment of inertia of haunch} \]

Because no external load is applied to the system, the net changes in the normal force and moment are zero (Eq. [6] and [7]). Strain compatibility and the assumption that plane sections remain plane allow three equations to relate the unknown changes in strain and curvature to one other (Eq. [2], [8], and [9]). The change in deck strain is calculated using Eq. (10), similar to Eq. (3) in the construction phase. Equation (11) represents the change in girder strain caused by the change in girder force. Equation (12) accounts for the change in haunch strain. The first term accounts for elastic strain and creep strain associated with the change in the force in the haunch and the second term represents the haunch shrinkage strain during the second interval. The change in prestressing strand strain is calculated using Eq. (13). The change in curvature caused by the change in moment in the deck and haunch and the creep curvature associated with the change in moment is calculated using Eq. (14) and Eq. (15), respectively. Equation (16) represents the change in curvature caused by the change in elastic moment in the steel girder. The haunch is included for completeness but can be neglected to simplify the method.

For simply supported bridges, the highest tensile stresses in the deck will occur due to time-dependent effects only. The live load causes additional compressive force in the deck panels. The most critical location for deck tension is along the bottom of the deck panels because this is where the girders provide the greatest restraint resulting in the maximum loss of precompression (Fig. 1).

For continuous multispan bridges, the entire bridge length was first evaluated as a simple-span case. For a concentric tendon profile, the change in curvature is constant along the bridge length. A restraint moment that develops because of an interior support can be computed easily using the force method. Using the change in curvature calculated with the system of equations, the displacement that would occur at support locations, if said supports were removed,
can be calculated. Then the force required to return these locations to zero displacement is calculated. An age-adjusted transformed moment of inertia is used in this calculation because these forces develop slowly. The live load moment envelopes per AASHTO LRFD specifications live load combinations were obtained using bridge engineering software. The stresses due to live load can be multiplied by the AASHTO LRFD specifications service III live load factor of 0.8, which is related to tension in prestressed concrete superstructures with the objective of crack control. Although the actual intention of this factor is to control the cracking in the tensile regions of prestressed concrete girders in positive bending, for this study the same factor of 0.8 was used for tension in the deck panel fibers in the region of negative moment. The resultant top-fiber deck stresses at the critical locations can then be compared with the tensile stress limit.

The method presented here was used by Bowers for both steel and concrete beams. She validated the results with a comparison to a time-dependent analysis done with commercially available finite element software by Sullivan.

**Recommended tensile stress limit**

A panel-to-panel connection that is always in compression is the most desirable situation, but economics and practical considerations may make this difficult to achieve. An acceptable tensile stress across the connections was investigated in an experimental study discussed in part 1 of this paper. Two prestress values were investigated through cyclic tests, and the connections that performed the best had total maximum tensile stress at the connections of 
\[ \frac{3}{8} \sqrt{f_c} \text{ psi} \text{ (0.25} \sqrt{f_c} \text{ in MPa).} \]

The connections with this maximum stress during cycling had a small amount of visible cracking but never leaked water. This led to the recommendation that the maximum allowable tensile stress at transverse connections of precast concrete deck panels be 
\[ \frac{3}{8} \sqrt{f_c} \text{ psi} \text{ (0.25} \sqrt{f_c} \text{ in MPa),} \]

where \( f_c \) is the smaller of the specified panel concrete or connection filler material compressive strength.

