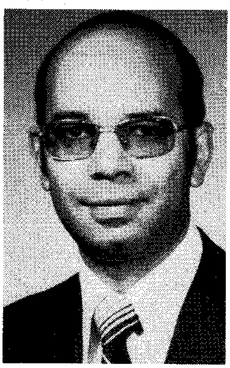
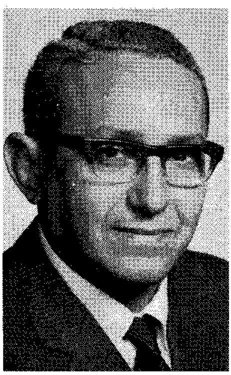
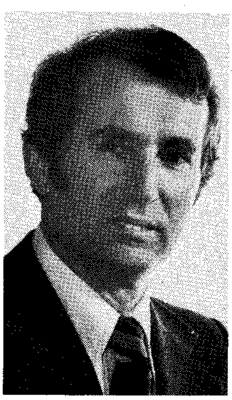
Long-Term Stresses and Deformation of Segmental Bridges



Maher K. Tadros
Associate Professor of Civil Engineering
The University of Nebraska at Omaha
Omaha, Nebraska



Amin Ghali
Professor of Civil Engineering
The University of Calgary
Calgary, Alberta



Walter H. Dilger
Professor of Civil Engineering
The University of Calgary
Calgary, Alberta

A step-by-step computer method is presented for predicting the deflections and stress distribution due to creep and shrinkage of concrete and relaxation of prestressed steel.

Reference is made to an available computer program which can be used for this purpose. The flow chart in Appendix II summarizes the analytical steps.

The computer program accounts for the presence of the non-prestressed steel, the difference in ages of the concrete segments, the multiple stages in which the external loads and prestressing are applied, and the changes in geometry and support conditions as the construction progresses.

The proposed method is applied to a three-span continuous bridge with the results presented to indicate the significance of the time-dependent effects.

In the segmental method of bridge construction,[1-4] units (or segments) of the bridge are precast, assembled in appropriate position and tied together by prestressing to form the bridge superstructure. Segments can also be cast in place.

Several methods are used for erecting segmental bridges.[5-7] One common method suitable for medium to long span bridges is called the "Cantilever Construction." In this method, segments are placed progressively to form a balanced cantilever, starting generally from the piers.

When the cantilevers from two adjacent piers meet, they are joined together to form a continuous span. Continuity prestressing cables are often used to resist bending moments caused by superimposed loads and those developed gradually with time as a result of creep and shrinkage of concrete and relaxation of prestressed steel.

While the erection of a cantilever is in progress, it continues to deflect with time; so do parts of the bridge which have been previously erected. When the cantilever is connected with the rest of the structure, there may be a significant difference between the levels of the ends to be joined. This difference can be eliminated or reduced by jacking or by constructing predetermined camber during the casting of the segments.

The prestressing forces, which are introduced in consecutive stages, will be subjected to gradual change due to creep and shrinkage of concrete and stress relaxation of prestressed steel. The analysis of the prestress loss and the associated deformations becomes more involved when the concrete segments are of different ages and when the segments are joined to form a statically indeterminate structure. Time-dependent statically indeterminate forces gradually develop, causing

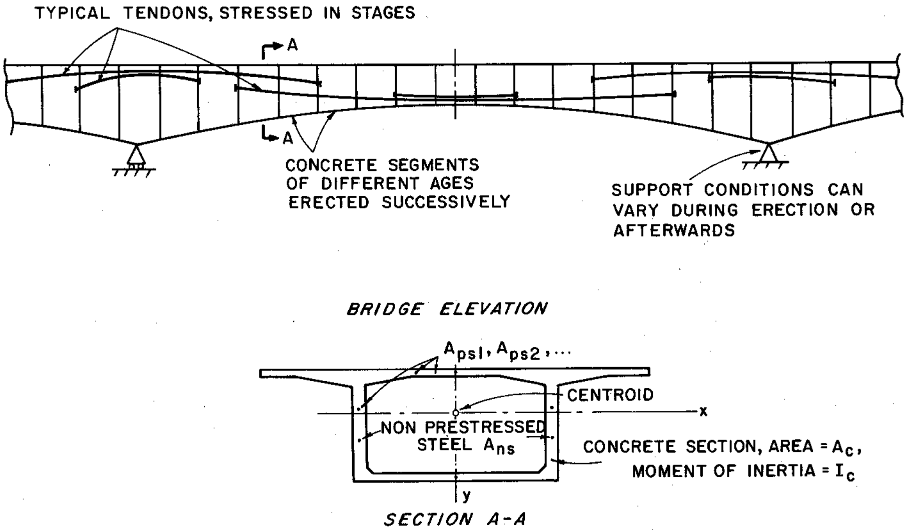


Fig. 1. Longitudinal view and section of typical segmental bridge.

further stress redistribution in the bridge. If these are not carefully accounted for, cracking and other signs of distress may appear in the first few years after construction.

When the segments are cast-in-place, concrete is loaded when it is relatively young. Also, a longer construction time is usually required. Time-dependent effects are therefore more significant.

Time-dependent effects in statically determinate structures prestressed in stages were studied by Danon and Gamble[8] and by Ghali, *et al.*[9] Stresses and deformations for each cross section were considered separately. Deflection was obtained by numerical integration of the curvatures at various cross sections along the member.

Thenoz[10] used the force method of analysis to investigate the effect of creep only on the statically indeterminate forces which develop when two concrete cantilevers are joined to form a single span with built-in ends. The concrete had the same age throughout and the effect of the presence of steel was not considered.

The objective of this paper is to present a method for evaluating the time development of the displacements and the stresses in concrete and steel at any cross section in a statically indeterminate plane frame due to the effects of creep and shrinkage of concrete and stress relaxation of steel. The frame is assumed to be composed of segments of different ages reinforced with non-prestressed steel and with multistage prestressing (see Fig. 1).

The method is directly applicable, but not limited, to bridges built by the cantilever erection method. The loads considered are self weight, prestress, and superimposed sustained loads. Variation of the support conditions during construction or afterwards is also considered. Data related to material and geometric properties, and construction schedule are assumed to be available.

The method of analysis assumes that concrete creep is proportional to stress, and thus superposition of stresses and strains is assumed valid. The stiffness (displacement) method is adopted for the determination of the stress and displacement increments of the structure in a step-by-step procedure.

General Description of Proposed Method

A step-by-step computer procedure is adopted. The same method was developed in an earlier paper[11] for continuous frames with composite cross sections. The theoretical derivations will not be repeated here. Only the differences in applying it to segmental construction as well as the basic principles and assumptions will be discussed.

The time for which the bridge is analyzed, usually an extensive period, is divided into discrete intervals. The stress increments in each interval are assumed to occur at the middle of the interval. The strain increments in an interval are calculated in terms of the stresses applied in that interval and in preceding ones. Instantaneous applied loads such as the prestressing or the self weight are assumed to occur at the middle of an interval of length zero.

