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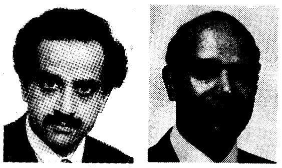
Computer Analysis of Spliced Girder Bridges



by Ahmad M. Abdel-Karim and Maher K. Tadros

To meet the growing demand for long span bridges, techniques for increasing the span range of existing girder shapes by splicing relatively short girder segments have been developed over the last 30 years. The state-of-the-art of spliced-girder bridges is reviewed briefly, followed by a description of a computer-based procedure for analysis of composite precast concrete girder bridges with cast-in-place topping. The computer program can be employed to evaluate stresses in concrete and steel at any cross section in a statically indeterminate composite beam or plane frame. It also gives the deflection at various stages of construction. The method of analysis presented here is also applicable to other types of multistage construction and prestressing involving the use of temporary falsework. A numerical example is presented to illustrate the hand calculations involved and the use of the computer program.

Keywords: bridges (structures); computer programs; continuity (structural); deflection; **girders;** I-beams; **post-tensioning;** precast concrete; prestressed concrete; segmental construction, serviceability; **splicing.**

Splicing of precast concrete bridge girders is not a new concept. It was used in the late 1960s and early 1970s to overcome transportation limitations on precast girder segments to extend their use to longer span bridge applications. This system, however, lacked the publicity of segmental box girder bridges. Unavailability of publications on splicing details and design examples further hindered a widespread use of this system. In an effort to provide such documentation, a recent survey on the design and construction of spliced-girder bridges (SGB) was conducted by the authors of this paper at the University of Nebraska-Lincoln (UN-L). The survey is part of a research program sponsored in part by the Precast/Prestressed Concrete Institute (PCI), and in part by the Center for Infrastructure Research (CIR) at UN-L. The study resulted in a state-of-the-art report on SGB design and construction practices, recently published as an official PCI document.

The UN-L research team developed a microcomputer-based analysis program of SGB. The analysis method used accounts for the interaction between the various materials constituting the bridge superstructure during the various stages of production, construction, and after bridge completion until creep and shrinkage effects are fully developed. Thermal effects caused by daily or seasonal temperature changes are also accounted for. The method was made general enough to encompass a wide range of problems encountered in practice.

A special effort was made to render the program as user friendly as possible, while maintaining its generality. The input data is limited to information commonly available to designers. The results of the computer program include forces, concrete and steel stresses, and deflections at all stages. To illustrate the procedures involved in the design of SGB, a preliminary design of a 350-ft long, two-span example bridge is included in this paper.

RESEARCH SIGNIFICANCE

Successful design of this type of bridge requires accurate prediction of the stresses and deformations during all stages of construction. This is particularly true in light of the continuing trends toward even longer spans and higher concrete strengths. An accurate prediction tool helps the engineer optimize his design for structural efficiency and economy. It also raises the level of confidence of the engineer in his design, thus reducing his reliance on excessive factors of safety. Time-dependent and temperature effects can have considerable influence on the behavior of the bridge superstructure. The analysis complications introduced by these important factors make hand calculations difficult in even the simplest of cases.

STATE OF THE ART

Prestressed concrete bridges in the range of 100 to 150 ft (30 to 46 m) have become common. In most cases, the girders for these structures are fabricated in lengths to accommodate the spans, and are sometimes spliced over the piers to provide continuity to resist the bending moments caused by live loads and superimposed dead loads. This type of construction is very cost-effective since the girders can be erected in one piece without falsework. However, some states prohibit the shipment of girders with lengths greater than approximately 130 ft (40 m), and in some cases the long girders have weights that exceed the capacity of the existing bridges on the ship-

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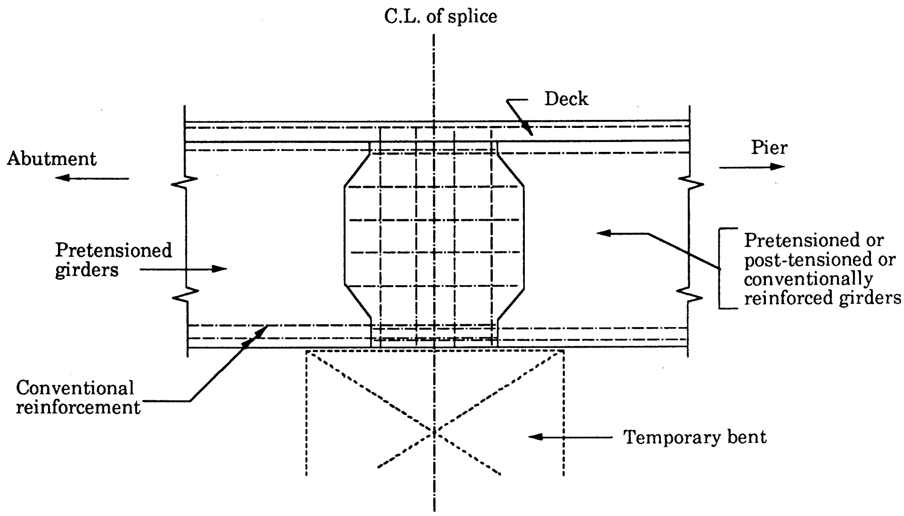
ping route. When concrete girders are too long, or too heavy to be shipped in lengths to accommodate the spans, segments are spliced in the span, with or without splicing at the piers.

In the design of concrete bridges using spliced girders, the span layout is determined by site conditions. The maximum segment size is usually governed by transportation constraints and facilities at the fabrication plant. Segment sizes are selected to accommodate these constraints and provide splices at locations that are accessible at the construction site, or at locations dictated by the flexural stresses in the girder.

