Creep and Shrinkage in Composite Cable-Stayed Bridges

S. G. ARZOUMANIDIS, R. G. BURG, AND J. SCHMID

The application of strict controls on the creep and shrinkage exhibited by the roadway deck concrete of composite cable-stayed bridges is of primary importance. Precast deck panels from concrete specifically designed for minimized creep and shrinkage effects, carefully cured and matured are almost exclusively used in these structures. Such concrete shows a reduced long-term modular ratio which is quite different from the modular ratio from the roadway deck concrete of conventional composite girders. The provisions of AASHTO for the design of composite girders. The provisions of ACI 209 can be used for the prediction of the creep and shrinkage effects of concrete. Creep tests are important for consideration of the specific material, project, and site data.

Several cable-stayed bridges with composite girders have been designed and built in North America in recent years. The concrete roadway deck of these bridges is an integral part of the steel support system and carries vertical loads and significant horizontal compressive forces. The latter forces, typical in cable-stayed bridges, result from the inclined cables which support the composite girders.

Due to economic considerations, the concrete deck, which is efficient in carrying compressive forces, is made composite with the steel girders (built-up members or trusses) for live as well as dead loads. The effectiveness of this composite structural system is related to the creep and shrinkage properties of the concrete. Shrinkage is the decrease with time of the moisture content and other physical-chemical changes. Creep on the other hand is the time-dependent increase of the concrete strain due to applied sustained loads. The effect of creep and shrinkage is the slow transfer of stresses from the concrete to the steel resulting in long-term reduced efficiency of the concrete in resisting loads.

The competitiveness of composite girders as opposed to other structural systems of cable-stayed bridges, entails the reduction of the concrete deck weight to the absolute minimum. This is achieved using high strength concrete, which reduces the thickness of the deck. Nevertheless, the high dead load stresses, primarily due to the horizontal forces from the cables throughout the length of the bridge, result in increased creep of the concrete. The effect of creep on the carrying capacity of short and medium length composite girder bridges is considered in the AASHTO specifications. For cast-in-place concrete, AASHTO requires a threefold increase of the modular ratio, defined as the ratio of steel modulus of elasticity to concrete modulus of elasticity. For example, for 6,000 psi concrete, the modular ratio for loads of short duration (live, earthquake, wind loads etc.) is 6 while for loads of long duration (dead loads etc.) is 18. This increase of the modular ratio implies that the modulus of elasticity of concrete (or the concrete stiffness) for long-term loads is three times smaller than the modulus of elasticity (or concrete stiffness) for short-term loads.

Although shrinkage is not specifically mentioned as contributing in the increase of the modular ratio, the AASHTO procedure has apparently worked satisfactorily for conventional composite girder bridges. For cable-stayed bridges, however, this approach to resolving the creep and shrinkage problem results in an uneconomical solution and is clearly inadequate.

MODULAR RATIO

The size of concrete and steel sections in composite members depends on the relative stiffness of the two materials. Consideration of the creep and shrinkage effects is essential in the design of composite members. Thus, composite members are sized considering the short and long-term stiffness of concrete using the transformed area method and the modular ratio of concrete for short and long-term loads.

For composite cable-stayed bridges, the effect of creep and shrinkage is controlled through the application of strict requirements on the long-term modular ratio. In actual designs of composite bridges this ratio has been specified as low as 11 (1,2,3).

The forces in the composite top chord members of a recent design of a two lane cable-stayed truss bridge are used to demonstrate the benefit of using a low long-term modular ratio. Dead plus live loads due to HS-20 loadings are considered. Assuming uniform concrete properties throughout the deck, the modular ratio for live loads is taken as 6, while for dead loads it is varied from 11 up to 18.

Table 1 shows the stresses in the steel and concrete corresponding to different long-term modular ratio NJ values. It also shows the change of stresses in the steel and the concrete for modular ratio values higher than 11. It is seen that for the increase of the modular ratio from 11 to 12 the steel stresses increase by as much as 5.3% and the concrete stresses decrease by as much as 2.6%. Similarly, for the modular ratio increase

1Steinman Boynton Gronquist & Birdsall, New York, NY 10038
2Construction Technology Laboratories, Inc. Skokie, IL 60077-1030
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decrease by as much as 15.4%. This change of stresses for cast-in-place concrete. The steel stresses for the long-term modular ratio 18 specified by AASHTO imply a considerable reduction in the long-term participation of the concrete deck in carrying loads and, consequently, diminished effectiveness. For this reason, the use of concrete with long-term modular ratio of 18 would be uneconomical.

