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SHORTENING OF WAITING PERIOD FOR CLOSURE POUR IN BRIDGE CONSTRUCTION

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ABSTRACT: California practice requires relatively long waiting period before closure pour can be made in bridge widening and staged construction, leading to unnecessary delay in project completion. The delayed casting of the closure slab is intended to avoid potential damage in the connected decks as a result of the differential displacement occurring between the new and previously constructed decks. The waiting period, which can take up to 60 days in California, does not take into account of the displacement capacity of the slab and the time-dependent deformation of the bridge. This paper proposes a more rational approach to the estimation of waiting period by limiting the displacement demand across the connected bridges to the displacement capacity of the closure slab. Numerical examples are provided in the paper to illustrate the procedure and preliminary results indicate that the current waiting period may be quite conservative, especially in staged construction.

KEYWORDS: Bridge widening; Closure slab; Staged construction;

1 INTRODUCTION

Mitigation of traffic congestion on highways often requires construction of new and/or reconfiguration of old bridges, and two types of construction are commonly used in California to increase the number of traffic lanes. In the first type, called "bridge-widening", one or two new bridges are constructed adjacent to the existing bridge, followed by casting of a closure slab to connect the new bridge to the existing bridge. In the second type, called "staged-construction", two new bridges are constructed in sequence or stages. A new bridge, called the stage I bridge, is built next to the existing bridge to reroute the traffic during construction. After the completion of the stage I bridge, stage II construction begins the demolition of the existing bridge, followed by construction of a second new bridge at the existing bridge location. The two new bridges are eventually connected by a closure slab to form a smooth transition for bridge traffic. Figure 1 highlights the two types of construction in California and the provision of closure slab in the two bridges.

Figure 1. Lane addition by widening or stage construction

Due to the age difference, the new bridge is expected to exhibit a larger time-dependent deflection compare to the old bridge, potentially damaging the closure slab if the connection is made too early. The relative deformation of the two bridges and its implication on closure slab is illustrated in Figure 2. A delayed cast, called waiting period, is imposed before the two bridges are connected to minimize the stress build-up in the closure slab.

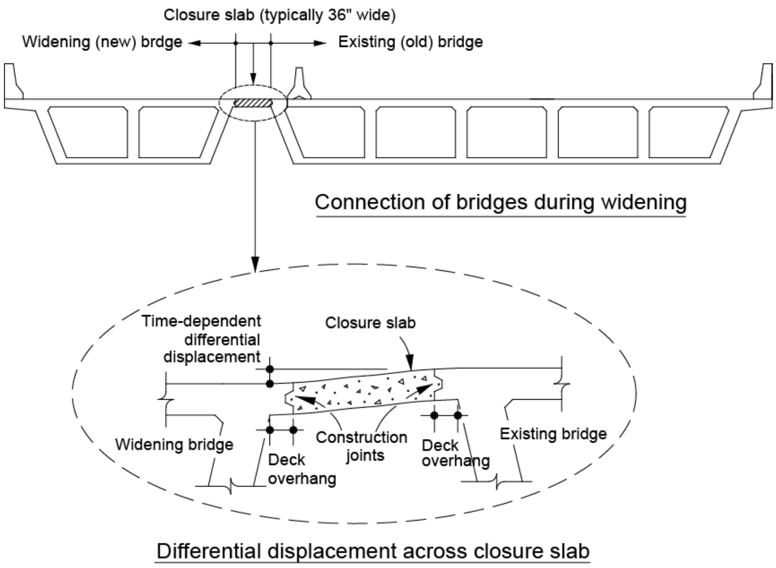


Figure 2. Construction of closure slab in bridge widening

Two alternatives are currently specified for the waiting time in California. The two alternatives, as specified by Caltrans [1], are as follows:

- Alternative 1: Falsework shall be released as soon as permitted by the specifications. Closure pour shall not be placed sooner than 60 days after the falsework has been released.
- Alternative 2: Falsework shall not be released less than 28 days after the last concrete deck has been placed. Closure pour shall not be placed sooner than 14 days after the falsework has been released.

The two alternatives result in quite different waiting periods in practice. Coupled with the requirement [2] that falsework cannot be released less than 10 days after the last concrete pour, the actual waiting period for Alternative 1 is 70 days. Alternative 2, on the other hand, offers a shorter waiting period of 42 days but the falsework has to remain in place for a longer duration of 28 days.

In either case, the waiting period has implications not only on the cost of construction but also on work safety. Temporary traffic barriers would have to be placed next to the closure during construction, thereby reducing the width of the traffic lanes and constricting the traffic flow. A shorter waiting period would allow an earlier removal of the traffic barrier providing a safer traffic condition. Also in the case of Alternative 2, traffic flow under the bridge also becomes impeded as falsework has to remain in place for a longer duration. Thus proper considerations of the waiting period for closure pour are important for widening of bridges.

2 BRIDGE DEFLECTION UPON FALSEWORK RELEASE

Falsework release in cast-in-place construction requires careful and coordinated removal of form support, and the superstructure must be self-supporting upon the release, which can be achieved after concrete has gained sufficient strength. The transfer of self-weight to the superstructure can be idealized as an instantaneous event with the gravity loading represented by a step function

$$w(t) = w_o U(t - t_1) \tag{1}$$

where w_o is the self-weight of the superstructure, t is the time from the day of casting, and $U(t - t_1)$ is a unit step function, taking on a value of 0 for $t < t_1$ and unity for $t \geq t_1$.