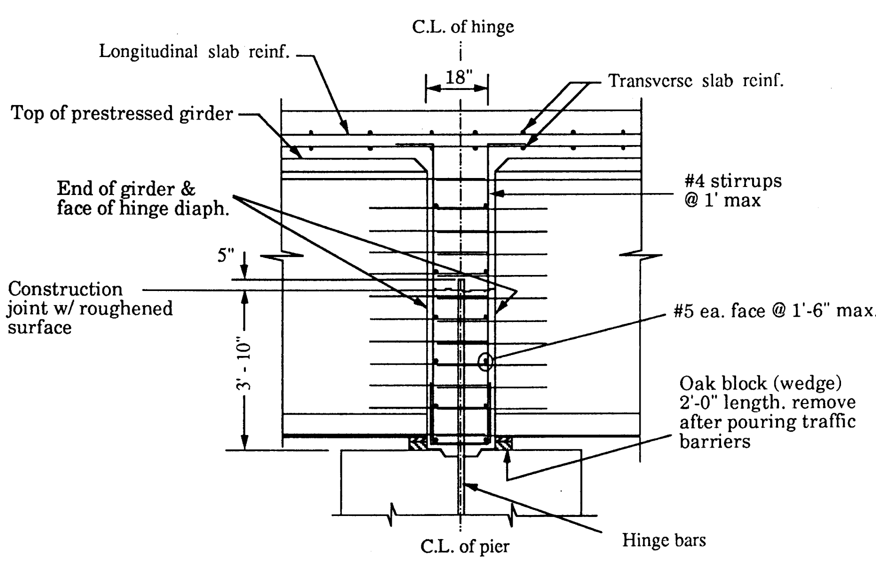
One of the features that is often overlooked as an advantage of this type of construction is the flexibility which it provides the designer in selecting the span lengths, number and location of piers, segment lengths, and splice locations. In any given situation, several different combinations of these options may be applicable. It is up to the designer to choose the most economical combinations that meet the various criteria dictated by the situation. Following is a brief description of the most commonly used types of splices of I-girders.

Conventionally reinforced splice

For continuous girders, reinforced concrete splices are usually located near the inflection point of the superimposed dead load moment diagram. Moments due to live loads are usually small at this location. The splice is made using falsework to erect precast girder segments that have reinforced projecting from the ends of the segments. This projecting reinforcement is lap spliced using cast-in-place (CIP) concrete. Sufficient



(a) Conventionally reinforced field splice



(b) Typical hinge diaphragm section over the pier

Fig. 1 — Cast-in-place reinforced concrete splices

length must be provided in the CIP splice to develop the lap splice in the reinforcement. Details of a reinforced concrete CIP field splice are shown in Fig. 1.

CIP post-tensioned splice

CIP post-tensioned splices may be used with conventionally reinforced or pretensioned girder segments. The segments are erected on falsework, and the post-tensioning tendons are installed. Concrete for the deck slab may be placed at the same time as the concrete for the splice, or the deck concrete may be placed after the splice has been post-tensioned. Fig. 2 shows details of a CIP post-tensioned splice.

Epoxy-filled post-tensioned splice

When short splices are desired, conventional reinforced concrete girder segments may be jointed by epoxy grout and post-tensioning. This type of splice does not require match casting or the use of precision bulkheads. Assembly of the girder segments can be done either in the final erected position, or on the ground. In both cases, the girder segments to be spliced are aligned as desired, with a small opening between the girder ends. Compressible gaskets are provided for the tendon conduit. Forms are then erected around the opening between the segment ends and the opening is filled with epoxy grout (see Fig. 3). After the epoxy grout has cured, the splice is post-tensioned and the conduit is filled with conventional grout.

Other types of splices include the "stitched" splice, drop-in splice, and the structural steel splice. These types are described in detail in Reference 1.

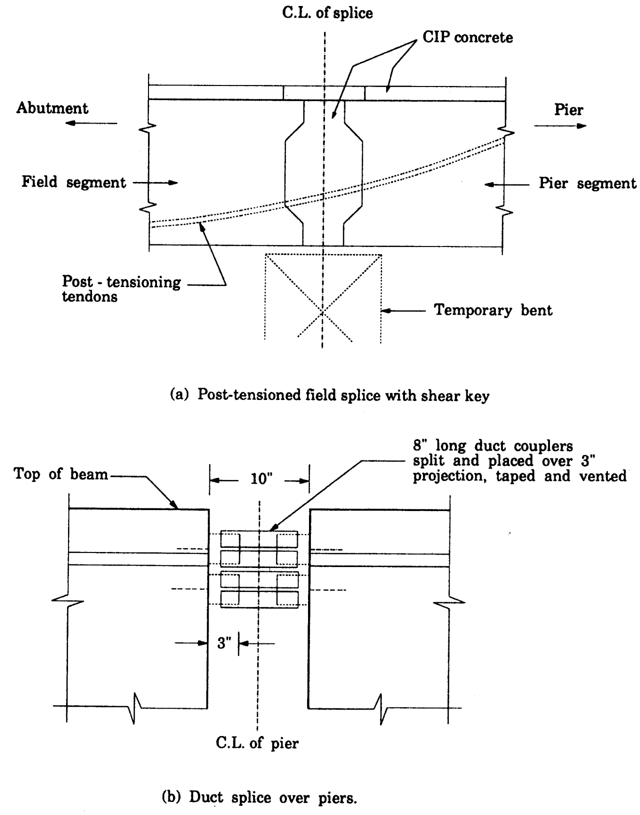


Fig. 2 — Cast-in-place post-tensioned splice

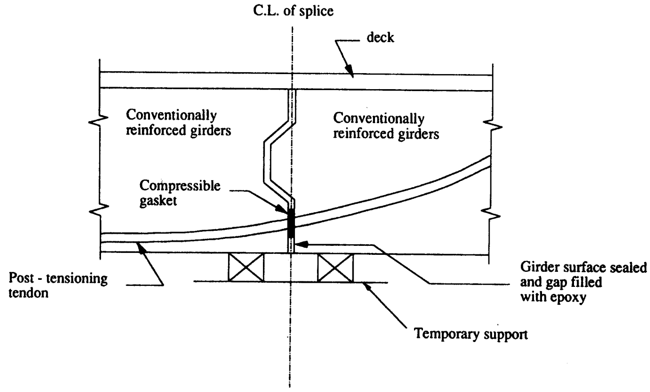


Fig. 3 — Epoxy-filled post-tensioned splice

VOLUME CHANGES IN COMPOSITE MEMBERS

In most cases involving stage construction, it is important to calculate accurately the cambers and deflections of the components of the structure during the various stages. The geometric compatibility of the different segments and the resulting riding surface can depend on the accuracy with which these calculations are performed. Therefore, the method of analysis should take into account the effects of creep and shrinkage of concrete, relaxation of steel, and temperature gradients.

Creep of concrete under constant stress condition

The instantaneous (elastic) strain in concrete due to a constant stress increment can be written as follows

$$\varepsilon_{ce} = \frac{f_c}{E_c} \tag{1}$$

where f_c is the concrete stress, and E_c is the secant modulus of elasticity of concrete.