### ERECTION CONSIDERATIONS

Although a few composite cable-stayed bridges adopted cast-in-place concrete roadway decks, most bridges have used precast deck panels. The precast deck panels are fabricated and cured under carefully controlled conditions and allowed to mature for an extended period of time. This procedure improves the creep properties of the concrete and, at the same time, removes a considerable percentage of the concrete shrinkage prior to the application of loads to the panels on the bridge.

Most often, the erection of the girders is performed by repeating a cycle of assembling steel components, cables and deck panels. The connection of the concrete with the steel is achieved through shear connectors.

### Figure 1 Precast panel connection detail

From 11 to 13, the corresponding maximum change of the steel and concrete stresses is 10.3% and 5.0% respectively.

It is further interesting to note the steel and concrete stresses for the long-term modular ratio 18 specified by AASHTO for cast-in-place concrete. The steel stresses increase by as much as 31.5% and the concrete stresses decrease by as much as 15.4%. This change of stresses implies a considerable reduction in the long-term participation of the concrete deck in carrying loads and,

### TABLE 1: STEEL AND CONCRETE STRESSES IN TOP CHORD

<table>
<thead>
<tr>
<th>Top Chord Member</th>
<th>STEEL</th>
<th></th>
<th>CONCRETE</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Stresses in Ksi</td>
<td>Percentage Increase with respect to Nl=11</td>
<td>Stresses in Ksi</td>
<td>Percentage Decrease with respect to Nl=11</td>
</tr>
<tr>
<td>No</td>
<td>Nl=11</td>
<td>Nl=12</td>
<td>Nl=13</td>
<td>Nl=18</td>
</tr>
<tr>
<td>U02'-U02</td>
<td>-10.3</td>
<td>-10.3</td>
<td>-10.3</td>
<td>-10.3</td>
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<td>U02 -U06</td>
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<td>-10.9</td>
<td>-11.0</td>
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<tr>
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<td>-19.0</td>
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<td>-23.3</td>
<td>-23.7</td>
<td>-25.1</td>
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<td>-24.3</td>
<td>-24.7</td>
<td>-26.7</td>
</tr>
<tr>
<td>U26 -U30</td>
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<td>-22.1</td>
<td>-22.6</td>
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</tr>
<tr>
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<td>-25.1</td>
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<td>-19.7</td>
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<tr>
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</tr>
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<td>-16.6</td>
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<td>-20.7</td>
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<td>U90 -U94</td>
<td>-15.2</td>
<td>-16.0</td>
<td>-16.7</td>
<td>-19.9</td>
</tr>
</tbody>
</table>

Figure 1: Precast panel connection detail

from 11 to 13, the corresponding maximum change of the steel and concrete stresses is 10.3% and 5.0% respectively.

Additional cast-in-place concrete is used to fill small openings in the deck panels to achieve the composite action between concrete and steel. The connection of the concrete with the steel is achieved through shear connectors. Figure 1 shows a connection detail of this type using shear studs.
Figure 2  Two arrangements of precast deck panels on bridge roadways

Figure 3  Karnali Bridge elevation
The erection procedures and in particular the timing of placing the cast-in-place concrete affect significantly the distribution of the dead load forces between concrete and steel. Consider the Karnali River Bridge which is a one tower asymmetric composite truss cable-stayed bridge, figure 3, currently under construction in Nepal. The distribution of the dead load forces in the steel and concrete components of the top chord prior to achieving composite action and at the completion of the erection are shown in figure 4. It is seen that the dead load forces of the concrete and steel components of the top chord are not the same throughout the length of the bridge. Sections of the chord near the tower carry higher forces than sections further away. By adjusting the erection procedures, it is possible to modify the level of dead load forces distributed between concrete and steel both prior to as well as after achieving composite action.

It is clear that the deck of composite cable-stayed bridges with precast panels essentially consists of precast and cast-in-place concrete sections. The cast-in-place sections do not undergo the rigorous curing and extended maturing of the precast concrete panels and they appear to be in relative disadvantage regarding their creep and shrinkage properties. To minimize or even eliminate the effect of shrinkage, shrinkage compensating cement may be implemented. Three factors appear to further limit the consequences from this apparent disadvantage of the cast-in-place concrete:

1. The relatively small percentage of the cast-in-place concrete which is typically around 18 percent of the total concrete volume.
2. The dead load force distribution in the roadway deck along the length of the bridge which shows a significant reduction of the forces away from the tower as shown in figure 4.
3. The history of dead load application during erection as discussed below.