The bridge superstructure is idealized as an assemblage of linear prismatic elements connected at nodes. The nodes are on the centroidal axis of the cross section. For each interval, the usual displacement (stiffness) method of structural analysis is used to calculate the increments of displacements and stresses.

Constitutive Relations

Three different materials are involved: (1) concrete; (2) prestressing steel; and (3) non-prestressed reinforcing bars. Because of creep, the first two materials have a time-dependent stress-strain relation. Non-prestressed steel is considered to be a linear elastic material obeying Hooke's law.

For stresses within the service range, the instantaneous strain and creep of concrete are assumed to be linearly proportional to the applied stress. Thus, a stress increment Δf_c, applied and sustained for a period of time, produces a strain which equals $(\Delta f_c/E_c)\,(1 + v)$; where E_c is the modulus of elasticity of concrete and v a creep coefficient equal to the ratio of creep at the end of the period considered to the instantaneous strain.

The value of E_c is considered a function of the age of concrete at the time of application of stress increment, while the creep coefficient v is a function of both the age at loading and the duration for which the stress is sustained.

If the time is divided into intervals during which the stress changes and also shrinkage occurs freely, the total concrete strain (instantaneous plus creep plus shrinkage) occurring between the beginning of the first interval and the end of any interval i is:

$$\epsilon_c\,(i + \tfrac{1}{2}, 0) = \left\{ \sum_{j=1}^{i} \frac{\Delta f_c(j)}{E_c\,(j)}\left[1 + v(i + \tfrac{1}{2}, j) \right] \right\} + s(i + \tfrac{1}{2}, 0) \qquad (1)$$

where i and j refer to the time at the middle of the ith and jth intervals; $[i + (\tfrac{1}{2})]$ refers to the time at the end of the ith interval; and 0 refers to the time at the beginning of the first interval, in which the first stress increment is applied to the concrete.

Thus, $\Delta f_c(j)$ = concrete stress increment introduced at the middle of the jth interval; $E_c(j)$ = modulus of elasticity of concrete at the middle of the jth interval; $s\,[i + (\tfrac{1}{2}), 0]$ = the free shrinkage of concrete between the beginning of the first interval and the end of the ith interval; and $v[i + (1/2), j]$ = creep coefficient = ratio of creep at the end of interval i to the instantaneous strain caused by a sustained stress introduced at the middle of interval j.

Eq. (1) is a simple superposition of strain caused by stress increments which are assumed to occur at the middle of the time intervals.

The strain change in prestressed steel occurring between the beginning of the first interval and the end of the ith interval is:

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

where E_{ps} is the modulus of elasticity of prestressed steel; $\Delta f_{ps}(j)$ is a change in its stress occurring during the middle of the jth interval; $\Delta f_r(j)$ is a "reduced" value of the stress relaxation during the same period. This last term is included because the relaxation causes a part of the tension to be lost without a corresponding change in strain.

In Eq. (2) a "reduced" relaxation value is used as compared with the "intrinsic" relaxation Δf_{ro} that would occur in a test in which the tendon is stretched between two fixed points. The reduction is due to the fact that in a prestressed concrete member, creep and shrinkage of concrete cause a continual shortening of the tendons. Their relaxation, therefore, occurs under the effect of lower tensile stresses than those encountered in constant length tendons.

Empirical equations based on laboratory tests, expressing the intrinsic relaxation f_{ro} in terms of time and initial stress, f_{pso}, are available.[12,13] Increment of reduced relaxation, $\Delta f_r(i)$ in interval i can be calculated in terms of time, initial stress, f_{pso} and tendon shortening in preceding intervals.[11]

In segmental construction, prestressing is usually introduced in stages. In the same cross section, cables tensioned at different stages exhibit different relaxation losses at any specific time. This is accounted for in the present method by considering each set of cables tensioned at one stage as a separate component.

The non-prestressed steel undergoes no relaxation, thus its strain is given by an equation similar to Eq. (2) but with the last term omitted:

$$\epsilon_{ns}(i + 1/2, 0) = \frac{1}{E_{ns}} \sum_{j=1}^{i} \left[\Delta f_{ns}(j) \right] \quad (3)$$

The subscripts ns refer to non-prestressed steel.

Step-by-Step Procedure

The equations of the previous section are used to calculate the incremental strain at any fiber of the bridge cross section. During the ith interval, the change in concrete strain is the difference of strain values calculated by Eq. (1) at the end of the intervals i and $i-1$.

$$\Delta\epsilon_c(i) = \frac{\Delta f_c(i)}{E_c(i)} \left[1 + v(i + 1/2, i) \right] + \left\{ \sum_{j=1}^{i-1} \frac{\Delta f_c(j)}{E_c(j)} \left[v(i + 1/2, j) - v(i - 1/2, j) \right] \right\} + \Delta s(i) \quad (4)$$

The first term in this equation is the instantaneous strain plus creep caused by the incremental stress $\Delta f_c(i)$ occurring between the instant when it is introduced (which is the middle of the ith interval) and the end of the same interval. The term inside the braces sums up the effect of creep during the ith interval due to stresses introduced in preceding intervals. The third term is the free shrinkage in interval i. Eq. (4) can be rewritten in the form:

$$\Delta\epsilon_c (i) = \frac{\Delta f_c (i)}{E_{ce} (i)} + \Delta\bar{\epsilon}_c(i) \qquad (5)$$

where $\Delta\bar{\epsilon}_c$ is equal to the sum of the second and third terms in Eq. (4); its physical meaning is discussed above. The value $E_{ce}(i)$ represents an "effective" modulus of elasticity for concrete in interval i:

$$E_{ce} (i) = \frac{E_c (i)}{1 + v\,(i + ½, i)} \qquad (6)$$

By a similar treatment using the constitutive relation for prestressed steel [see Eq. (2)] the incremental strain occurring during the ith interval can be expressed as follows:

$$\Delta\epsilon_{ps} (i) = \frac{\Delta f_{ps} (i)}{E_{ps}} + \Delta\bar{\epsilon}_{ps} (i) \qquad (7)$$

where

$$\Delta\bar{\epsilon}_{ps} (i) = -\frac{\Delta f_r (i)}{E_{ps}} \qquad (8)$$

The increment in strain in the nonprestressed steel occurring during the ith interval [see Eq. (3)] is:

$$\Delta\epsilon_{ns} (i) = \frac{\Delta f_{ns} (i)}{E_{ns}} \qquad (9)$$

The constitutive relations for the three materials involved are in a linear form [Eq. (9)] or a pseudolinear form [Eqs. (5) and (7)]. The strain $\Delta\bar{\epsilon}(i)$ in each of Eqs. (5) and (7) represents "initial" deformation independent of the stress increment introduced in interval i, and thus its value can be determined if the stress increments in preceding intervals are known [see the last two terms in Eq. (4)].