Figure 3(a) shows a simply-supported bridge where the immediate mid-span deflection after falsework release is denoted by $\delta_{instant}$. As time progresses, the bridge deforms further with the mid-span deflection denoted by $\delta_{midspan}(t)$. The ratio of mid-span deflection to instantaneous deflection, $\kappa_s(t)$, termed as the deflection factor, can be defined as

$$\kappa_s(t) \equiv \frac{\delta_{midspan}(t)}{\delta_{instant}} \tag{2}$$

The deflection factor depends to a large extent on the properties of the materials.

Figure 3(b) shows the response of concrete under a unit compressive stress $\sigma = 1$, which can be characterized by the creep compliance function $J(t, t_1)$. At the time of load application t_1, the instantaneous elastic strain is given by $J(t_1, t_1)$, while at time $t > t_1$, the time-dependent strain is given by $J(t, t_1)$. A normalized creep compliance function, $\kappa_m(t)$, can be defined as the ratio of time-dependent strain to the instantaneous strain i.e.

$$\kappa_m(t) \equiv \frac{J(t, t_1)}{J(t_1, t_1)} \tag{3}$$

By invoking the "correspondence principle", as commonly assumed in linear viscoelasticity [3,4], the deflection factor $\kappa_s(t)$ may be taken as the normalized creep compliance function $\kappa_m(t)$. Thus

$$\delta_{midspan}(t) = \delta_{instant} \frac{J(t, t_1)}{J(t_1, t_1)} \tag{4}$$

It is recognized that the correspondence between material level deformation and structural level deformation can only be considered as approximate, the assumption is nonetheless deemed acceptable for purposes of expediency in waiting period estimation. The mid-span deflection in Eq. (76) assumes the release of falsework is instantaneous, but in reality, the actual release may take several hours or days to complete in the field. The duration of falsework release can be taken into account by modifying Eq. (4). To that end, let t_1 be the time at the start of falsework release, t_2 be the time at the end of release, and $\delta_{measured}$ be the deflection measured at the end of the falsework release, the mid-span deflection suitable for estimating the waiting period of closure pour can be written as

$$\delta_{midspan}(t) = \delta_{measured} \frac{J(t, t_1)}{J(t_2, t_1)} \tag{5}$$

where $J(t_2, t_1)$ is now the creep compliance calculated using $t = t_2$.

Many creep compliance functions have been proposed in the literature but the short form of the B3 model [5] will be considered in this paper. The suitability of the model is assessed by comparing the prediction of the model with the measured bridge deflections reported in [6]. Figure 4(a) shows the mid-span deflection of the Santa Rosa Creek Bridge and the prediction by the B3 model [5]. The Santa Creek Bridge is a simply-supported post-tensioned box-girder bridge with a span of 170 ft (51.8 m) and is located on Highway 101 in Santa Rosa, California, The field measured and theoretical curves have been normalized by the 'instantaneous' deflection which was recorded at the end of the falsework release which took two days to complete. The short form of the B3 model [5], presented later in the appendix, assumes the following values:

start of falsework release at $t_1 = 12$ days, end of release $t_2 = 14$ days, duration of concrete curing $t_o = 7$ days, mean 28-day compressive strength $\overline{f_c} = 6200$ psi (42.7 MPa), ambient relative humidity $h = 0.7$, volume-to-surface area ratio $v/s = 4.9$ in (124 mm) and effective cross-section thickness of $D = 9.8$ in (249 mm). It can be seen from Figure 4(a) that the short-form of the B3 model compared well the normalized mid-span deflection of the Santa Rosa Creek Bridge for up to about 180 days, after which the measured deflection became affected by the traffic loading upon closure.

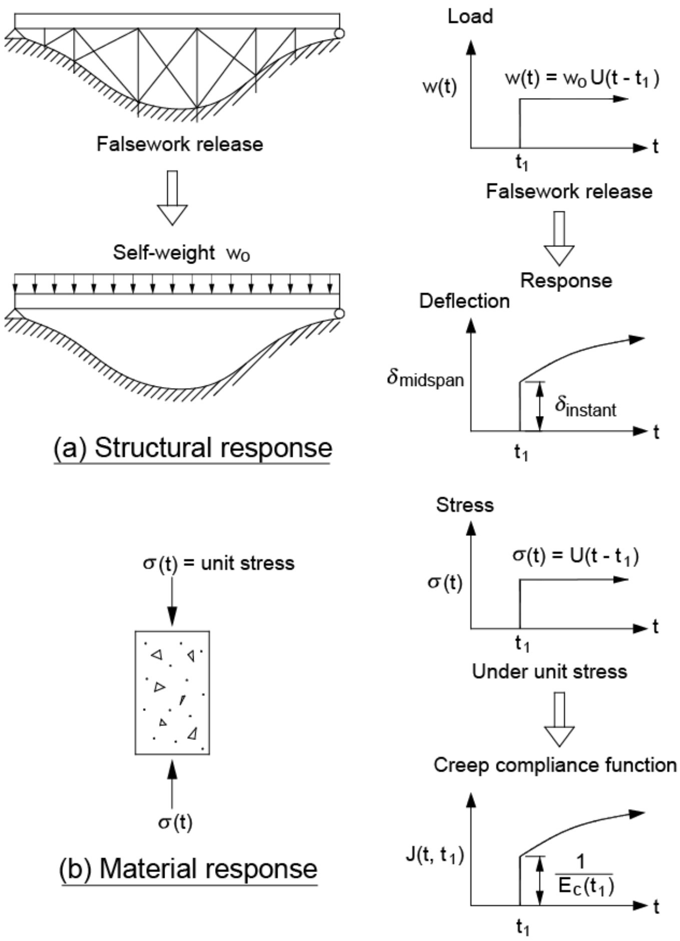
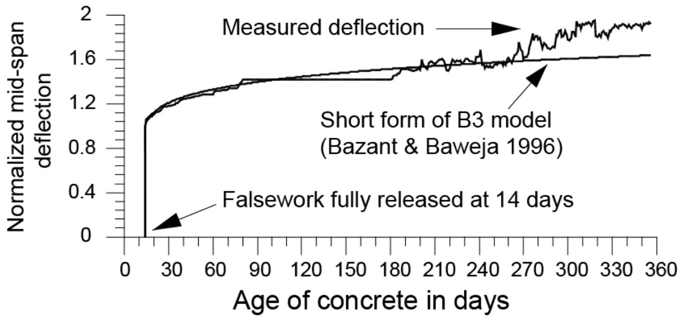
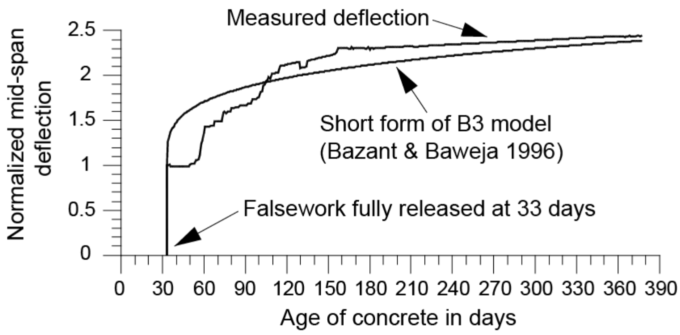