**Parametric study**

A parametric study on time-dependent effects was conducted based on information available for 22 bridges in Indiana (Table 1). These bridges have cast-in-place concrete decks but were studied as examples of bridges that might be redecked in the future with precast concrete deck panels. The sample includes simple-span, two-span, and three-span continuous bridges with span lengths ranging from 46 to 98 ft (14 to 30 m). To simplify the parametric study, the following assumptions were made:

- Full composite action was considered between the deck panels and girders.
- A straight tendon profile with zero eccentricity was used in the deck panels.
- Spans with equal lengths were used for multispan cases. Bridges with uniform girder sections were used. For three-span continuous bridges, the maximum span length was used to model all three spans. The longer spans resulted in higher negative moments and lower reactions due to restraint from the change in curvature.
- For multispan bridges, the critical deck panel connection was assumed at a distance of 4 ft (1.2 m) from the interior support and was checked for the greatest tensile stresses at the top of the deck. The offset is based on the practice of centering a typical panel width of 8 ft (2.4 m) over the interior support.
- Prestressing was assumed to take place when the deck panels were 55 days old.
- Full composite action was assumed to be imposed when the deck panels were 60 days old.
- The analysis was conducted for a single girder line with an integer number of prestressing strands added such that the allowable stress requirement was met.

**Material properties**

The rolled steel girders were assumed to have a modulus of elasticity equal to 29,000 ksi (2.0 \times 10^5 MPa). The precast concrete deck panels used in the analysis had a 28-day compressive strength of 5000 psi (35 MPa) with an aging coefficient of 0.7. The precast concrete deck panels were 8.5 in. (216 mm) thick, prestressed with \( \frac{1}{2} \) in. (13 mm) diameter Grade 270 (1860 MPa) low-relaxation strands. Each bridge had a 1 in. (25 mm) thick haunch with the same material properties as the deck panels. The prestressing strand modulus was 28,500 ksi (1.97 \times 10^6 MPa).

The creep and shrinkage of concrete deck panels and the grout in the haunch were estimated using AASHTO LRFD specifications creep and shrinkage models, as they have proved to be relatively accurate in the prediction of prestress losses in bridge girders. The creep and shrinkage models can be found in article 5.4.2.3 of the AASHTO LRFD specifications. In addition, the relaxation of the prestressing strand was calculated using Equation (5.9.5.4.2c-1) from the AASHTO LRFD specifications.

**Results**

**Example calculations for typical two-span bridge**

One example is presented to illustrate the steps required in the calculations and to show the relative magnitudes of
The first step is to determine the average stress in the concrete after jacking and seating. The jacking stress was 0.72 $f_{pu}$ (where $f_{pu}$ is the specified tensile strength of the prestressing steel), or 194.4 ksi (1340 MPa). Based on a wobble coefficient of 0.0002/ft (0.0007/m), and a total tendon length of 134 ft (40.8 m), the dead end stress is:

$$f_{ps,\text{dead}} = f_{ps,\text{live}}e^{-k\ell} = 194e^{-0.0002 \times (134)} = 189.3 \text{ ksi (1305 MPa)}$$

where

- $f_{ps,\text{dead}}$ is stress in the prestressing steel at the dead end (non-stressing end) of the tendon
- $f_{ps,\text{live}}$ is stress in the prestressing steel at the live end (stressing end) of the tendon

The process is iterative because a number of strands must first be assumed and all calculations performed, and then the final stress is checked. Depending on the result, the number of strands is increased or decreased and the calculations repeated until the final stress is acceptable. In this example, the final iteration, which required twelve 0.5 in. (13 mm) prestressing strands per girder line, is presented.
Next, the stresses caused by the restraint of downward deflection at the interior pier are calculated. The results of the calculation of the self-equilibrating stresses indicate a uniform curvature of \(7 \mu \epsilon /\text{in.} \) \((0.28 \mu \epsilon /\text{mm})\). If there were no interior support, this would result in a downward displacement of 2.3 in. \((58 \text{ mm})\) at the center support location. The force to return the location to zero displacement is calculated using the transformed section properties based on the age-adjusted modulus of elasticity for the deck and the grout concrete. This results in a transformed moment of inertia of 22,230 in. \(^4\) \((9.253 \times 10^9 \text{ mm}^4)\) based on the modulus of elasticity of the girder \((29,000 \text{ ksi } [200,000 \text{ MPa}])\) (Table 2).