Under a constant stress condition, the strain continues to increase with time due to creep. As shown in Fig. 4, when a constant stress f_c is applied at time t_1 and sustained until time t_2, total strain at time t_2, $\varepsilon_c(t_2)$, which is the instantaneous plus creep strains, can be stated as follows

$$\varepsilon_c(t_2) = \frac{f_c}{E_c(t_1)} = [1 + C(t_2,\ t_1)]\varepsilon_{ce} \tag{2}$$

where $C(t_2,t_1)$ is a creep coefficient that represents the ratio of creep to instantaneous strain, and a function of concrete age at loading t_1 and duration for which creep is calculated. Its value increases with the decrease of age at loading and with the increase of duration for which the stress is sustained. The correction factors that affect the creep coefficient are given in the reports of the CEB-FIP Committee[2] and ACI Committee 209.[3]

The total strain, instantaneous plus creep, is linearly proportional to the applied stress. This linear relationship is generally acceptable within the range of stress in service conditions and allows the superposition of strains.

Shrinkage of concrete

Shrinkage is defined as deformation unrelated to load application or temperature change. Drying of concrete results in reduction in volume. Shrinkage occurs at a high rate initially and decreases over time.

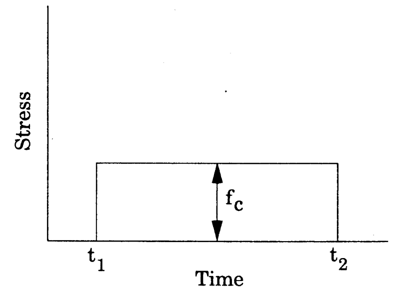
The main factors influencing shrinkage are similar to those of creep: ambient relative humidity, thickness of members, water-cement ratio, aggregate type, and method of curing. The equations to calculate shrinkage are given in the CEB-FIP Committee[2] and ACI Committee 209[3] publications.

Temperature effects

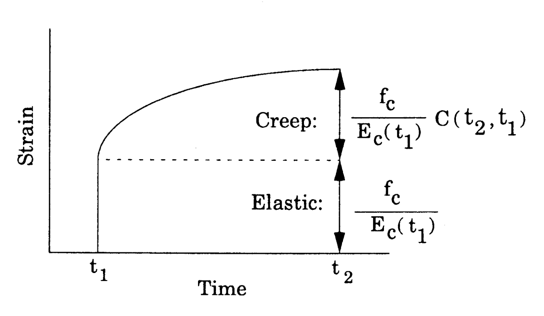
Thermal stresses and movements are among the major considerations in bridge design. The American Association of State Highway and Transportation Officials (AASHTO)[4] requires that provisions be made to account for the effects resulting from temperature variations.

Thermal effects become even more significant as the trend towards jointless bridges continues. The expansion joints, which were relied upon heavily to relieve thermal stresses, are being eliminated to reduce maintenance costs and improve the riding surface of the bridge. Also, the increasing popularity of girder splicing naturally eliminates the need for expansion joints. As a result, bridges that were typically constructed as simple spans in the past are now made continuous. Unlike those in statically determinate structures, thermal strains generated in indeterminate structures can cause considerable stresses. Due to the growing popularity of continuous (statically indeterminate) bridge superstructures, there is an increasing tendency to design bridges to resist thermal stresses as opposed to providing for thermal movements.

Both short-term (daily) and long-term (seasonal) temperature changes contribute significantly to the longitudinal deformation of bridges. In the daily cycle, the surface exposed to the sun (the deck slab) usually gets heated more quickly than the shaded portions. As a result, temperature gradients similar to the one depicted in Fig. 5(c) can result.[9] The yearly cycle is related to the seasonal variation of temperature. Its effect usually exceeds that of the daily cycle. Bridges that are constructed in extreme weather are usually more vulnerable to seasonal changes. For example, bridges that are constructed in the warm season suffer considerable thermal effects during the cold season, and vice versa. An example of the temperature distribution across the depth of the girder due to seasonal changes is shown in Figure 5(d).



(a) Constant stress increment in concrete



(b) Corresponding strain of concrete due to a constant stress

Fig. 4 — Stress-strain relation for constant stress

Relaxation of prestressed steel

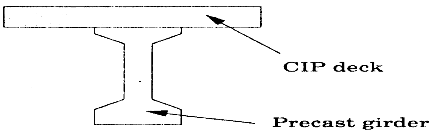
Relaxation is defined as the loss of stress in a prestressed steel when it is kept at constant length. It is similar to the phenomenon for creep of concrete. Relaxation that occurs under the constant strain between two fixed points (maintain constant length) is called intrinsic relaxation. The amount of the intrinsic relaxation is dependent mainly upon initial prestress force and time.

One of the equations to calculate intrinsic relaxation based upon a PCI Committee report[5] is

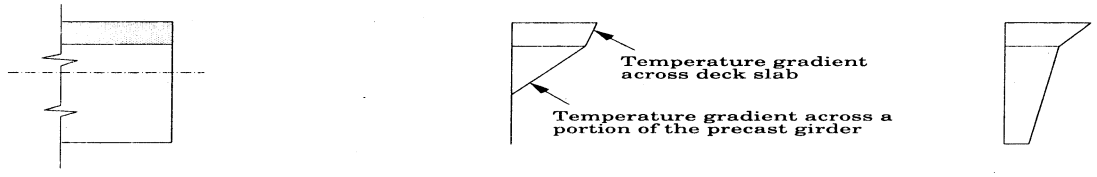
$$f_{ro} = kf_{pi}\left(\frac{f_{pi}}{f_{py}} - 0.55\right) \log\left(\frac{24t+1}{24t'+1}\right) \tag{3}$$

k = constant: $\frac{1}{10}$ = stress-relieved steel
$\frac{1}{45}$ = low-relaxation steel

f_{pi} = initial stress in prestressed steel at time t' in days



(a) Cross section



(b) Girder Elevation (c) Daily temperature changes (°F) (d) Seasonal temperature changes (°F)

Fig. 5 — Temperature distributions in composite girders

f_{py} = stress at 1 percent elongation of prestressed steel
f_{ro} = intrinsic relaxation = loss of prestress due to steel relaxation over time of the period $(t - t')$ in days

METHOD OF ANALYSIS

Analysis equations

The method presented here is based on the step-by-step method developed by Tadros, Ghali, and Dilger.[6] First, the time duration considered in design is divided into discrete time intervals. The length of each time interval may be determined depending upon the time at which various loading stages are applied. Fig. 6 shows the definition of time intervals in this method. *Y*-axis in this figure indicates actions that may be strains, stresses, normal forces, or bending moments. Due to various loads such as prestress forces, self-weight of the members, and any other superimposed loads, the instantaneous actions occur at the specific applied time, and then are sustained thereafter. That is, loading causes sudden variation in actions. The length of intervals of suddenly applied loads is assumed to be zero. It is assumed here that the time intervals are chosen such that the gradual change in the sustained actions due to time-dependent effects is small and can be neglected. Because time-dependent effects are most appreciable immediately following the application of a sustained load increment, it is recommended that small intervals be chosen in that period of time. The length of the intervals can be increased progressively as time goes on. As shown in Fig. 6, i - ½, i, and i + ½ refer to the beginning, middle and end of interval i, respectively. The length of interval i-1 is equal to zero since sudden actions are introduced during this interval i-1.