Figure 3. Correspondence between structural and material deformations



(a) Santa Rosa Creek Bridge



(b) San Joaquin River Bridge

Figure 4. Comparison of measured deflection with creep compliance function

Figure 4(b) shows a comparison of the mid-span deflection of the San Joaquin River Bridge with the short form of the B3 model [5]. The San Joaquin River Bridge is a 5-span conventionally reinforced concrete box-girder bridge, located on Interstate Freeway I5 near Lathrop, California. The falsework release in span 5, where the deflection in Figure 4(b) is measured, took less than one day to complete, which means that the prediction of time-dependent mid-span deflection can be made using Eq. (4). The following values are assumed in the B3 model [5]: time of falsework release $t_1 = 33$ days, duration of concrete curing $t_o = 7$ days, mean 28-day compressive strength $\overline{f_c} = 6200$ psi (42.7 MPa), ambient relative humidity $h = 0.6$ (a smaller value to reflect the slightly drier condition compared to the Santa Rosa Creek Bridge), volume-to-surface area ratio $v/s = 3.6$ in (91 mm) for the cross-section and an effective cross-section thickness of $D = 7.2\,\text{in}\,(183\,\text{mm})$. The San Joaquin River Bridge showed a rather constant deflection immediately after falsework release, followed by a rapid increase between 56 and 102 days. This measured deflection is in contrast to the analytical model which tends to give a smooth increase with time. It can be seen from Figure 4(b) that the B3 model [5] overestimates the mid-span deflection of the San Joaquin River Bridge between the time of falsework release and about 110 days, after which the measured deflection catches up with the model with reasonable prediction for $t > 110$ days. It is recognized that actual deflection is often affected by unscheduled construction maneuvers, including the placing and removal of heavy equipment or construction materials on the adjacent spans, and these activities may have affected the measured deflection of the San Joaquin River Bridge.

3 PROCEDURE FOR DETERMINING CLOSURE POUR WAITING PERIOD

A methodology for determining the closure pour waiting period is proposed here on the basis that the differential displacement between the existing and new bridges must be less than or equal to the design displacement capacity of the closure slab. Figure 5 shows the displacement capacity of 200 mm $(8$ in$)$ thick closure slabs tested under monotonic differential displacement [7]. The test slab varied in length from 450 mm $(18$ in$)$ to 900 mm $(36$ in$)$, covering most of the closure slab dimensions in California. In calculating the waiting period, it would be prudent to use a conservative design displacement capacity δ_c, which is set in this paper at 60% of the measured ultimate displacement capacity δ_u. The determination of design displacement capacity is denoted by the lower line in Figure 5.

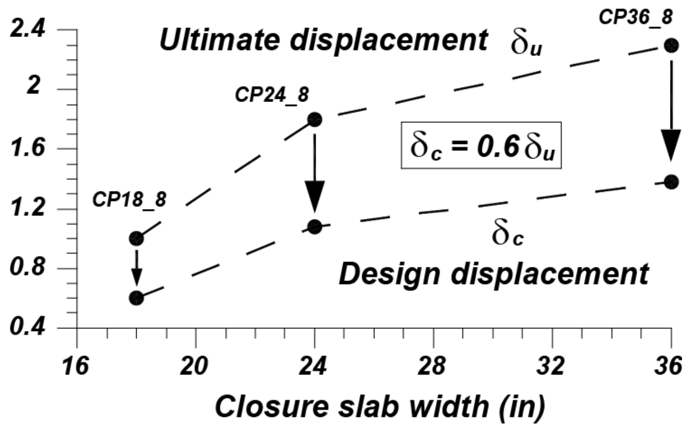


Figure 5. Displacement capacity of California closure slabs

For determining the closure pour waiting period, the following steps are proposed:

(i) Determine the deflection, $\delta_{instant}$ for use in Eq. (4), or $\delta_{measured}$ in Eq. (5). This deflection is best provided by field measurement of the bridge deflection upon falsework release, even though an estimation of the instantaneous deflection is theoretically possible via elastic beam theory. For a simply-supported span, the instantaneous deflection corresponds to the mid-span displacement whereas for the continuous span, the instantaneous deflection should correspond to the largest downward displacement in any of the spans.