At this point the first-stage time-dependent analysis can be performed. For the one-step equation (Eq. \([5]\)), the initial creep-producing force in the deck is calculated as:

\[ N_{dc} = f_{p,avg}A_{pd} = 185.2[(12)(0.153)] = 340 \text{ kip (1510 kN)} \]

where \(f_{p,avg} = \text{average stress in the prestressing steel, ksi (MPa)}\)

Because this is a compressive force in the deck, the sign convention is negative, -340 kip \((-1510 \text{ kN})\). Thus, the original compressive stress in the deck is -0.580 ksi \((-4.00 \text{ MPa})\).

The shrinkage over the 5-day construction phase is small: -9\(\mu \epsilon\). The relaxation loss was calculated to be -1.9 ksi \((-13 \text{ MPa})\). The change in tendon force is calculated using Eq. \([5]\):

\[ \Delta N_{pd} = \frac{(-340)(0.107)}{(586.5)(4030)} + \frac{(-9 \times 10^{-6}) + \left( \frac{-1.9}{28,500} \right)}{1 + \frac{0.7(0.107)}{(1.84)(28,500)}} \]

\[ = -4.50 \text{ kip (20 kN)} \]

Subtracting this from the initial precompression, the creep-producing force in the deck at the start of the next phase is -335.5 kip \((-1492 \text{ kN})\), and the average compressive stress in the deck is -0.572 ksi \((-3.94 \text{ MPa})\).

With -335.5 kip as the value of \(N_{doc}\) for the service phase and other known values as presented in Table 3, the system of 12 equations can be written and solved. Table 4 presents the results. Using the forces and moments in the deck, haunch, and girder, the distribution of stresses in the system can be calculated. The changes in stress in the deck are summed with the compressive stress at the end of the construction phase \(-0.572 \text{ ksi compression}\). Figure 3 presents the stress distribution. If this were a simply supported span, this would be the stress distribution due to prestress. Because this is a two-span example, the effects of continuity must also be considered.

---

### Table 2. Inputs for example bridge

<table>
<thead>
<tr>
<th>Input</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Span length</td>
<td>67 ft</td>
</tr>
<tr>
<td>Number of spans</td>
<td>2</td>
</tr>
<tr>
<td>Girder spacing</td>
<td>5.75 ft</td>
</tr>
<tr>
<td>Girder type</td>
<td>W36 × 150</td>
</tr>
<tr>
<td>Girder depth</td>
<td>35.86 in.</td>
</tr>
<tr>
<td>Deck thickness</td>
<td>8.5 in.</td>
</tr>
<tr>
<td>Top-flange width</td>
<td>12 in.</td>
</tr>
<tr>
<td>Haunch thickness</td>
<td>1 in.</td>
</tr>
<tr>
<td>Concrete compressive strength</td>
<td>5 ksi</td>
</tr>
<tr>
<td>Concrete unit weight</td>
<td>150 lb/ft(^3)</td>
</tr>
<tr>
<td>Haunch grout compressive strength</td>
<td>5 ksi</td>
</tr>
<tr>
<td>Deck age at stressing</td>
<td>55 days</td>
</tr>
<tr>
<td>Deck age at composite action</td>
<td>60 days</td>
</tr>
<tr>
<td>Relative humidity</td>
<td>70%</td>
</tr>
<tr>
<td>Number of prestressing strands</td>
<td>12</td>
</tr>
<tr>
<td>Jacking stress</td>
<td>194 ksi</td>
</tr>
</tbody>
</table>

Note: 1 in. = 25.4 mm; 1 ft = 0.305 m; 1 lb = 4.448 N; 1 ksi = 6.895 MPa.