In the step-by-step method, a complete analysis of the structure is done in each interval. When the analysis is done for interval i, the stress increments in all preceding intervals would have been determined in earlier steps.

Thus, the initial strains $\Delta\bar{\epsilon}(i)$ are known quantities, which can be treated in the same way as if they were produced by a change of temperature of known magnitude. The term "initial," often used in conjunction with the stress analysis for the effect of temperature variation, should not be confused with the instantaneous elastic deformation.

From the above discussion, it can be seen that for each time interval a linear elastic analysis is performed for the bridge structure, which is assumed to have a composite cross section made up of the three materials: concrete, prestressed steel and nonprestressed steel.

For concrete, an effective modulus of elasticity is used, with a value varying in each interval according to Eq. (6); for the two kinds of steel the constant values E_{ps} and E_{ns} are employed in all intervals. In any interval i, the concrete and the prestressed steel are considered as though subjected to a change in temperature producing initial (free) strains $\Delta\bar{\epsilon}_c(i)$ and $\Delta\bar{\epsilon}_{ps}(i)$ of known magnitude.

The stress increment $\Delta f(i)$ in each of the three materials is an unknown to be found by the analysis for interval i; it represents the stress produced by the external loads (if any) applied at the middle of the interval plus the stress necessary to restore compatibility of strain in the three materials forming the composite cross section. This compatibility has been temporarily violated by the introduction of the initial strain $\Delta\bar{\epsilon}(i)$.

In the analysis, plane cross sections of the bridge are assumed to remain plane during deformation. This implies that the bending theory of shallow beams is applicable. Brown and Burns[14] in their analysis of instantaneous stresses and deformations of the Corpus Christi segmental bridge, using a "finite segment" method, have confirmed the applicability of this assumption.

Further, compatibility of strain is assumed in the concrete and adjacent steel at all fibers and at all sections; that is, the strain increment in a given time interval is equal in both concrete and adjacent steel. This second assumption is not always valid during construction of the bridge. Grouting of the prestressing ducts in post-tensioned segmental bridges is often postponed until the end of some or all of the prestressing stages. During the period the tendons are left unbonded, the time-dependent stress changes should be "averaged out" through the entire length of the tendon.

The proposed method can be extended to account for this by numerically integrating the prestressing steel strain increments along the tendon length and adjusting the concrete and steel stress accordingly. However, it is believed that this further iteration within each time-interval would result in a considerable increase in computation cost without significantly improving the overall stress and deformation-time relationships. Further research would help evaluate this hypothesis.

Computer Implementation

A computer program based on the above theory is described by the flow chart in Appendix II. It is concerned with structures considered as plane frames. The strain, and hence the stress at any fiber, are determined from three nodal displacements: translation components in two orthogonal directions plus a rotation at one point of the cross section (usually chosen as the centroid of the section). The structure may have curved members of variable cross section, but in the analysis they are approximated as an assemblage of straight prismatic elements with average properties of the two end sections of each element.

The prestressing forces are represented by a set of self-equilibrating nodal forces. Their values are the initial forces including friction losses at transfer. Losses due to creep, shrinkage and relaxation of steel are automatically accounted for in the procedure described above.

As mentioned earlier, linear elastic analysis is performed for each time interval; the usual displacement (stiffness) method of analysis is employed. The analysis gives increments of displacements and stresses occurring in the interval. These are to be added to the increments in all the preceding intervals to give the total values. The analysis is thus done step by step until the end of the last time interval.

Because the analysis of the structure covers a period that includes its construction stages, the length, number of nodes and boundary conditions of the structure can be different for different time intervals. In the choice of the time intervals, their limits should coincide with the events of application of external loads including prestressing and of the change in boundary conditions. External applied loads are considered to be introduced at the middle of time intervals of zero duration. This is further clarified by the example in the section "Application."

Additional details of the method of analysis are given in Reference 11. A listing of the computer program and sufficient instructions for its use are given in Reference 15. In this program, the time variation of the concrete modulus of elasticity E_c, the creep coefficient v, the shrinkage s, and the intrinsic relaxation of prestressed steel are assumed according to equations listed in Appendix 1 of Reference 11. If, however, the analyst would like to use different equations, they can be accommodated in the program by changing the corresponding

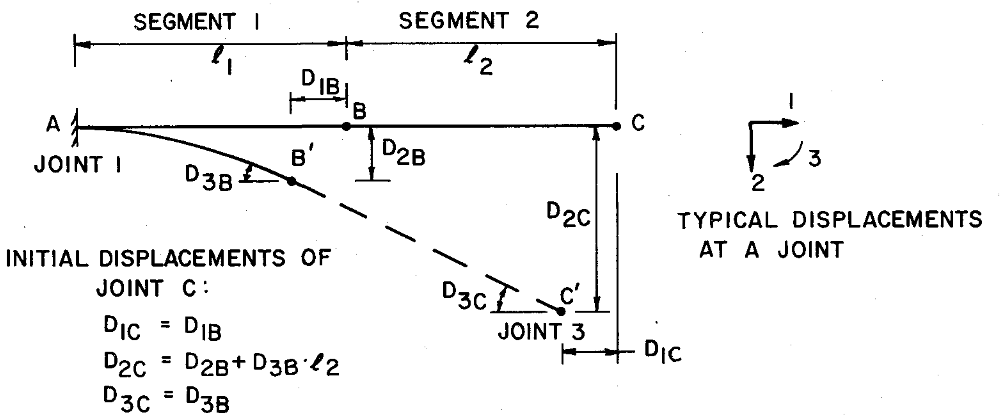


Fig. 2. Joint displacements before application of loads of Segment 2 in match-cast cantilever construction method.

statements; the details of how this can be done are given in Reference 15. In this way, the program can be used with any chosen time functions.

Deflections During Cantilever Erection

Segmental construction is distinguished by the type of joint between the segments. The two basic types of joints used are the wide cast-in-place joints and the match-cast joints.[7] With the wide joints, curving (shaping) of the structure may be obtained within the joint. Match-cast joints are more commonly used. Perfect fit of adjacent elements is achieved by casting each segment against its neighbor. In this type, the required shape of the superstructure has to be built into the segments during the casting procedure. Match-cast jointing will be considered in the following discussion.

The step-by-step method described here gives increments of displacements occurring at each stage of construction. Fig. 2 shows the horizontal and vertical translations and the rotation of Joint B just before erecting Segment 2. The displacement increments in Stage 2, caused by the new prestressing and the self weight of Segment 2, are to be measured from a displaced "datum" AB′C′ to give the total displacements measured from the horizontal ABC.

Fig. 3a shows the time-dependent deflection of a cantilever built in four stages. For clarity, the deflected shape of each segment is represented by a straight line and no horizontal displacements are shown. At the end of Stage 4, and just before connecting the cantilever with the remainder of the structure, the deflected shape of the cantilever is given by the line AB′C′D′E′.