(ii) The task at hand is to determine, for a given closure time, whether a closure slab of a particular length and reinforcement details has adequate design displacement capacity to cope with the expected differential displacement. To that end, let t_c denotes the time at closure, measured relative to the last concrete pour in the new bridge in the case of widening or in the stage II bridge in the case of staged construction. The waiting period can thus be written as $\Delta t_w = t_c - t_1$, where t_1 is the time of falsework release. Figure 6 shows the definition for these time parameters. The long-term mid-span deflection, $\delta_{midspan}(t_\infty)$, assumed to occur at 10000 days (about 30 years), can be calculated using Eq. (4) or Eq. (5) with $t \rightarrow \infty$. In the case of bridge widening, the displacement demand on the closure slab can be written as

$$\delta_d = \delta_{midspan}(t_\infty) - \delta_{midspan}(t_c) \quad \text{for bridge widening} \tag{6}$$

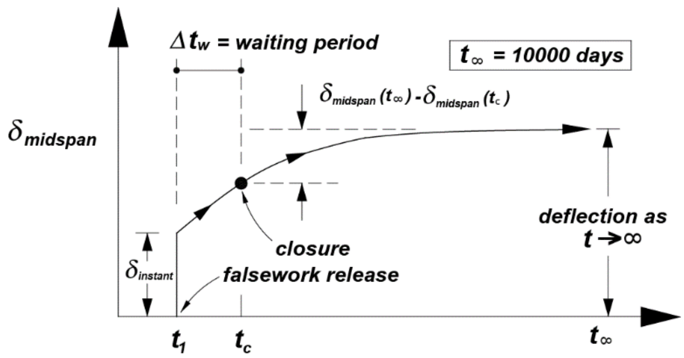


Figure 6. Definition of time parameters - t_c and t_∞

For staged construction, the displacement demand can be similarly determined. However, the mid-span deflection should be separately computed for both stage I and stage II bridges, taking into account of the age difference of the two bridges at closure. More specifically, mid-span deflections for stage I and II bridges at closure can be calculated as $\delta^{I}_{midspan}(t_c + \Delta t_{age})$ and $\delta^{II}_{midspan}(t_c)$, where Δt_{age} represents the age difference between the two bridges. Likewise the mid-span deflection at $t \rightarrow \infty$ can be calculated as $\delta^{I}_{midspan}(t_\infty)$ and $\delta^{II}_{midspan}(t_\infty)$ for stage I and stage II bridges. Thus for staged construction, the displacement demand at the time of closure can be written as

$$\left(\delta^{II}_{midspan}(t_\infty) - \delta^{II}_{midspan}(t_c)\right) - \left(\delta^{I}_{midspan}(t_\infty) - \delta^{I}_{midspan}(t_c + \Delta t_{age})\right) \quad \text{for staged construction} \quad (7)$$

(iii) By comparing the displacement demand, δ_d, on the closure slab from Eq. (6) or (7) with the design displacement capacity, δ_c, of the closure slab, a decision can be made on whether a selected or proposed closure time t_c is acceptable. Thus,

- if $\delta_d \le \delta_c$, the proposed closure pour time t_c, or equivalently waiting period Δt_w, is acceptable,
- on the other hand, if $\delta_d > \delta_c$, the proposed waiting period is unacceptable, and the process needs to be repeated with a longer period and iterated until the displacement demand is smaller or equal to the displacement capacity of the slab.

3.1 Illustrative examples

Two examples will be used to demonstrate the proposed procedure for closure pour waiting period. The first example assumes a staged construction in a simply-supported bridge, while the second example assumes widening of an existing multiple-span continuous bridge.

3.1.1 Example 1 - Simply-supported Bridge

Two new simply-supported bridges having the same span of 51.8 m (170 ft) are constructed in stages. Falsework release is initiated at 10 days for both stage I and II bridges and takes three days to complete for both bridges. The falsework release, and subsequent schedule for closure pour, essentially follows Alternative 1 of current Caltrans practice [1]. The time parameters are therefore $t_1 = 10$ days and $t_2 = 13$ days for both bridges. It is assumed that mid-span deflections measured at $t_2 = 13$ days are $\delta^{I}_{measured} = 1.30$ in (33 mm) for stage I bridge, and $\delta^{II}_{measured} = 1.25$ in (32 mm) for stage II bridge. It is worth noting that these assumed deflections are reasonable for prestressed concrete bridges of that span in California. It is also assumed that the age of the concrete for stage I bridge is 360 days older than that of stage II bridge i.e. $\Delta t_{age} = 360$ days. Duration of curing is taken as $t_o = 7$ days, and the ambient relative humidity is assumed to be $h = 0.7$, the same for both bridges. The same mean 28-day compressive strength of $\overline{f_c} = 6200$ psi (42.7 MPa) and the same effective cross-section thickness of $D = 10$ in (254 mm) are used for both bridges. The closure is made by a slab of 24 in (610 mm) length and 8 in (200 mm) thickness. The design displacement capacity for the closure slab, estimated from Figure 5, is $\delta_c = 1.1$ in (28 mm). It is proposed that the waiting period for closure pour be shortened to $1/2$ of that in the current specification, i.e. $\Delta t_w = 30$ days, compared to the 60-day waiting period specified in Alternative 1. The objective here is to determine if the proposed waiting time is acceptable with respect to the design displacement capacity of the closure slab. Using the procedure outlined, we have the following values for the B3 model [5]

- The measured mid-span deflections at the end of falsework release t_2 are

$\delta^{I}_{measured} = 1.30$ in (33 mm) for stage I bridge

and

$\delta^{II}_{measured} = 1.25$ in (32 mm) for stage II bridge

- The time parameters are

$t_c = t_1 + \Delta t_w = 10 + 30 = 40$ days for stage II bridge

$t_c + \Delta t_{age} = 40 + 360 = 400$ days for stage I bridge

$t_\infty = 10000$ days for both bridges

Using US customary units for the B3 model [5] (see the appendix for equations).