Using the principle of superposition, the constitutive equation to refer to total strain at the end of an intermediate interval i may be written as follows

$$\varepsilon_c\left(i+\frac{1}{2},\ 0\right)=\sum_{j=1}^{i}\frac{\Delta f_c(j)}{E_c(j)}\left[1+C\left(i+\frac{1}{2},j\right)\right]+\varepsilon_{sh}\left(i+\frac{1}{2},0\right) \tag{4}$$

where $\varepsilon_c(i + ½, 0)$ is total strain of concrete occurring between interval 1 and interval i. $\varepsilon_{sh}(i + ½, 0)$ is free shrinkage from interval 1 to interval i. $\Delta f_c(j)$ and $E_c(j)$ are the stress increment at interval j and modulus of elasticity of concrete at the beginning of interval j, respectively. $C(i + ½, j)$ represents the creep coefficient in the period of time $(i + ½ - j)$.

Nonprestressed steel is considered to be elastic material obeying Hook's Law. The relation between strain and stress is

$$\varepsilon_{ns}=\frac{f_{ns}}{E_{ns}} \tag{5}$$

The subscript ns refers to nonprestressed steel. For prestressed steel, the relaxation of the prestressed tendon should be taken into account. Total strain of prestressed steel at the end of interval i is

$$\varepsilon_{ps}\left(i+\frac{1}{2},\ 0\right)=\frac{1}{E_{ps}}\sum_{j=1}^{i}[\Delta f_{ps}(j)\ -\ \Delta f_r(j)] \tag{6}$$

where ps refers to prestressed steel, and $\Delta f_{ps}(j)$ and $\Delta f_r(j)$ are the stress increment and the relaxation stress during j, respectively. Based on the equations of these three materials, creep analysis is developed in the next section.

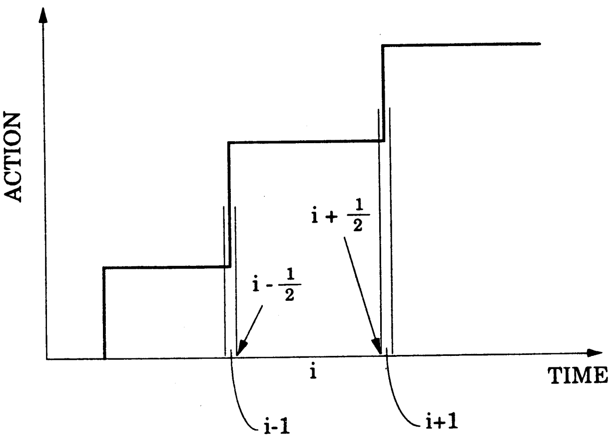


Fig. 6 — Actions versus time

Time-dependent analysis

Time dependent analysis in the step-by-step method is performed by using the "initial" strain concept. Initial strain is that strain which is independent of the change in strain during a certain interval considered. By using the initial strain concept, the analysis can be transformed into an incremental linear analysis. The creep problem can thus be converted into a pseudoelastic problem. The term "initial" should not be confused with the instantaneous elastic deformation.

The stress at the end of each interval is calculated in terms of stress increments that have occurred in preceding intervals. Thus, to get an increment axial concrete strain during the time interval i, i in Eq. (1) is replaced by i-1, and this replaced equation is deducted from Eq. (1). Linear creep law is adopted, and the stress distribution of a cross section is always linear. Thus, during interval i an increment axial strain $\Delta\varepsilon_c(i)$ occurs

$$\Delta\varepsilon_c(i)=\frac{\Delta N_c(i)}{A_c E_{ce}(i)}+\Delta\varepsilon_c'(i) \tag{7}$$

where $\Delta\varepsilon_c(i)$ is the strain increment in concrete during interval i, $\Delta\varepsilon_c'(i)$ is defined by

$$\Delta\varepsilon_c'(i)=\sum_{j=1}^{i-1}\frac{\Delta N_c(j)}{A_c E_c(j)}\left[C\left(i+\frac{1}{2},j\right)-C\left(i-\frac{1}{2},j\right)\right]+\Delta\varepsilon_{sh}(i) \tag{8}$$

where j refers to the time at the middle of the jth intervals, and $(i - ½)$ and $(i + ½)$ are the time at the beginning and end of the ith interval, respectively; $\Delta\varepsilon_{sh}(i)$ is free shrinkage strain during interval i, $\Delta N_c(j)$ and $E_c(j)$ are the axial force increment and the modulus of elasticity of concrete at the middle of interval j, respectively; and A_c is the cross-sectional area of concrete. $\Delta\varepsilon_c'(i)$ is defined as "initial" strain.

In Eq. (8), it is assumed that the stress variation of concrete in preceding intervals continuously occurs in the middle of these intervals. Initial strain is independent of stress introduced at interval i. It is computed from summing up results of the preceding intervals. That is, the strain increment occurring in interval i is equal to the first term of the right-hand side in Eq. (7) and expresses the strain increment in the ith interval plus initial strain due to creep and shrinkage. Initial strain is assumed to be known before using Eq. (7).

The effects of gradual stress change within interval i itself is accounted for by the use of the age-adjusted effective modulus of concrete E_{ce}, defined by Eq. (9)

$$E_{ce}(i) = \frac{E_c(i)}{1 + C\left(i + \frac{1}{2},\ i\right)} \tag{9}$$

This implies that the effect of gradual stress is assumed equal to that of an instantaneous stress occuring at the middle of interval i. This approximation is adequate in practice, as gradual "loading" occurs due to time-dependent effects.

Similar equations for the moment-curvature relation can be developed by

$$\Delta\phi_c(i) = \frac{\Delta M_c(i)}{I_c E_{ce}(i)} + \Delta\phi_c'(i) \tag{10}$$

where $\Delta\phi_c'(i)$ is defined by

$$\Delta\phi_c'(i) = \sum_{j=1}^{i-1} \frac{\Delta M_c(j)}{I_c E_c(j)} \left[C\left(i + \frac{1}{2}, j\right) - C\left(i - \frac{1}{2}, j\right) \right] \tag{11}$$

$\Delta f_c(i)$ is the incremental curvature in interval i, $\Delta f_c'(i)$ is initial curvature, and ΔM_c and I_c are bending moment increment and moment of inertia of concrete, respectively. Initial (free) strains and curvatures due to temperature gradients may be included in Eq. (8) and (11), similar to $\Delta\varepsilon_{sh}(i)$ of Eq. (8).