A method to eliminate or reduce the discontinuity (relative rotation and relative deflection) of the two meeting ends of adjacent cantilevers is to build camber into the segments. To eliminate the deflection of the cantilever at the end of Stage 4 (Fig. 3b), the camber to be built into each segment, is equal and opposite to the deflection measured from the tangent to its joint with the preceding segment. It is therefore very important to accurately predict deformation of the structure at various stages of construction.

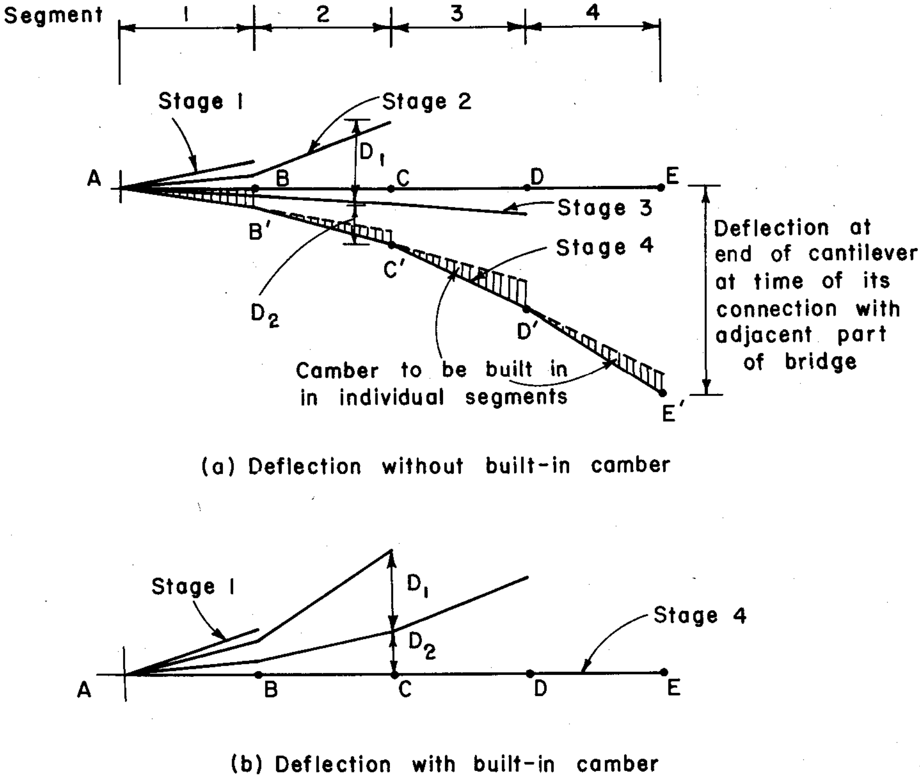


Fig. 3. Deflection of a four-segment cantilever.

Application

The method of analysis is applied to the three-span symmetrical bridge shown in Figs. 4 and 5. The erection of the bridge starts at Pier 1 where four segments are successively placed on each side of the pier, and tied together by post-tensioning in four stages. While the cantilever erection is in progress, the cantilever is temporarily fixed to the pier to produce a stable system in which unbalanced cantilever moments, if any, can be transferred to the pier.

Span 1 is completed at Stage 5 by assembling the remaining three segments, while supported on falsework, and then stressing the bottom cables in the span. The rotational restraint at Pier 1 is then released. The process is repeated for the other half of the bridge starting from Pier 2 at Stages 6 through 10.

The bridge construction is completed by dropping in the central segment and stressing the continuity cables in Span 2 while allowing horizontal movement at Pier 2.

The area and eccentricity of the prestressing steel at the various construction stages are given in Fig. 4. Other dimensions and concrete section properties are shown in Fig. 5. Non-prestressed steel is uniformly distributed in the cross section.

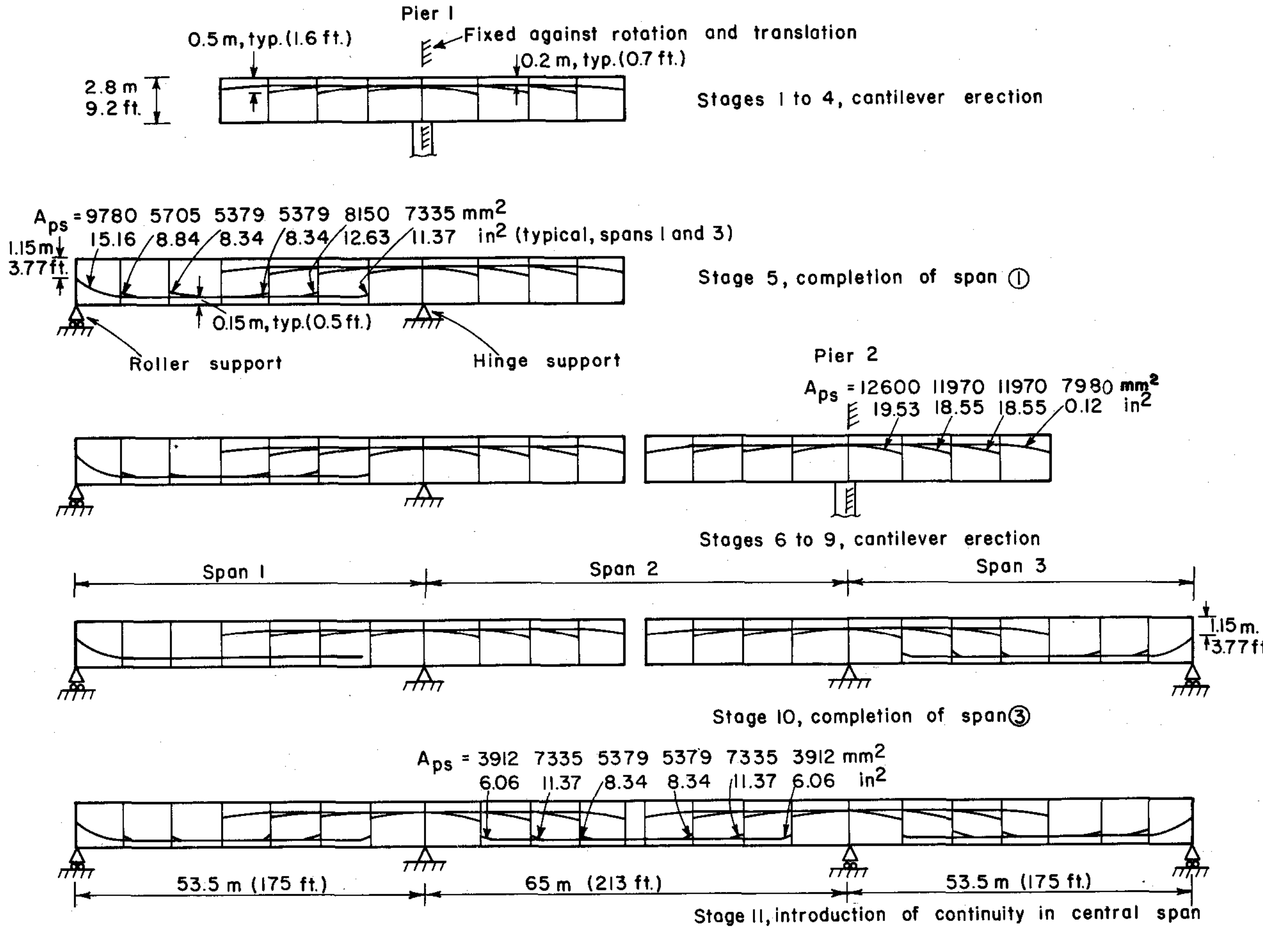
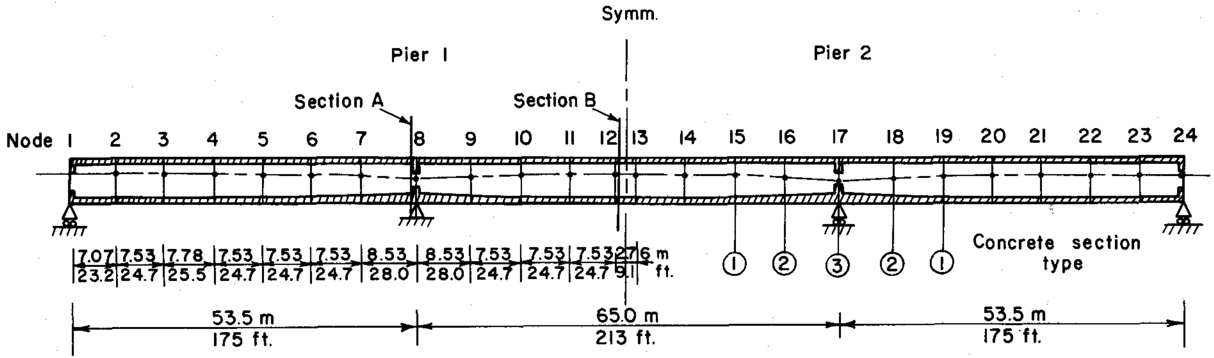
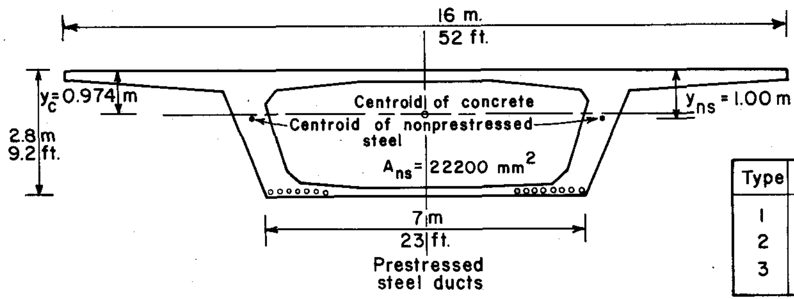