$$q_1 = \frac{0.6 \times 10^6}{57000\sqrt{\bar{f}_c}} = \frac{0.6 \times 10^6}{57000\sqrt{6200}} = 0.134$$

$$q_o = \frac{200}{\sqrt{\bar{f}_c}} = \frac{200}{\sqrt{6200}} = 2.54$$

$$q_5 = \frac{6000}{\bar{f}_c} = \frac{6000}{6200} = 0.968$$

$$H(t_1) = 1 - (1-h)\tanh\sqrt{\frac{t_1 - t_o}{32D^2}}$$

$$= 1 - (1-0.7)\tanh\sqrt{\frac{10-7}{32 \times 10^2}}$$

$$= 0.991$$

$$H(t_2) = 1 - (1-h)\tanh\sqrt{\frac{t_2 - t_o}{32D^2}}$$

$$= 1 - (1 - 0.7)\tanh\sqrt{\frac{13 - 7}{32 \times 10^2}}$$

$$= 0.987$$

$$H(t_\infty) = 1 - (1 - h)\tanh\sqrt{\frac{t_\infty - t_o}{32D^2}}$$

$$= 1 - (1 - 0.7)\tanh\sqrt{\frac{10000 - 7}{32 \times 10^2}}$$

$$= 0.717$$

The values of q_1, q_o, q_5, $H(t_1)$, $H(t_2)$ and $H(t_\infty)$ are applicable to both stage I and II bridges. We also need the following values for the two bridges:

$$H\left(t_c + \Delta t_{age}\right) = 1 - (1 - h)\tanh\sqrt{\frac{t_c + \Delta t_{age} - t_o}{32D^2}}$$

$$= 1 - (1 - 0.7)\tanh\sqrt{\frac{400 - 7}{32 \times 10^2}}$$

$$= 0.899 \qquad \text{for stage I bridge}$$

and

$$H(t_c) = 1 - (1 - h)\tanh\sqrt{\frac{t_c - t_o}{32D^2}}$$

$$= 1 - (1 - 0.7)\tanh\sqrt{\frac{40 - 7}{32 \times 10^2}}$$

$$= 0.970 \qquad \text{for stage II bridge}$$

The creep compliance values are

$$J(t_c + \Delta t_{age}, t_1) = \left(q_1 + q_o \ln\left\{1 + \Psi[t_1^{-m} + \alpha]\left(t_c + \Delta t_{age} - t_1\right)^n\right\} + q_5\left[e^{-3H\left(t_c + \Delta t_{age}\right)} - e^{-3H(t_1)}\right]^{0.5}\right) \times 10^{-6}$$

$$= (0.134 + 2.54\ln\{1 + 0.3[10^{-0.5} + 0.001](400 - 10)^{0.1}\} + 0.968[e^{-3\times 0.899} - e^{-3\times 0.991}]^{0.5}) \times 10^{-6}$$

$$= \quad 0.662 \times 10^{-6}\ \text{psi}^{-1} \qquad \text{for stage I bridge}$$

and

$$J(t_c, t_1) = \left(q_1 + q_o \ln\{1 + \Psi[t_1^{-m} + \alpha](t_c - t_1)^n\} + q_5\left[e^{-3H(t_c)} - e^{-3H(t_1)}\right]^{0.5}\right) \times 10^{-6}$$

$$= (0.134 + 2.54 \ln\{1 + 0.3[10^{-0.5} + 0.001](40 - 10)^{0.1}\} + 0.968[e^{-3\times 0.970} - e^{-3\times 0.991}]^{0.5}) \times 10^{-6}$$

$$= \quad 0.509 \times 10^{-6}\ \text{psi}^{-1} \qquad \text{for stage II bridge}$$

and

$$J(t_\infty, t_1) = \left(q_1 + q_o \ln\{1 + \Psi[t_1^{-m} + \alpha](t_\infty - t_1)^n\} + q_5\left[e^{-3H(t_\infty)} - e^{-3H(t_1)}\right]^{0.5}\right) \times 10^{-6}$$

$$= (0.134 + 2.54 \ln\{1 + 0.3[10^{-0.5} + 0.001](10000 - 10)^{0.1}\} + 0.968[e^{-3\times 0.717} - e^{-3\times 0.991}]^{0.5}) \times 10^{-6}$$

$$= \quad 0.925 \times 10^{-6}\ \text{psi}^{-1} \qquad \text{for both bridges}$$

Since the "compliance" is assumed to correspond to the deflection measured at the end of falsework release i.e. at t_2, Eq. (5) should be used. In this case, $J(t_2, t_1)$ can be calculated as

$$J(t_2, t_1) = \left(q_1 + q_o \ln\{1 + \Psi[t_1^{-m} + \alpha](t_2 - t_1)^n\} + q_5\left[e^{-3H(t_2)} - e^{-3H(t_1)}\right]^{0.5}\right) \times 10^{-6}$$

$$= (0.134 + 2.54 \ln\{1 + 0.3[10^{-0.5} + 0.001](13 - 10)^{0.1}\} + 0.968[e^{-3\times 0.987} - e^{-3\times 0.991}]^{0.5}) \times 10^{-6}$$

$$= \quad 0.414 \times 10^{-6}\ \text{psi}^{-1}, \text{ applicable for both bridges}$$