Next, the stresses caused by the restraint of downward deflection at the interior pier are calculated. The results of the calculation of the self-equilibrating stresses indicate a uniform curvature of \(7 \mu \epsilon /\text{in.} \) \((0.28 \mu \epsilon /\text{mm})\). If there were no interior support, this would result in a downward displacement of 2.3 in. \((58 \text{ mm})\) at the center support location. The force to return the location to zero displacement is calculated using the transformed section properties based on the age-adjusted modulus of elasticity for the deck and the grout concrete. This results in a transformed moment of inertia of 22,230 in. \(^4\) \((9.253 \times 10^9 \text{ mm}^4)\) based on the modulus of elasticity of the girder \((29,000 \text{ ksi } [200,000 \text{ MPa}])\) (Table 5). The force to displace the center support location 2.3 in. \((58 \text{ mm})\) is 16.9 kip \((75.2 \text{ kN})\). At the location of the critical connection, 4 ft \((1.2 \text{ m})\) from the center support, the restraint moment \(M_{\text{restraint}}\) is:

\[ M_{\text{restraint}} = \frac{16.9}{2}(67 - 4) = 531 \text{ kip-ft (720 kN-m)} \]

The resulting stress at the top of the deck can be computed with the transformed moment of inertia, the distance from the transformed section centroid to the top of the deck, and the transformed modular ratio, as follows:

\[ \Delta f_{\text{d, restraint}} = \frac{(531)(12)(14.9)(0.086)}{22,230} = 0.365 \text{ ksi (2.52 MPa)} \]
For transient loads, transformed section properties were calculated based on the elastic modulus for the deck and haunch concrete, and the moment of inertia is $25,060 \text{ in.}^4 (10.43 \times 10^9 \text{ mm}^4)$. The top-of-deck stress is calculated as follows:

$$\Delta f_{d,liveload} = 0.398 \text{ ksi} (2.74 \text{ MPa})$$

where

$$\Delta f_{d,liveload} = \text{stress at the top of the deck due to live load moment}$$

Summing the restraint stress with the previously calculated top fiber stress (Fig. 3) results in $-0.118 \text{ ksi} (-0.814 \text{ MPa})$ remaining compression in the deck. This is a considerable reduction from the original precompression of $-0.580 \text{ ksi} (-4.0 \text{ MPa})$.

Finally, the live load moments are determined using the bridge engineering software, which returns the moment due to truck plus lane loads for one lane loaded. This number was then multiplied by the distribution factor, and the multiple presence factor. For this bridge, the live load moment at the critical connection was $488 \text{ kip}\cdot\text{ft} (661 \text{ kN}\cdot\text{m}).$

For transient loads, transformed section properties were calculated based on the elastic modulus for the deck and haunch concrete, and the moment of inertia is $25,060 \text{ in.}^4 (10.43 \times 10^9 \text{ mm}^4)$. The top-of-deck stress is calculated as follows:

$$\Delta f_{d,liveload} = \frac{(488)(12)(12.3)(0.139)}{25,060}$$

$$= 0.398 \text{ ksi} (2.74 \text{ MPa})$$

where

$$\Delta f_{d,liveload} = \text{stress at the top of the deck due to live load moment}$$

Therefore, the total stress after creep and shrinkage with
This method was used for each bridge in the set of examples to investigate the typical range of initial stress required to keep the final stress below the recommended allowable stress.

### Initial stress required under standard conditions for example bridges

Table 6 presents the initial and final compressive stresses across the connection at the end of service for the set of bridges. The initial compression is the average compression in the deck immediately following seating of the tendons.

For simple-span bridges, deck bottom fiber behavior controls the initial compression required because of the