The constitutive relation for prestressed steel is as follows

$$\Delta\varepsilon_{ps}(i) = \frac{\Delta N_{ps}(i)}{A_{ps} E_{ps}} + \Delta\varepsilon_{ps}'(i) \tag{12}$$

When initial strain in prestressed steel $\Delta\varepsilon'_{ps}(i)$ is defined by

$$\Delta\varepsilon_{ps}'(i) = -\frac{\Delta f_r(i)}{E_{ps}} \tag{13}$$

$\Delta\varepsilon'_{ps}(i)$ is independent of the stress increment in interval i, which is shown in the first term of the right-hand side of Eq. (12). $\Delta f_r(i)$ is defined as reduced relaxation of prestressed steel in interval i. Eq. (3) for intrinsic (fixed length) relaxation is adopted here. The value of relaxation is actually less than that of intrinsic relaxation due to creep and shrinkage of concrete. A reduction factor is introduced in determining accurate relaxation. The relation between intrinsic relaxation f_{ro} and reduced relaxation f_r is

$$f_r = x_r f_{ro} \tag{14}$$

where x_r is the relaxation reduction coefficient. It is a function of tendon shortening due to creep and shrinkage of surrounding concrete, and of the ratio of initial to ultimate steel of steel used. x_r may be obtained from a table by Ghali and Neville[7] or by using a formula given by Elbadry and Ghali.[8] Importance of the relaxation reduction coefficient x_r diminishes as the time interval length is reduced. Also, the fact that low-relaxation tendons are becoming common in practice, the accuracy resulting from use of x_r may not always justify time-consuming iterative solutions.

A member composed of concrete and prestressed steel has the time-dependent variation with stress and strain. The constitutive relations for time-dependent materials that were discussed previously are used for the step-by-step method. The creep analysis is therefore transformed into the

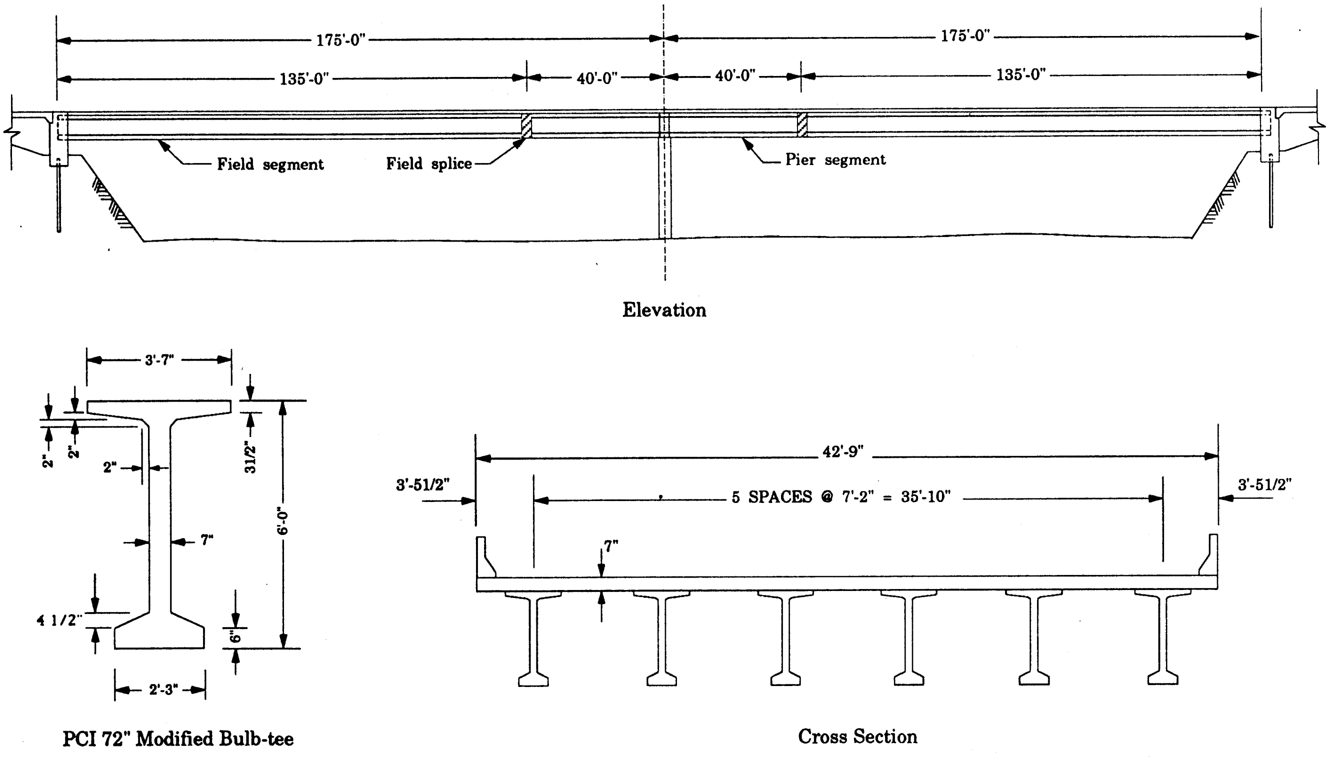


Fig. 7 — General layout of the two-span bridge example

incremental procedure in which the stress and strain increments can be evaluated in an intermediate interval i. Let the beginning interval = 1 and the total number = m. The procedure is done from the first i = 1 to the last interval i = m by a step-by-step manner. For any interval i, initial strain is expected to be known by use of the values occurring in preceding intervals (up to interval i-1). The different elastic materials should be separated in obtaining the initial strains.

This step-by-step method is suitable for computer-based analysis, and for incrementally constructed bridges.