Thus the mid-span deflections for stage I bridge are

$$\delta^I_{midspan}(t_c + \Delta t_{age}) = \delta^I_{measured}\left[J\left(t_c + \Delta t_{age}, t_1\right)/J(t_2, t_1)\right]$$

$$= 1.30[0.662/0.414] = 2.08 \text{ in } (52.8\ mm)$$

$$\delta^I_{midspan}(t_\infty) = \delta^I_{measured}[J(t_\infty, t_1)/J(t_2, t_1)]$$

$$= 1.30[0.925/0.414] = 2.90 \text{ in } (73.7\ mm)$$

and the mid-span deflections for stage II bridge are

$$\delta^{II}_{midspan}(t_c) = \delta^{II}_{measured}[J(t_c, t_1)/J(t_2, t_1)]$$

$$= 1.25[0.509/0.414] = 1.54 \text{ in } (39.1\ mm)$$

$$\delta^{II}_{midspan}(t_\infty) = \delta^{II}_{measured}[J(t_\infty, t_1)/J(t_2, t_1)]$$
$$= 1.25[0.925/0.414] = 2.79 \text{ in } (70.9\ mm)$$

Finally the differential displacement demand on the closure slab, determined according to Eq. (7), is

$$\delta_d = \left(\delta^{II}_{midspan}(t_\infty) - \delta^{II}_{midspan}(t_c)\right) - \left(\delta^{I}_{midspan}(t_\infty) - \delta^{I}_{midspan}(t_c + \Delta \mathrm{t}_{age})\right)$$
$$= (2.79 - 1.54) - (2.90 - 2.08)$$
$$= 0.43 \text{ in } (10.9\ mm)$$

- The comparison of the displacement demand with the design displacement capacity indicates that

$$\delta_d = 0.43 \text{ in } (10.9\ mm) < \delta_c = 1.1 \text{ in } (27.9 \text{ mm})$$

which would mean that the closure slab has sufficient displacement capacity to tolerate the differential displacement across the closure slab. Thus the proposed 30 days waiting period for closure pour is deemed acceptable in the proposed methodology.

3.1.2 Example 2 - Three-span continuous Bridge

A widening situation is considered for a three-span continuous, prismatic bridge, as shown in Figure 7. It is assumed that all three spans are cast at the same time. Falsework is first released for span 1 at 28 days, followed by span 3 at 29 days, and then span 2 at 30 days. It is further assumed that the falsework release took less than one day to complete. The schedule for falsework release and closure pour essentially follows Alternative 2 of Caltrans practice [1]. Time parameters for this example are $t_1 = 28$ days for span 1, $t_1 = 29$ days for span 3 and $t_1 = 30$ days for span 2. The downward deflection is assumed to be measured for each span immediately after falsework release, with their values are listed in Table 1. The largest deflection occurs in span 2, which is the longest span in the bridge. It is also assumed that the concrete is cured for a period of $t_o = 7$ days for all three spans and the ambient relative humidity is $h = 0.6$. The mean 28-day compressive strength is taken as $\overline{f_c} = 6200$ psi (42.8 MPa), and an effective cross-section thickness of $D = 10$ in (250 mm) is used for all three spans. It is also assumed that closure is made by a slab of 36 in (910 mm) width and 8 in (200 mm) thickness. The differential displacement capacity of the closure slab from Figure 5 is $\delta_c = 1.4$ in (35.6 mm). Consideration will now be made to whether the waiting period for closure pour can be shortened to 1/2 of that specified for Alternative 2, i.e. $\Delta t_w = 7$ days, compared to the 14-day waiting period currently specified.

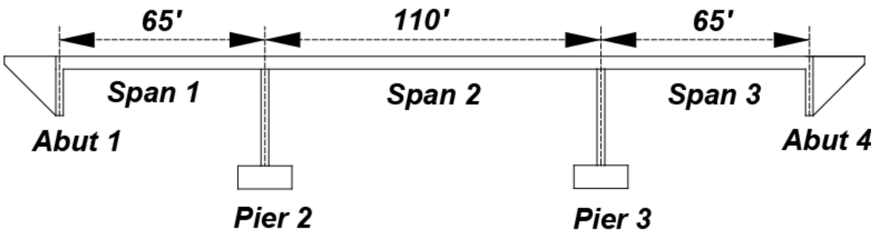


Figure 7. Example - 3-span continuous bridge

Table 1. Instantaneous span deflection upon falsework release

Span	δ^1_{max}	δ^2_{max}	δ^3_{max}	Falsework release
1	0.15 in (3.8 mm)	0 in	0 in	28 days
3	0.17 in (4.3 mm)	0 in	0.15 in (3.8 mm)	29 days
2	0.18 in (4.6 mm)	0.55 in (14.0 mm)	0.16 in (4.1 mm)	30 days

Using the design procedure as proposed:

- Since the duration of falsework release is less than one day, Eq. (4) will be used. The determination of the waiting period is made using the largest instantaneous deflection among the three spans, i.e.

$$\delta_{instant} = \delta^2_{max} = 0.55 \text{ in (14.0 mm)}$$

- Using the time for closure

$$t_c = t_1 + \Delta t_w = 30 + 7 = 37 \text{ days}$$

and

$$t_\infty = 10000 \text{ days}$$

For the mean 28-day compressive strength of $\overline{f_c} = 6200$ psi (42.7 MPa), $q_1 = 0.134, q_o = 2.54$ and $q_5 = 0.968$, the same as example 1.