Fig. 4. Construction sequence of bridge example.



(a) Longitudinal section



(b) Cross section

Type	A_c m^2	y_c m	I_c m^4
1	10.035	0.974	10.058
2	11.055	1.093	12.027
3	12.075	1.340	14.942

(c) Concrete section properties
(1 m = 3.28 ft.; 1 m^2 = 10.7 ft^2; 1 m^4 = 116 ft^4)

Fig. 5. Dimensions of bridge example.

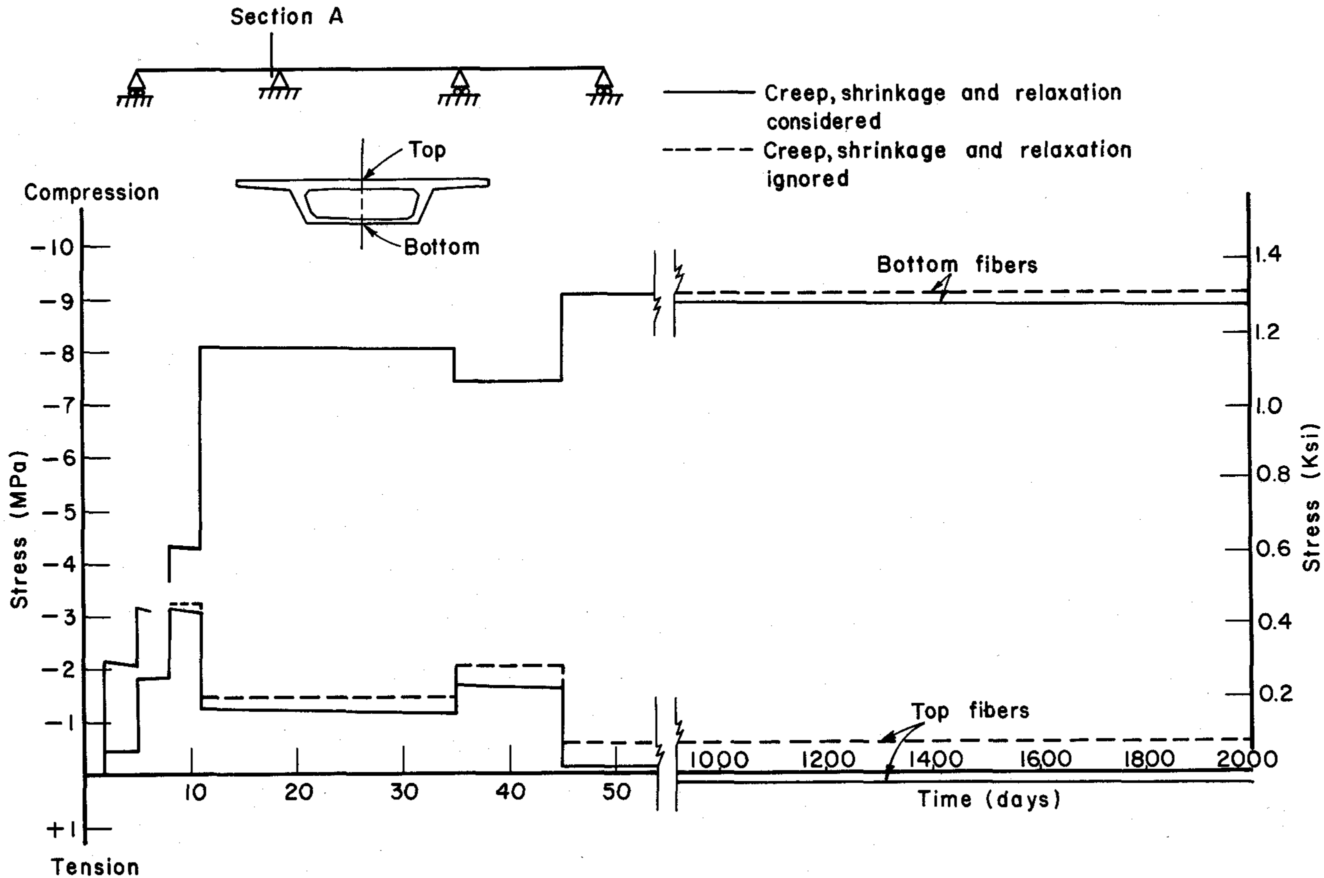


Fig. 6. Concrete stress in Section A, near Pier 1.