<table>
<thead>
<tr>
<th>Table 4. Results of system of equations for service phase</th>
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</thead>
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<tr>
<td>Change in strain at centroid of deck ( \Delta \varepsilon_d )</td>
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<tr>
<td>Change in strain at centroid of girder ( \Delta \varepsilon_g )</td>
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<tr>
<td>Change in strain at centroid of haunch ( \Delta \varepsilon_h )</td>
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<tr>
<td>Change in strain in posttensioning strands in deck ( \Delta \varepsilon_{ptd} )</td>
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<td>Change in force at centroid of deck ( \Delta N_d )</td>
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<tr>
<td>Change in force at centroid of girder ( \Delta N_g )</td>
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<td>Change in force at centroid of haunch ( \Delta N_h )</td>
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<tr>
<td>Change in force at centroid of posttensioning in deck ( \Delta N_{ptd} )</td>
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<tr>
<td>Change in curvature ( \Delta \chi )</td>
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<td>Change in moment in deck ( \Delta M_d )</td>
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<tr>
<td>Change in moment in girder ( \Delta M_g )</td>
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<tr>
<td>Change in moment in haunch ( \Delta M_h )</td>
</tr>
</tbody>
</table>

Note: 1 in. = 25.4 mm; 1 kip = 4.448 kN.

For simple-span bridges, deck bottom fiber behavior controls the initial compression required because of the

![Figure 3. Stress distribution in composite system after redistribution. Note: 1 ksi = 6.895 MPa.](image-url)
First, the required initial compression was compared with the ratio of girder moment of inertia to deck moment of inertia $I_g/I_d$ (Fig. 4). The required initial compression was also compared to the ratio of girder area to deck area $A_g/A_d$ (Fig. 5). Table 7 shows the parameters. The results shown indicate that as girder stiffness increases relative to the deck, the initial compression required also increases. Greater girder stiffness offers more restraint to the deck panels, resulting in a greater loss of precompression.

In Fig. 6, the initial precompression is compared with the transformed moment of inertia divided by the span length. This is an indication of the flexural stiffness of the girder relative to the span. For the same relative stiffness, the two-span systems require greater initial precompression. The figure also shows that bridges with higher relative stiffness require relatively lower initial precompression. This is primarily due to the lower live load stresses that develop in the deck.

**Initial stress required: accelerated construction effects**

An initial assumption of 55 days of curing time was considered in the parametric study, but such a long curing time might not be possible with an accelerated schedule. Table 8 presents the initial compression needed in the structures from the parametric study if construction begins on day 28 and ends on day 33. For simple-span bridges, to keep the deck in compression the initial compression increased to -346 psi (-2.39 MPa) compared with -277 psi (-1.91 MPa), as previously demonstrated.

A similar increase in initial compression required was shown for the two- and three-span cases (Table 8). For two-span cases the initial compression required ranged

---

**Table 5. Moments of Inertia for Continuous System Calculation**

<table>
<thead>
<tr>
<th></th>
<th>Area $A$, in.²</th>
<th>Moment of Inertia $I$, in.⁴</th>
<th>Centroid from bottom, in.</th>
<th>Modulus of Elasticity $E$, ksi</th>
<th>Creep Coefficient $\phi$</th>
<th>$E/1 + \mu\phi$, ksi</th>
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</thead>
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<tr>
<td>Deck</td>
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<tr>
<td>Prestress</td>
<td>1.836</td>
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<td>41.1</td>
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<tr>
<td>Girder</td>
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<td>17.93</td>
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<td>29,000</td>
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<td>Composite girder for short-term loads</td>
<td>129.2</td>
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<td>33.1</td>
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<tr>
<td>Composite girder for long-term loads</td>
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<td>22,230</td>
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<td>29,000</td>
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</tbody>
</table>

Note: $\mu$ = aging coefficient. 1 in. = 25.4 mm; 1 ksi = 6.895 MPa.
Conclusion

For simple-span bridges, an initial precompression of -200 to -300 psi (-1.4 to -2.1 MPa) is adequate to keep the joints in compression over a 10,000-day service life, for panels that are approximately 55 days old at the time of posttensioning. For continuous bridges it is difficult to provide an average value of initial precompression required to keep the joints in compression. The tensile stresses caused by the time-dependent restraint moments at interior piers and from the negative live load moments vary greatly among bridges.