$$H(t_1) = 1 - (1-h)\tanh\sqrt{\frac{t_1 - t_o}{32D^2}}$$

$$= 1 - (1-0.6)\tanh\sqrt{\frac{30-7}{32\times 10^2}}$$

$$= 0.966$$

$$H(t_1+\Delta t) = 1 - (1-h)\tanh\sqrt{\frac{t_1 + \Delta t - t_o}{32D^2}}$$

$$= 1 - (1-0.6)\tanh\sqrt{\frac{30.01-7}{32\times 10^2}}$$

$$= \quad 0.966 \text{ (essential the same as } H(t_1)\text{)}$$

$$H(t_c) = 1 - (1-h)\tanh\sqrt{\frac{t_c - t_o}{32D^2}}$$

$$= 1 - (1-0.6)\tanh\sqrt{\frac{37-7}{32\times 10^2}}$$

$$= 0.961$$

$$H(t_\infty) = 1 - (1-h)\tanh\sqrt{\frac{t_\infty - t_o}{32D^2}}$$

$$= 1 - (1-0.6)\tanh\sqrt{\frac{10000-7}{32\times 10^2}}$$

$$= 0.623$$

The creep compliance values for the widening bridge are

$$J(t_c,t_1) = (q_1 + q_o \ln\{1+\Psi[t_1^{-m}+\alpha](t_c - t_1)^n\}$$

$$+q_5\left[e^{-3H(t_c)} - e^{-3H(t_1)}\right]^{0.5}\Big) \times 10^{-6}$$

$$= (0.134 + 2.54\ln\{1 + 0.3[30^{-0.5} + 0.001](37-30)^{0.1}\}$$

$$+0.968[e^{-3\times 0.961} - e^{-3\times 0.966}]^{0.5}) \times 10^{-6}$$

$$= 0.325 \times 10^{-6}\ \text{psi}^{-1}$$

and

$$J(t_\infty,t_1) = (q_1 + q_o \ln\{1+\Psi[t_1^{-m}+\alpha](t_\infty - t_1)^n\}$$

$$+q_5\left[e^{-3H(t_\infty)} - e^{-3H(t_1)}\right]^{0.5}\Big) \times 10^{-6}$$

$$= (0.134 + 2.54\ln\{1 + 0.3[30^{-0.5} + 0.001](10000-30)^{0.1}\}$$

$$+0.968[e^{-3\times 0.623} - e^{-3\times 0.966}]^{0.5}) \times 10^{-6}$$

$$= 0.768 \times 10^{-6}\ \text{psi}^{-1}$$

Since the falsework release is assumed to have taken less than 1 day to complete, Eq. (4) should be used. In this case, $J(t_1, t_1)$ can be calculated as

$$J(t_1,t_1) \approx 1/E_c(t_1) = J(t_1 + \Delta t, t_1)$$

$$= J(30.01, 30)$$

$$= (q_1 + q_o \ln\{1+\Psi[t_1^{-m}+\alpha](t_2 - t_1)^n\}$$

$$+q_5\left[e^{-3H(t_2)} - e^{-3H(t_1)}\right]^{0.5}\Big) \times 10^{-6}$$

$$= \quad (0.134 + 2.54\ln\{1 + 0.3[30^{-0.5} + 0.001](30.01 - 30)^{0.1}\}$$

$$+0.968[e^{-3\times0.966} - e^{-3\times0.966}]^{0.5}) \times 10^{-6}$$

$$= \quad 0.222 \times 10^{-6}\ \text{psi}^{-1}$$

Thus deflections for the widening bridge at closure and at t_∞ are

$$\delta_{midspan}(t_c) = \delta_{instant}[J(t_c, t_1)/J(t_1, t_1)]$$

$$= 0.55[0.325/0.222] = 0.81\ \text{in}\ (20.6\ \text{mm})$$

$$\delta_{midspan}(t_\infty) = \delta_{instant}[J(t_\infty, t_1)/J(t_1, t_1)]$$

$$= 0.55[0.768/0.222] = 1.90\ \text{in}\ (48.3\ \text{mm})$$

which gives the differential displacement demand across the closure slab, according to Eq. (6), as

$$\delta_d = \delta_{midspan}(t_\infty) - \delta_{midspan}(t_c)$$

$$= 1.90\ \text{in} - 0.81\ \text{in}$$

$$= 1.09\ \text{in}\ (\ 27.7\ \text{mm})$$

- The differential displacement demand is therefore smaller than the estimated displacement capacity of the closure slab as

$$\delta_d = 1.09\ \text{in}\ (27.7\ mm) < \delta_c = 1.4\ \text{in}\ (35.6\ \text{mm})$$

Thus a closure slab of 36 in (910 mm) length and 8 in (200 mm) thickness is expected to have sufficient displacement capacity to accommodate the differential displacement across the closure slab. The shortening of the waiting period to 7 days for closure pour is deemed acceptable.

4 CONCLUSIONS

In widening of bridges, closure slabs are commonly used to connect new and old bridges to provide a smooth transition of the bridge deck. Final connection is to be delayed until sufficient deflection has occurred in the new bridge to minimize the potential damage in the connecting slab arising from differential deflection. Current practice in California requires a significant waiting period before closure pour can be made, up to 60 days in some cases, and the waiting time often leads to unnecessary delay in completion of bridges in widening or staged construction.

It is recognized that the waiting period in California does not take into account the displacement capacity of the closure slab nor time-dependent differential deflection imposed on the closure slab. The methodology proposed in this paper, which is capable of handling both bridge widening and staged construction cases, constitutes a more rational approach to the estimation of waiting period for closure pour. The premise of the approach requires the displacement demand, which is estimated by a normalized creep compliance function, to be less than the displacement capacity of the slab. The time-dependent deflection is shown to correlate well with field measurements for two box-girder bridges. Numerical examples indicate that the current waiting period may be unnecessarily conservative, especially for bridges with small instantaneous displacement and in staged constructed bridges where the differential displacement across the closure slab is expected to be small.