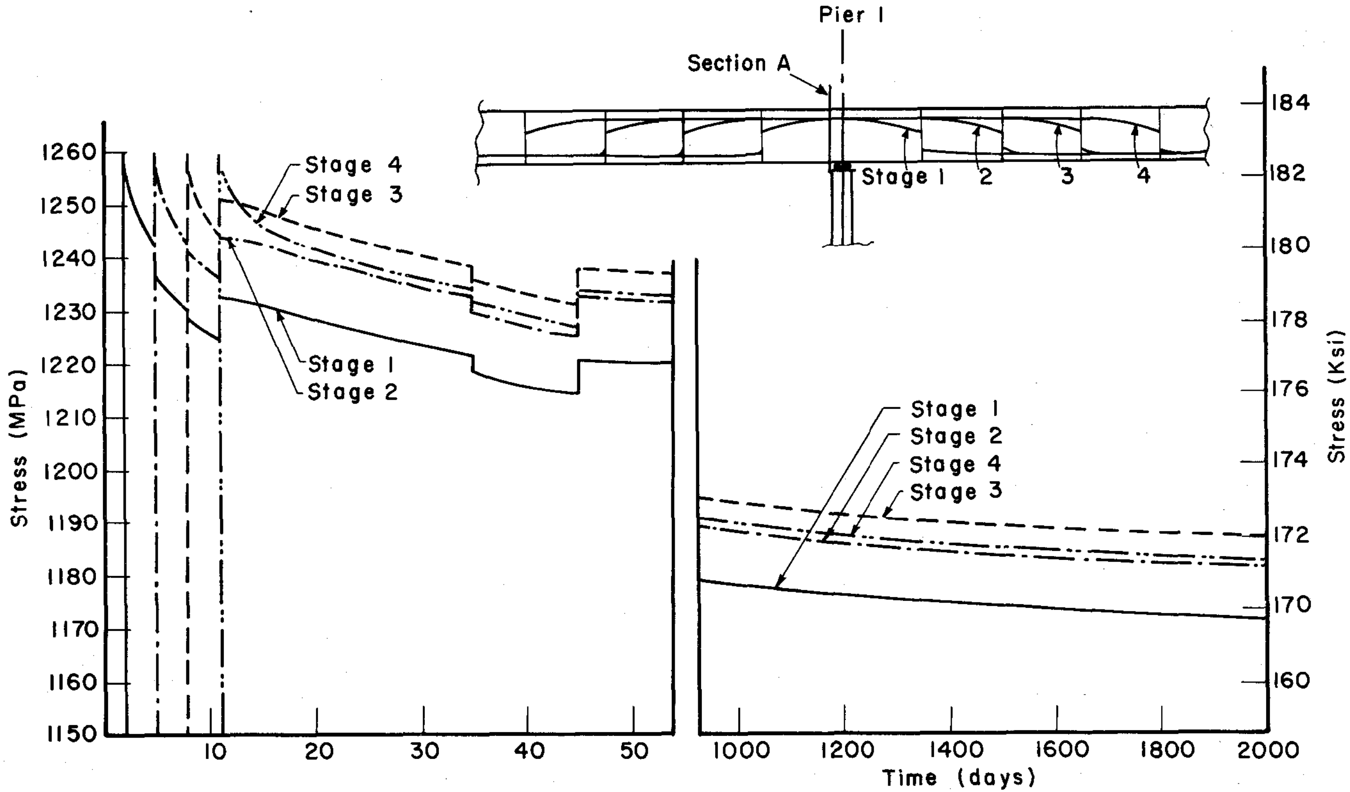


Fig. 7. Stress in the prestressing steel at Section A, near Pier 1.

Material Properties

Concrete:
Weight = 20 kN/m³ (125 lb/ft³)
E_c (28 days) = 34 GPa (5000 ksi)

Non-prestressed steel:
E_{ns} = 196 GPa (28,000 ksi)

Prestressing steel:
E_{ps} = 190 GPa (27,000 ksi)
f_{psu} = 1.77 GPa (256 ksi)
f_{psy} = 1.50 GPa (218 ksi)
f_{pso} = 1.24 GPa (179 ksi) and intrinsic relaxation at time infinity
f_{ro} = 31 MPa (4.5 ksi)

Segments are assumed to be cured for 3 days ($t_d = 3$) and then kept in storage until they are transported and erected at the age of 28 days;

Creep coefficient[16,17] $v_N = 2.0$ and shrinkage $s(t_\infty, 28 \text{ days}) = 300 \times 10^{-6}$.

The time variation of E_c, v, s and f_{ro} are assumed to follow the equations given in the Appendix of Reference 11.

Time Intervals

29 time intervals are chosen; the interval limits are (days): 2, 2, 5, 5, 8, 8, 11, 11, 15, 15, 19, 19, 22, 22, 25, 25, 28, 28, 32, 32, 35, 35, 45, 45, 60, 100, 200, 500, 1000, and 2000.

Intervals with a zero duration are those at the "middle" of which "sudden" loads are applied or support conditions changed.

Loading

Loads representing prestress and self weight are applied in intervals 1, 3, 5, 7, 9, 11, 13, 15, 17, 19 and 21 which correspond to the eleven construction stages. In Interval 23, superimposed dead load of intensity 45kN/m (3.1 kips/ft) is applied.

Support Conditions

Support conditions are changed in Intervals 1, 9, 11, 19 and 21; that is, in Stages 1, 5, 6, 10 and 11 (see Fig. 4).

Discussion of Results

A computer program[15] based on the method presented in this paper was used to give, for selected cross sections of the bridge, the variation with time of stress in concrete, non-prestressed steel and the prestressing steel. The program also gives the time development of the deflection, horizontal displacement and rotation at the nodes numbered in Fig. 5. Figs. 6-11 are plots of some of the results.

Fig. 6 depicts the concrete stresses in Section A, near Pier 1 (see Fig. 5). The vertical lines indicate the instants of load application. The broken lines give the stresses when the time-dependent effects are ignored, that is when $v = s = f_{ro} = 0$. The time-dependent analysis yields tension stresses in the top fibers of the section, while calculations ignoring the time-dependent effects would show compression in the same fibers. Basing the live load capacity on this compression stress is therefore not conservative.

The time-dependent effects on the concrete stress at Section B, near the center of Span 2 (Fig. 9) are more pronounced compared to Section A. Connecting the two ends of the cantilevers at Stage 11, rendering the structure statically indeterminate, results in time-dependent indeterminate forces preventing any further development of differential deflections and rotations of the cantilever ends.

Figs. 7 and 8 give the time-variation of the stress in the multi-stage prestressing steels and in the nonprestressed reinforcement at Section A.