Table 6. Initial compression and final maximum stress limit in the deck of 3

<table>
<thead>
<tr>
<th>Structure number</th>
<th>NBI structure number</th>
<th>Number of spans</th>
<th>Initial compression, psi</th>
<th>Final stress, psi</th>
<th>Critical stress location</th>
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Note: $\sigma = \text{specified compressive strength of concrete; NBI = National Bridge Inventory. 1 psi = 6.895 kPa.}$

from -754 to -1185 psi (-5.20 to -8.17 MPa). The three-span cases required between -305 and -837 psi (-2.1 and -5.77 MPa) initial compression. Structures 9 and 12 required the greatest precompression, -1179 psi (-8.13 MPa) and -1185 psi (-8.17 MPa), respectively. To achieve this precompression, a tendon with four $\frac{1}{2}$ in. (13 mm) diameter strands would need to be provided every 12 in. (300 mm) across the width of the deck. The creep and shrinkage losses are greater for concrete stressed at early ages; thus, considerably higher initial compression was required.
Figure 4. Initial compression versus $I_g/I_d$. Note: $I_g =$ moment of inertia of effective deck; $I_d =$ moment of inertia of girder. 1 psi = 6.895 kPa.

Figure 5. Initial compression versus $A_g/A_d$. Note: $A_g =$ cross-sectional area of effective deck; $A_d =$ cross-sectional area of girder. 1 psi = 6.895 kPa.
A maximum tensile stress at the top of the panel, including live load and time-dependent stresses, of $3.0\sqrt{f_c}$ psi ($0.25\sqrt{f_c}$ MPa) where $\sqrt{f_c}$ is the smaller of the concrete compressive strength and the grout compressive strength is recommended. However, if the bridge is in a particularly aggressive or benign environment, this limit could be adjusted at the discretion of the engineer.

The recommended stress was achieved in the example three-span bridges with -200 to -750 psi (-1.4 to -5.2 MPa) of initial prestress. Two-span systems are the most critical, with some of the example bridges examined in this research requiring close to -1000 psi (-6.90 MPa) of initial prestress. The analysis is affected by the construction schedule, which should be considered. Panels stressed when they are 28 days old will require over 25% more initial compressive stress on average to meet the final stress criteria compared with panels stressed when they are 55 days old. Storing the panels for a longer period will reduce the creep coefficient and the remaining shrinkage, so the required prestress will be less. The age-adjusted effective modulus method can be used by engineers to study the feasibility of full-depth precast concrete panels for deck replacement and preliminary design, and is adequate for typical structures. Detailed time-dependent finite element analysis should be conducted for final design of more complex systems. Any time dependent analysis method requires a model for the time dependent properties.
of concrete, and there is a great deal of variability in these properties.

Acknowledgments

The research project was done with the cooperation of the Virginia Transportation Department and Virginia Center for Transportation Innovation and Research. Their financial support and assistance contributed significantly to this project. In addition, the Virginia Polytechnic Institute and State University Via Department of Civil and Environmental Engineering provided resources for the implementation of the project. The authors appreciate their support and assistance on this project.

References


Table 8. Initial compression and final maximum stress limit of $3\sqrt{f_c}$ psi in the deck in an accelerated construction schedule

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<tr>
<th>Structure number</th>
<th>NBI structure number</th>
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Note: $f_c$ = specified compressive strength of concrete; NBI = National Bridge Inventory. 1 psi = 6.895 kPa.