Assuming a ten day erection cycle for a typical bridge segment between consecutive stays, figure 5 shows the loading history during erection of three sections of the concrete deck in the main span. It can be seen that the sections of the deck with the highest stresses are loaded at the slowest rate and receive their full load after a considerable period of time.

Figure 4  Dead load force distribution in the concrete and steel of the top chord in main span.

Figure 5  Loading history of three deck sections during erection.
CREEP TESTING PROGRAM

Although the long-term modular ratio of a given concrete mix can be estimated using calculation methods based on fresh and hardened concrete properties (4), a better value can be developed based on actual creep tests conducted on several candidate concrete mixes. Creep testing is conducted in accordance with ASTM C512-87 entitled "Standard Test Method for Creep of Concrete in Compression" (5).

Creep tests are conducted by subjecting standard 6x12 inch concrete specimens to a sustained compressive load and at specified time intervals measuring changes of the concrete strain. To account for strain resulting from drying shrinkage, drying induced strains in companion unloaded specimens are measured and the resulting strains are subtracted from load induced strains. Creep tests and the corresponding shrinkage tests, are conducted in a controlled temperature and humidity room maintained at 73.4 ± 3.0°F and 50 ± 4% relative humidity.

The imposed load for creep testing may be as high as 40 percent of the compressive strength of the concrete measured at the age of loading. If the stress level in the structure is known, it is desirable to conduct the test at that stress level. However, if the stress level in the structure varies or it is not known, a stress of between 30 and 40 percent of the concrete strength may be safely used. Several researchers (6,7) have reported that for stress levels less than about 40 to 50 percent of concrete strength, creep strains are approximately proportional to the sustained stress and obey the principal of superposition of strain history.

Because age of loading has a profound effect on the creep properties of any concrete, creep tests are conducted at several different ages. Typical loading ages include 2, 7, 28, 90 days and 1 year. Later age loading is desirable especially if the construction schedule is such that deck panels will not be loaded until long after they are cast. Because it is desirable to have at least 3 months of creep data on which to base long-term modular ratio predictions, it is apparent that creep tests must be started early in the construction phase of a project. If this is not possible, the effect of loading age on long-term modular ratio can be estimated from a series of creep tests performed at loading ages between 2 and 28 days.

LONG TERM MODULAR RATIO BASED ON CREEP TESTS

The short-term modular ratio is denoted as

\[ N_S = \frac{E_S}{E_C} \]  (1)

where

\[ N_S = \text{short-term modular ratio} \]
\[ E_S = \text{modulus of elasticity of steel} \]
\[ E_C = \text{modulus of elasticity of concrete} \]

and the long-term modular ratio as

\[ N_l = \frac{E_S}{E_{eff}} \]  (2)

where

\[ N_l = \text{long-term modular ratio} \]
\[ E_{eff} = \text{long-term effective modulus of elasticity of concrete} \]

The long-term effective modulus of elasticity of concrete includes the effect of initial elastic deflection and long-term deflection due to creep and shrinkage. If the importance of shrinkage is minimized through measures as considered above, the long-term effective modulus of elasticity of concrete can be expressed in terms of the modulus of elasticity at time \( t \) and the ultimate creep coefficient as shown in the following expression (4)

\[ E_{eff}(t) = E_C(t) \frac{1 + \mu_u}{1 + \mu_u^a} \]  (3)

where

\[ E_C(t) = \text{modulus of elasticity of concrete at time } t \]
\[ \mu_u = \text{ultimate creep coefficient defined as the ratio of ultimate creep strain to initial strain} \]

Using the above relationships, the long-term modular ratio can be expressed in terms of the short-term modular ratio and the ultimate creep coefficient as follows

\[ N_l = N_S(1 + \mu_u K_a Y_{H}) \]  (4)

where

\[ K_a = \text{loading age correction factor, 1.0 at 7 days} \]
\[ Y_{H} = \text{humidity correction factor} \]
\[ Y_{H} = \text{element thickness correction factor} \]

Creep analysis based on the above approach is appropriate only when the gradual changes of stress due to creep are relatively small and do not result in fundamental change in the distribution of stresses and the response of a structure.

The ultimate creep coefficient needed for the prediction of the long-term modular ratio can be established from creep tests. As shown in equation (4), several adjustments must be made to the ultimate creep coefficient to account for age of loading of the creep test specimens as compared to the actual members in the structure, effects of member size as compared to the standard test specimen size, and ambient humidity conditions at the site as compared to the humidity conditions at the laboratory. The following paragraphs describe how creep test data and knowledge of site and specific structure conditions can be used to estimate long-term modular ratio.