The deflected shape of the bridge at various construction stages is shown in Fig. 10. By accurately predicting deflection, it is possible to build in appropriate camber while casting the segments to eliminate the vertical deflection[18,19] at Nodes 4, 21, 12 (or 13). Fig. 11 demonstrates the significance

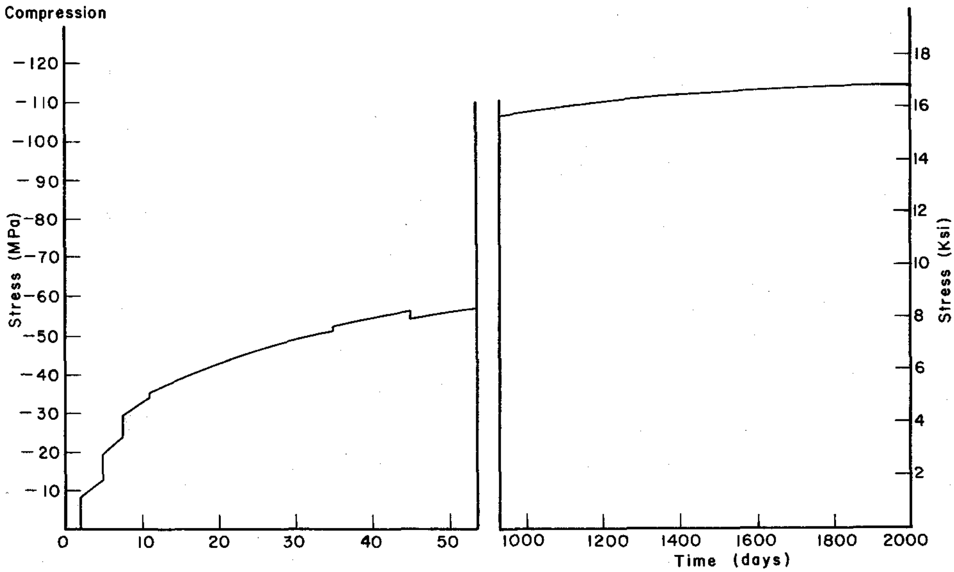


Fig. 8. Stress in the non-prestressed steel at Section A, near Pier 1.

of time-dependent effects on vertical deflection near the center of the inner span.

The values plotted in Fig. 10 are deflections measured from one horizontal datum without any modification which may be done by building in camber in the precast segments, or by initially setting the falsework. For example, at t = 15 days, the deflection at Node 4 is 1.7 mm (0.07 in.); the abrupt change in deflection shown in Fig. 10 can be eliminated in the actual structure by adjusting the level of the falsework supporting the segments at the left of Node 4.

Conclusions

The prediction of deflections and stress redistribution in prestressed concrete segmental construction can be greatly in error unless accurate account is taken of the effects of creep and shrinkage of concrete and the relaxation of prestressed steel.

The presence of the non-prestressed steel should not be ignored, particularly because of its important effect on the deflections. Such an analysis requires lengthy computations, because the loads are applied in several stages and because the structure is composed of segments of different ages.

Furthermore, the structure changes in geometry and support conditions as the construction progresses. The problem is well suited to the step-by-step computer method of solution presented and used in this paper.

The bridge example analyzed is a precast segmental bridge erected by the cantilever method. The results clearly demonstrate the significance of

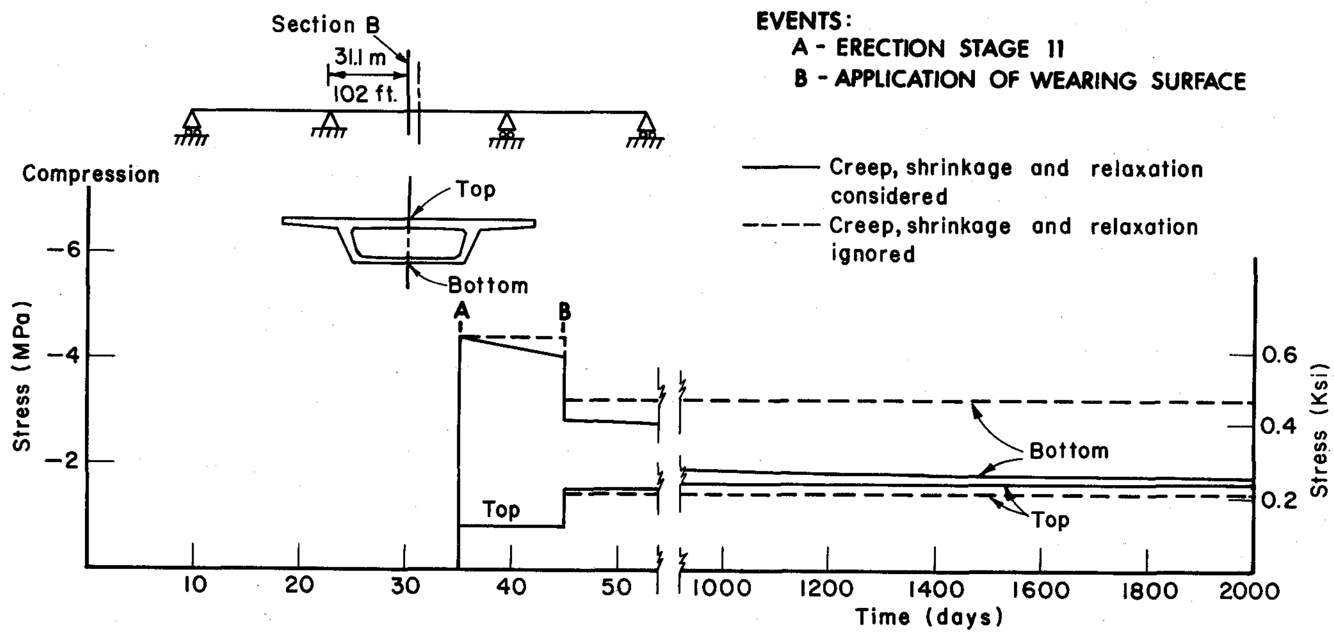


Fig. 9. Concrete stress in Section B near the middle of Span 2.