COMPUTER PROGRAM

Scope and capabilities

The main source code routines of the computer program described here were developed by Tadros.[6] Numerous changes and additions were made over the years since the program first appeared. A PC version of the program was recently developed by Abdel-Karim.[1] Several new features were added to this newest version. The program presents a procedure for analysis of composite precast concrete girder bridges with cast-in-place topping. Girder splicing with or without post-tensioning is used to introduce continuity in the bridge superstructure. The method of analysis presented here is also applicable to other types of multistage construction and prestressing involving the use of temporary falsework. The computer program can be employed to evaluate stresses in concrete and steel at any cross section in a statically indeterminate composite beam or plane frame. It also gives the deflection at various stages of construction. Additional features of the program include up to three different materials in a cross section; concrete hardening; temperature effects; supports settlement; internal generation of prestressing profiles; calculation of friction and anchorage set losses; multistage post-tensioning; and shear deformation. The program also offers limited node and element data generation capabilities.

DESIGN EXAMPLE

A two-span bridge example will be used here to illustrate the procedure involved in preparing the input data for the computer program. The essential problem data is summarized in the following. Fig. 7 shows the bridge layout and cross section. In the preliminary design of this example, a lump-sum procedure was used to account for the time-dependent effects. However, the final design of the bridge should be based on a more detailed method of analysis such as the one described previously.

Construction schedule

The bridge is assumed to be constructed according to the schedule shown in Table 1.

Material properties

Concrete — Stage 1 concrete (precast girder concrete): self weight = 150 lb/ft^3 (23.56 kN/m^3); $f_c'(28)$ = 7000 psi (48.3 MPa); $E_c(28)$ = 5072.2 ksi (34, 973 MPa).

Stage 2 concrete (CIP splice and deck concretes): self weight = 150 lb/ft^3 (23.56 kN/m^3); $f_c'(28)$ = 4000 psi (27.6 MPa); $E_c(28)$ = 5072.2 ksi (34,973 MPa).

Steel — Prestressing steel: Grade 270 low-relaxation, E_s = 28,000 ksi (193,060 MPa). Mild steel: Grade 60 steel, E_s = 29,000 ksi (199,955 MPa).

Table 1

Construction step	Days since precasting
Pretension and pour concrete	0
Release of pretensioning force	1
Erection of precast beams	28
CIP deck and joints	35
Post-tensioning and support removal	42
Application of superimposed dead load (SIDL)	60
Application of live load (bridge open to traffic)	70

Bridge idealization

Because the bridge is symmetric about the centerline of the middle pier, it is necessary to model only one span. A descretized model of the bridge showing the elements, nodes, cross sections, and nodal coordinates is given in Fig. 8. This figure also shows the boundary conditions and the times during which they apply. The pretensioning and post-tensioning forces and profiles are given in Fig. 9.

Geometric properties

The properties of the two concrete types, identified in Fig. 8, are shown in Table 2.

y, A, and *I* are the centroidal distance, area, and moment of inertia of the stage concrete. The distance *y* is measured from a reference level, chosen here to be the top of the precast girder.

Time intervals

The time intervals are selected based on the construction schedule and loading sequence of the bridge in question. A total of 19 time intervals are chosen in this example. The interval limits in days are shown in Table 3.

An interval length of zero signifies a sudden change in loading or boundary conditions.

Loads

A total of six loading cases are considered. They are shown in Table 4 [see also Fig. 10(a) and (b)].

The pretensioning force used is equal to the force immediately before release. The instantaneous prestress loss due to elastic shortening of the girders is computed internally by the program. Because post-tensioning is assumed to take place in one step, no instantaneous loss in the post-tensioning cables occurs. In the case of sequential post-tensioning of different tendons, a separate time interval should be assigned to each post-tensioning process. The elastic shortenings and the resulting losses of prestress in previously post-tensioned cables are accounted for internally.

The results of the analysis using the computer program are compared to those obtained from hand calculations in Fig. 11. In this figure, the stresses in the top fibers at the maximum positive moment section at B are shown versus the time since casting of the girders. Since both analyses are based on elastic solutions, they result in the stresses as expected. However, this exercise illustrates how the program can be used conveniently to verify preliminary hand calculations done prior to the final design.

The computer solution is repeated with essentially the same data as before, but with no simplifying assumptions regarding