According to reference 4, the creep coefficient at time \( t \) for loading age of 7 days for moist cured concrete and for 1 to 3 days steam cured concrete can be expressed in the following form

\[ \varepsilon_t = \frac{f + t^{0.6}}{f + t^{0.6}} \]  (5)
where

\[ \varepsilon_t = \text{creep coefficient at time } t \]
\[ \varepsilon_u = \text{ultimate creep coefficient} \]
\[ f = \text{half time in days} \]

Applying regression analysis to test data for concrete specimens loaded after 7 days of moist curing, values can be determined for the half time \( f \) and the ultimate creep coefficient \( \varepsilon_u \). Reasonable values for the ultimate creep coefficient can be obtained after 90 days of creep test data become available.

Figure 6 shows creep test data the authors developed for the three potential concrete mixes of Table 2 for use for the Karnali River Bridge. Each mix was specifically designed to minimize creep and shrinkage. In the effort to minimize creep, the concrete compressive strength exceeded the required by strength considerations. The final selection of mix No. 2 was based on its creep as well as workability characteristics.

All three concrete mixes were loaded after 7 days of moist curing. Test data are expressed in terms of specific creep values which are converted to values of the creep coefficient \( \varepsilon_t \) by multiplying with the modulus of elasticity. Using these data, ultimate creep coefficients of 1.34, 1.42 and 1.57 were calculated for concrete mixes denoted 1, 2 and 3 respectively. Reference 4, indicates that the ultimate creep typically ranges from 1.3 to 4.15.

The same three concrete mixes were also subjected to creep tests after 14 and 28 days of moist curing to establish the effect of loading age on the ultimate creep coefficient. According to reference 4, the correction factor for loading age of concrete loaded at ages subsequent to 7 days of moist curing has the following form

\[ K_a = A t^b \]

![Figure 6 Specific creep of three concrete mixes loaded after 7 days moist cure](image)

**TABLE 2: MIX PROPORTIONS AND PROPERTIES OF FRESH CONCRETE**

<table>
<thead>
<tr>
<th>Material</th>
<th>Quantity in a Mix per Cubic Yard</th>
<th>Parameter</th>
<th>Loading Age</th>
<th>Correction Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>No. 1</td>
<td>No. 2</td>
<td>No. 3</td>
<td>Slump, in</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Unit Weight, lb/ft³</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Air Content, %</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Water to Cement Ratio %</td>
</tr>
</tbody>
</table>

Water to cement ratio includes water in admixtures
Thus, it is apparent that longer preloading periods can significantly reduce the ultimate creep coefficient resulting in a proportional reduction of the long-term modular ratio. If the entire load is not applied at a discrete point of time but rather over an extended period as shown in figure 5, a suitable correction factor must be selected to account for this effect. Furthermore, since the largest change in the loading age correction factor occurs during the early ages, much benefit can be gained by small delays in the early application of loads.

The ultimate creep coefficient must be further adjusted for the specific site conditions of average relative humidity and element thickness. Both of these adjustments are straightforward and well documented in reference 4.

Table 3 presents the long-term site specific modular ratios developed for the three concrete mixes investigated by the authors. As anticipated, the mix with the highest compressive strength and modulus of elasticity developed the lowest short and long-term modular ratios.

CONCLUSION

This paper identifies the requirements for the creep and shrinkage properties for the concrete deck of composite cable-stayed bridges and presents a rational evaluation of the modular ratio of the concrete mixes used in a project. This approach further provides the means for consideration of the specific material, project and site data in the evaluation of the creep and shrinkage effects. It was found that:

1. Long-term modular ratios based on code suggested values are overly conservative for certain concretes and values should be established using creep tests with material, project and site specific data.

REFERENCES

### TABLE 3: LONG TERM MODULAR RATIO FOR THREE CONCRETE MIXES

<table>
<thead>
<tr>
<th>Mix No</th>
<th>Compressive Strength 28 days, psi</th>
<th>Modulus of Elasticity ksi</th>
<th>Short-term Modular Ratio</th>
<th>Creep Coefficient 7 days</th>
<th>Loading Age Factor 1 Year</th>
<th>Humidity Correction Factor</th>
<th>Thickness Correction Factor</th>
<th>Long-term Modular Ratio</th>
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<tr>
<td>1</td>
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<td>0.623</td>
<td>0.821</td>
<td>0.957</td>
<td>11.8</td>
</tr>
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</table>

4. ACI Committee 209 "Prediction of Creep, Shrinkage, and Temperature Effects in Concrete Structures" ACI Manual of Concrete Practice, Part 1