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5 APPENDIX

5.1 Creep compliance function

Concrete creep deformation manifests itself as a complex time-dependent response depending on many factors including the age of concrete at loading, ambient and curing conditions, type of cement, water-to-cement ratio, size and shape of the member, load duration etc. Many models have been proposed in the literature for predicting the creep deformation of concrete, with perhaps the ACI-209 [8], CEB-FIP Model Code [9], GL2000 model [10,11] and B3 model [5,12] among the most recognized. Most of these models are empirically based, requiring a varying number of input parameters depending on the level of sophistication intended in the model. While the procedure proposed in this paper can be adapted, in principle, to any of these models, the 'short form' or abridged version of the "B3" model [5] is used to estimate differential displacement between new and existing bridges. The short form of the B3 model is selected for its noted accuracy on creep prediction, yet sufficiently simple for general use in design or field offices, and is presented here for completeness.

Despite its intended broad application, the short form of the B3 model is nonetheless limited to Portland cement concrete, having a mean 28-day cylinder compressive strength in the range of 2500 to 10000 psi, a water-to-cement weight ratio 0.35 to 0.85, a cement content in the range of 10 to $45\,\mathrm{lb/ft^3}$ (160 to $721\,\mathrm{kg/m^3}$), an aggregate-to-cement weight ratio 2.5 to 13.5, and a minimum curing of 1 day [5]. The range of parameters is sufficiently large to encompass nearly all of the concrete used for bridge construction in California. The creep compliance of concrete is given by the sum of the instantaneous deformation, the basic creep deformation, and the additional deformation due to drying creep, and is expressed as

$$J(t,t_1) = \left(q_1 + C_o(t,t_1) + C_d(t,t_1,t_o)\right) \times 10^{-6} \text{ where } t_1 > t_o \qquad (8)$$

in which t is the time at which the deformation is to be estimated in days, t_1 is the age of concrete at loading, t_o is the age of concrete when drying starts i.e. at the end of curing, q_1 is the instantaneous strain due to a unit stress, scaled by a factor 1×10^6, to be calculated by

$$q_1 = \frac{0.6 \times 10^6}{E_{28}} \qquad (9)$$

where E_{28} is the elastic modulus of the concrete at 28 days in psi units, which can be calculated by $E_{28} = 57000\sqrt{\overline{f_c}}$, where $\overline{f_c}$ is the mean 28-day compressive strength of the concrete in psi units which may be related to the design strength f_c' by $\overline{f_c} = f_c' + 1200$ psi [5]. The term $C_o(t,t_1)$ in Eq. (8) corresponds to the compliance for basic creep, which is calculated from

$$C_o(t,t_1) = q_o \ln(1 + \Psi(t_1^{-m} + \alpha)(t - t_1)^n) \qquad (10)$$

where $q_o = 200\left(\overline{f_c}\right)^{-0.5}$ and material parameters $m = 0.5$, $n = 0.1$, $\alpha = 0.001$ and $\Psi = 0.3$. The term $C_d(t,t_1,t_o)$ in Eq. (8) corresponds to the compliance function for drying creep, which is calculated from

$$C_d(t,t_1,t_o) = q_5\left(e^{-3H(t)} - e^{-3H(t_1)}\right)^{1/2} \text{where } t_1 > t_o \qquad (11)$$

where $q_5 = 6000\left(\overline{f_c}\right)^{-1}$ in units of 1/psi, and parameters $H(t)$ and $H(t_1)$ are related to the cross-section shape of the member as well as the ambient relative humidity, which is calculated by

$$H(t) = 1 - (1 - h)S(t) \qquad (12)$$

in which h is the ambient relative humidity in decimal value with $0 \le h \le 1$ and $S(t)$ is a time-dependent function given by

$$S(t) = \tanh \sqrt{\frac{t - t_o}{\eta_{sh}}} \quad (13)$$

where t_o is the age of concrete at the end of curing and η_{sh} is a size-dependent parameter given by

$$\eta_{sh} = 32D^2 \quad (14)$$

in which $D = 2v/s$ is an effective cross-section thickness in inches, where $v =$ volume of the member in in^3 and $s =$ surface area of the member in in^2. Hence v/s represents the volume-to-surface ratio of the member. Note that for a slab, D is simply equal to the slab thickness.

Determination of the normalized bridge deflection, for purposes of determining the waiting period for closure pour, necessitates the calculation of the creep compliance function at the time of falsework release. The substitution of $t = t_1$ into the creep compliance function in Eq. (8) however gives rise undesirably to $J(t_1, t_1) = q_1 \times 10^{-6} = 0.6/E_{28}$, which is a constant independent of the time of falsework release. Since the value of the creep compliance function at the time of falsework release corresponds to the elastic deflection of the bridge, a constant compliance does not represent the elastic deflection of the bridge which is expected to vary upon falsework release at different ages. To overcome the deficiency, the elastic modulus of the concrete is determined by a modification of the creep compliance function, as recommended by [5]

$$E_c(t_1) = 1/J(t_1 + \Delta t, t_1) \quad (15)$$

where $E_c(t_1)$ represents the elastic modulus of the concrete at time of falsework release, and Δt corresponds to a time shift, called a "stress duration", which may be taken as 0.01 day [5]. The creep compliance $J(t_1, t_1)$ at the time of falsework release can thus be approximated by

$$J(t_1, t_1) \approx \frac{1}{E_c(t_1)} = J(t_1 + \Delta t, t_1) \quad (16)$$

By selecting a value of $\Delta t = 0.01$ day, Eq. (15) has been noted to give an elastic modulus that is in reasonable agreement with ACI's elastic modulus given by $57000\sqrt{\bar{f}_c(t_1)}$, where $\bar{f}_c(t_1)$ is the mean compressive strength of the concrete at the time of falsework release in psi units [5].