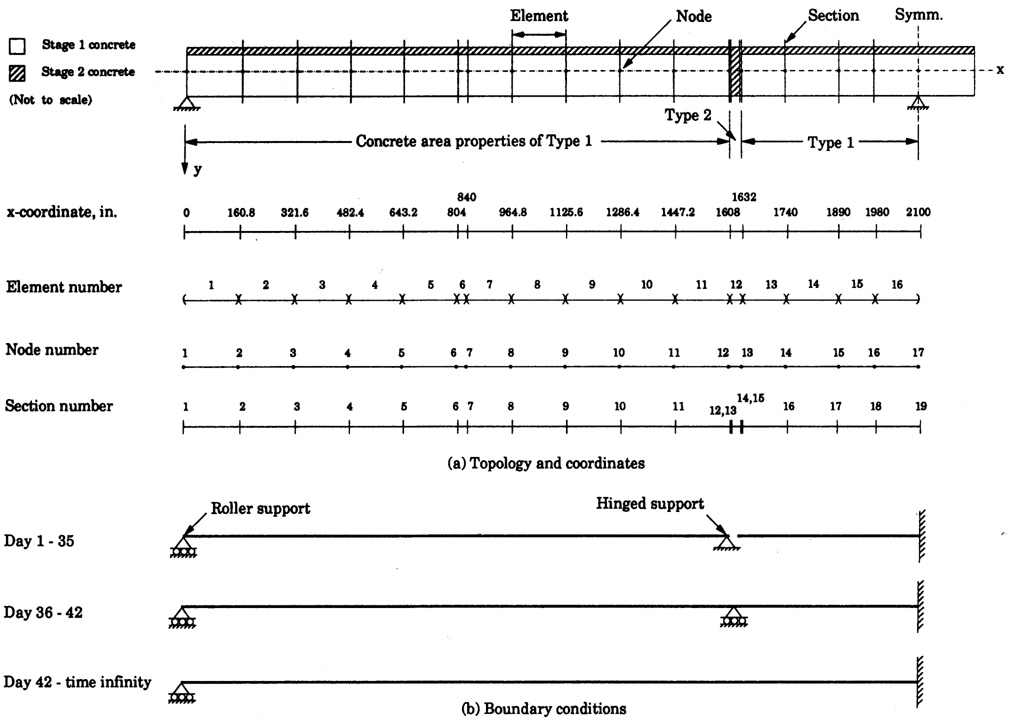


Fig. 8 — Bridge idealization

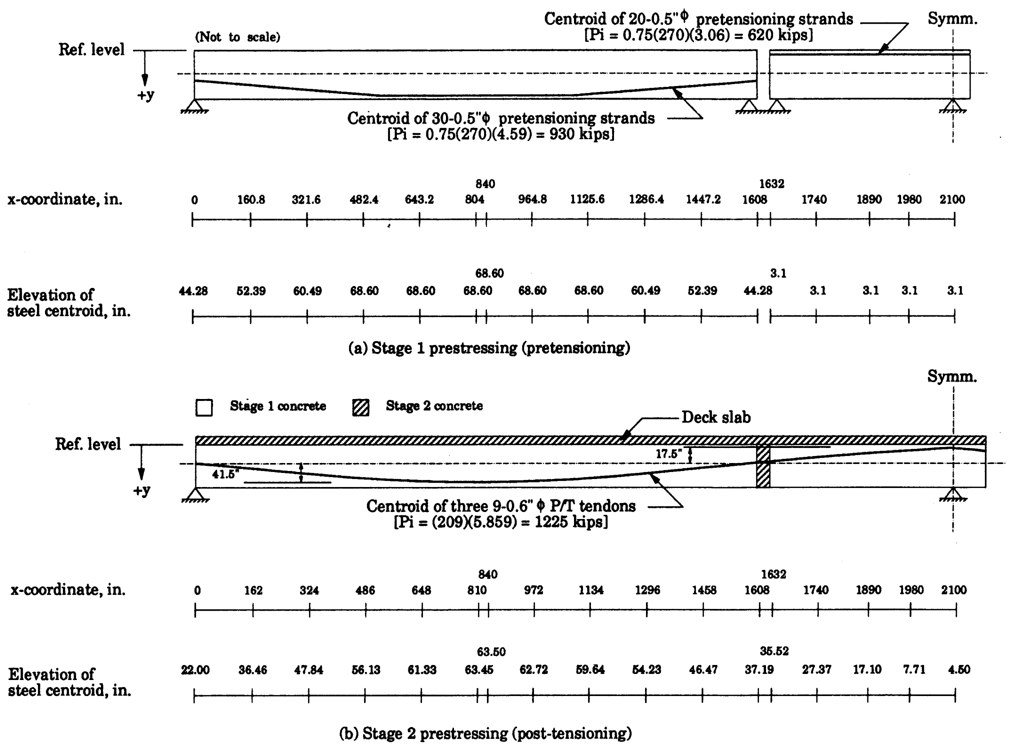


Fig. 9 — Prestressing profiles and eccentricities

time-dependent effects. This analysis also accounts for friction and anchorage set losses, which results in a various tendon force profile. The results of this analysis are given in Fig. 12, which also shows the results of the previous simplified analysis for comparison purposes.

CONCLUSIONS AND RECOMMENDATIONS

As the need for longer span, jointless bridges continues, so does the need for reliable methods of analysis. Due to the complicated nature of problems involving composite, spliced-girder bridges constructed in stages, hand calculations are often very difficult. The computer program presented here is a step toward satisfying the growing need for accurate analysis tools for everyday use in design offices.

One important benefit of the computer program is that it enables the user to conduct parametric studies on a number of selected variables to optimize his design. Examples of these variables include span lengths, pretensioning or post-tensioning steels, concrete strengths, and construction schedule. Such studies help the designer determine the effect of a selected group of variables on the final design. By varying one variable at a time, while keeping all the other variables unchanged, the effect of this one variable on stresses, deflections, or any other relevant quantity can be isolated and evaluated. Based on this evaluation, certain conclusions can be made regarding the value of this variable to be used or tolerated in the final design. Also, by comparing the effects of a number of variables on the behavior of the structure, the user may modify the design to reflect some of the trends revealed by the parametric study.

The design of a spliced girder bridge must consider all aspects of the fabrication and construction sequence. Proper attention to details is important to assure a practical and economical design. Site conditions, availability of lifting equipment, shipping restrictions, and capability of local precasters and constructors must be considered in the selection of the size of segment and type of splice to use.

Table 2

	Stage 1, girder			Stage 2, CIP deck		
Section type*	*y*, in.	*A*, in.^{2}†	*I*, in.^{4}	*y*, in.	*A*, in.^{2}	*I*, in.^{4}
1 (prismatic)	35.45	839	576,978	−3.5	−7.0	2458
2 (diaphragm)	0	0	0	32.5	6794	3,533,446

*See Fig. 8.
†1 in. = 25.4 mm.

Table 3

Time limits	1	1	1	3	7	15	28	30	35	35	42	42	42	50	60	60	100	300	800	2000
Intervals	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	

Table 4

Loading case	Introduced at interval
Pretensioning force immediately before transfer	1
Precast girder self-weight	2
Deck self-weight	9
Post-tensioning	11
Support removal	12
Superimposed dead load (SIDL)	15