Notation

\[ a \] = distance between centroids of girder and haunch
\[ A \] = cross-sectional area
\[ A_d \] = cross-sectional area of effective deck
\[ A_g \] = cross-sectional area of girder
\[ A_h \] = cross-sectional area of haunch
\[ A_{ptd} \] = total area of posttensioning strands in deck
\[ b \] = distance between deck centroid and girder centroid
\[ E \] = modulus of elasticity
\[ E_d \] = modulus of elasticity of deck
\[ E_g \] = modulus of elasticity of girder
\[ E_h \] = modulus of elasticity of haunch
\[ E_{ptd} \] = modulus of elasticity of posttensioning strand
\[ f'_c \] = specified concrete compressive strength
\[ f_d,\text{total} \] = total stress at top of deck
\[ f_{p,\text{avg}} \] = average stress in prestressing steel
\[ f_{p,dead} \] = stress in prestressing steel at dead (nonstressing) end of tendon
\[ f_{p,\text{live}} \] = stress in prestressing steel at live (stressing) end of tendon
\[ f_{pu} \] = specified tensile strength of prestressing steel
\[ I \] = moment of inertia
\[ k \] = wobble coefficient
\[ \ell \] = tendon length
\[ L \] = span length
\[ I_d \] = moment of inertia of effective deck
\[ I_g \] = moment of inertia of girder
\[ I_h \] = moment of inertia of haunch
\[ I_{\text{trans}} \] = transformed moment of inertia
\[ M_{\text{restraint}} \] = restraint moment
\[ N_{do} \] = initial force at centroid of deck
\[ N_{doc} \] = initial force at centroid of deck for composite phase
\[ \Delta f_{d,\text{live}} \] = stress at top of deck due to live load moment
\[ \Delta f_{d,\text{restraining}} \] = stress at top of deck due to restraint moment
\[ \Delta f_{r} \] = change in stress due to relaxation of deck strands over a given time interval
\[ \Delta M_d \] = change in moment in deck
\[ \Delta M_g \] = change in moment in girder
\[ \Delta M_h \] = change in moment in haunch
\[ \Delta N_d \] = change in force at centroid of deck
\[ \Delta N_g \] = change in force at centroid of girder
\[ \Delta N_h \] = change in force at centroid of haunch
\[ \Delta N_{ptd} \] = change in force at centroid of posttensioning in deck
\[ \Delta \chi \] = change in curvature
\[ \Delta \chi_d \] = change in curvature in deck
\[ \Delta \chi_g \] = change in curvature in girder
\[ \Delta \chi_h \] = change in curvature in haunch
\[ \Delta \epsilon_d \] = change in strain at centroid of deck
\[ \Delta \epsilon_g \] = change in strain at centroid of girder
\[ \Delta \epsilon_h \] = change in strain at centroid of haunch
\[ \Delta \epsilon_{ptd} \] = change in strain in posttensioning strands in deck
\[ \epsilon_{shd} \] = shrinkage strain in deck concrete over given time interval
\[ \epsilon_{shh} \] = shrinkage strain in haunch concrete over given time interval
\[ \mu \] = aging coefficient
\[ \mu_d \] = aging coefficient for deck
\[ \mu_h \] = aging coefficient for haunch
\[ \phi_d \] = creep coefficient for deck concrete over a given time interval
\[ \phi_h \] = creep coefficient for haunch concrete over a given time interval
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Abstract

Precast, posttensioned concrete deck panels can be used for rapid replacement of bridge decks; however, the appropriate initial stress to avoid cracking and leaking in bridge deck panels remains a question. This paper uses the age-adjusted effective modulus method to account for creep, shrinkage, and stress redistribution in the composite system. A parametric study was conducted on simple- and continuous-span bridge configurations with steel girders using the maximum tensile stress limit from the companion paper (maximum tensile stress of $3.0 \sqrt{f_c}$ psi [0.25 $\sqrt{f_c}$ MPa]).

For simple-span bridges, an initial precompression of 200 to 300 psi (1.38 to 2.07 MPa) was adequate. In continuous three-span bridges, the recommended stress was typically achieved with 200 to 750 psi (5.17 MPa) of initial prestress. Two-span systems were most critical; some bridges required close to 1000 psi (6.90 MPa) of initial prestress. For continuous-span bridges, the initial compressive stress required increased substantially for accelerated construction schedules.

Keywords

Bridge, creep, deck, panel, posttensioning, shrinkage, steel beam, time dependence.

Review policy

This paper was reviewed in accordance with the Precast/Prestressed Concrete Institute’s peer-review process.

Reader comments

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