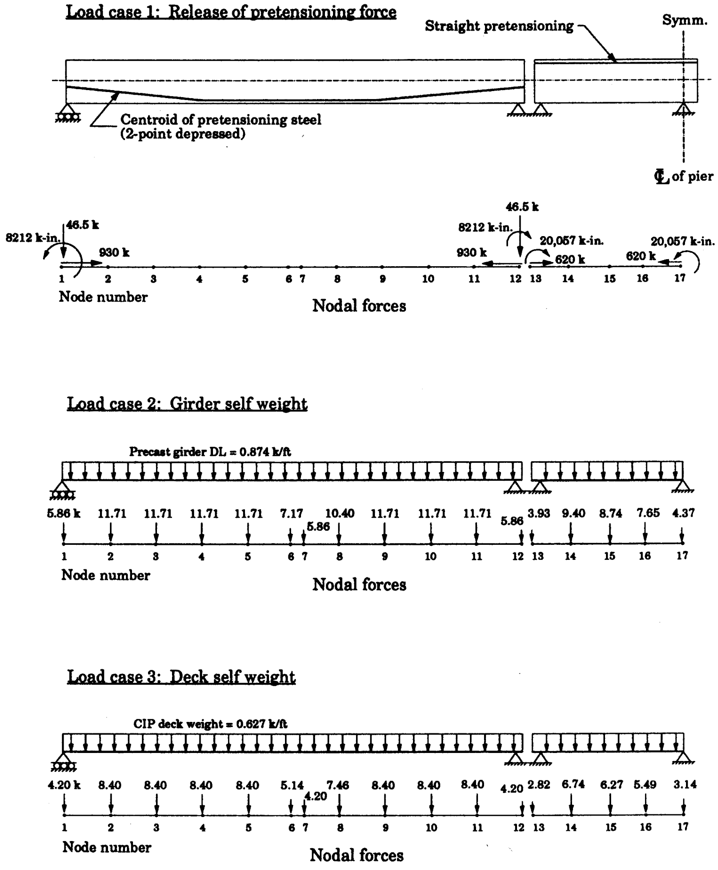


Fig. 10(a) — Load cases

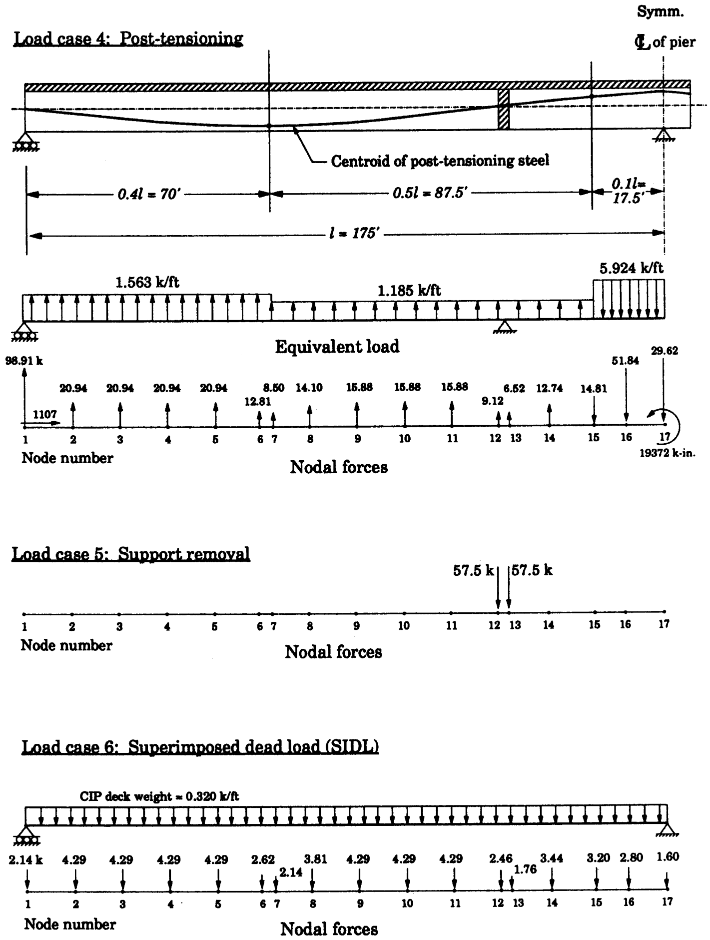


Fig. 10(b) — Load cases (continued)

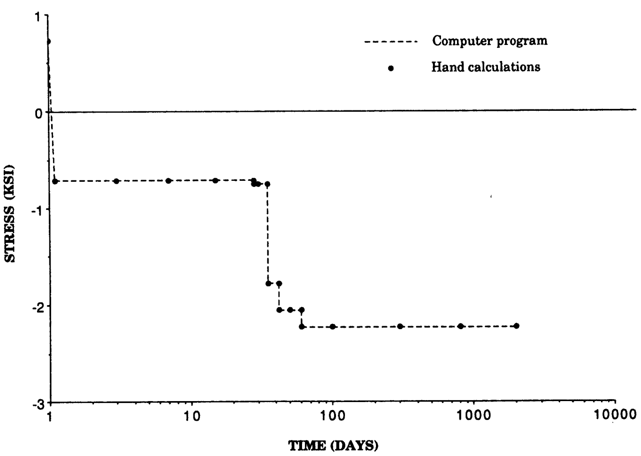


Fig. 11 — Stress history at Point B, 0.4 L *from the abutment (top fibers)*

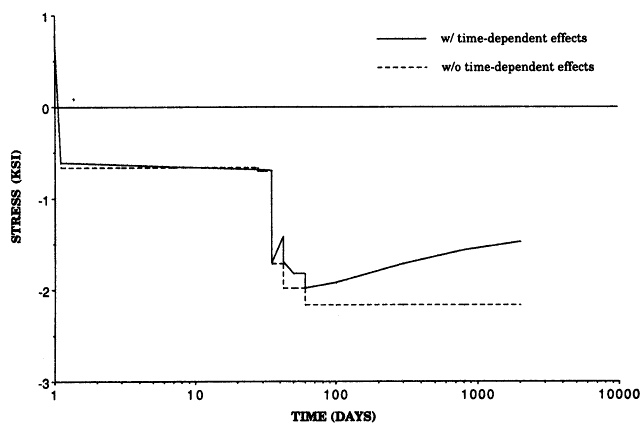


Fig. 12 — Stress history at Point B, 0.4 L *from the abutment (top fibers)*

Major factors to be considered in selecting the segment size are the hauling limitations and the type of lifting equipment available. Selection of the construction method and type of splice will depend on the terrain, the available equipment, and the experience of the local constructors. Experience has shown that bridges designed and constructed with spliced girders are economical and price-competitive for spans up to 280 ft (86 m).

CONVERSION FACTORS

1 in. = 25.4 mm
1 in.^{2} = 6.45 cm^2

1 kip = 4.45 kN
1 ksi = 6.895 MPa
1 lb - ft = 1.356 N - m
$t_F = 1.8\,t_c + 32$

NOTATION

A = cross-sectional area
$C(i + ½, j)$ = creep coefficient = ratio of creep strain at end of interval i to instantaneous strain due to load applied at middle of interval j
c = neutral axis depth from extreme compression fiber in cracked section
d = distance from top fiber to centroid of steel
E = modulus of elasticity
E_{ce} = age-adjusted modulus of elasticity of concrete
f = normal stress
fc' = compressive strength
f_r = reduced relaxation of prestressed steel
f_{ro} = intrinsic relaxation of prestressed steel
f_{ru} = modulus of rupture
f_y = yield strength of steel
I = moment of inertia
i, j = time interval number or time at middle of interval i and j; i - ½ and i + ½ refer to the beginning and end of interval i, respectively
m = total number of intervals
n = modular ratio = ratio of modulus of elasticity of steel to concrete
P = prestress force
t = time
t_o, t_1 = time or age at which load is introduced
Δ = increment or decrement; positive indicates increment
ε = normal strain
ε' = initial strain
ϕ = curvature. When used with the CEB-FIP recommendation, it indicates creep coefficient and its correction coefficients
t = intermediate time

Subscripts

bot (or 1) = bottom fiber of concrete section of each stage
c = concrete
com = composite area properties, transformed to the first cast concrete
ns = nonprestressed steel
ps = prestressed steel
ref = values at reference line
res = restraining forces due to initial strain
s = steel
sh = shrinkage
top = top fiber of concrete section of each stage
∞ = time infinity

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