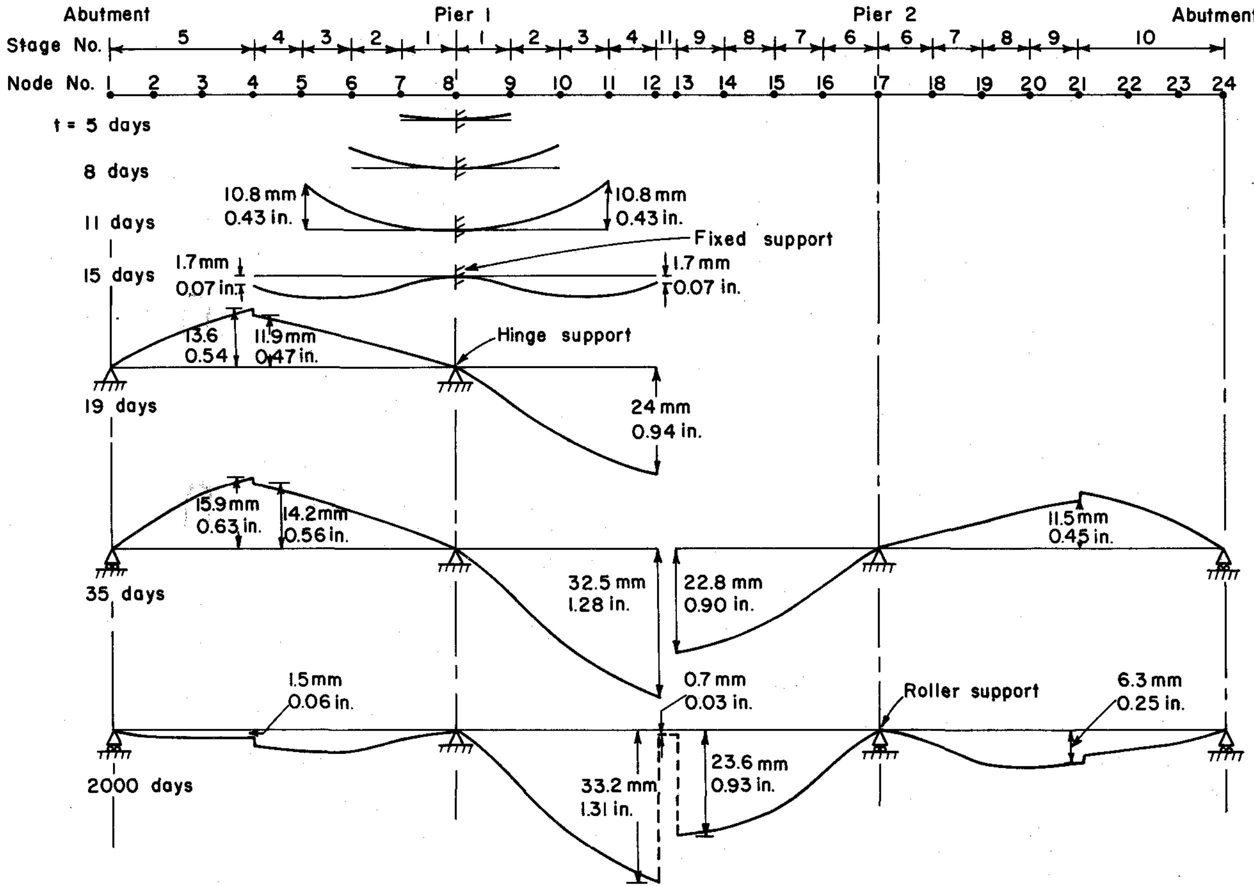


Fig. 10. Vertical deflection of bridge at various construction stages (without any corrections, as for example by built-in camber during casting of segments).

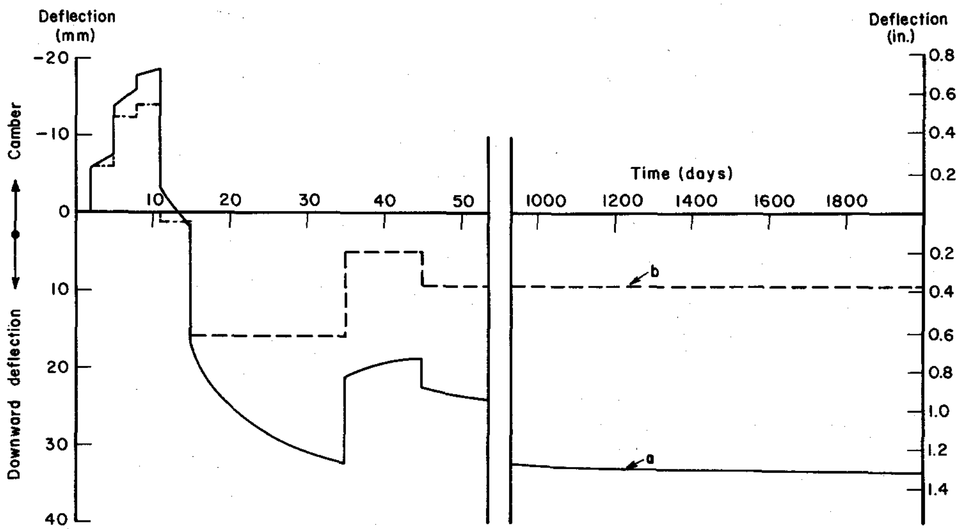


Fig. 11. Deflection of Node 12; (a) Creep, shrinkage and relaxation considered; (b) Creep, shrinkage and relaxation ignored.

the time-dependent effects despite the fact that in precast construction, creep and shrinkage of concrete are considerably less than those encountered when the bridge segments are cast in place.

Considerable misalignment problems can develop due to differential time-dependent deflections at the ends of two joining parts of a bridge. These deflections can be predicted accurately by the method presented. The anticipated misalignment can then be prevented by adjusting the formwork before the segments are cast.

Acknowledgment

This research was financially supported by grants from the National Research Council of Canada, The University of Calgary and West Virginia University.

References

1. Podolny, Jr., Walter, "An Overview of Precast Prestressed Segmental Bridges, PCI JOURNAL, V. 24, No. 1, January-February 1979, pp. 56-87.

2. Ballinger, C. A., and Podolny, W., "Segmental Bridge Construction in Western Europe—Impressions of an IRF Study Team," Proceedings, Conference conducted by Transportation Research Board, National Academy of Sciences, Washington, D.C., TRR No. 665, V. 2, September 1978, pp. 182-191. See also a detailed report by Ballinger, Podolny and Abrahams, International Road Federation, 1023 Washington Building, Washington, D.C., July 1978, 150 pp.

3. Libby, J. R., and Perkins, N. D., *Modern Prestressed Concrete Highway Bridge Superstructures, Design Principles and Construction Methods,* Grantville Publishing Company, San Diego, California, 1976, 254 pp.

4. *Precast Segmental Box Girder Bridge Manual*, Prestressed Concrete Institute, Chicago, 1978, 124 pp.

5. Lacey, G. C., Breen, J. E., and Burns, N. H., "State-of-the-Art for Long Span Prestressed Concrete Bridges of Segmental Construction," PCI JOURNAL, V. 16, No. 5, September-October 1971, pp. 53-77. See also discussion of paper in PCI JOURNAL, V. 17, No. 1, January-February 1972, pp. 104-108.

6. Muller, Jean, "Ten Years of Experience in Precast Segmental Construction," PCI JOURNAL, V. 20, No. 1, January-February 1975, pp. 28-61.

7. PCI Committee on Segmental Construction, "Recommended Practice for Segmental Construction in Prestressed Concrete," PCI JOURNAL, V. 20, No. 2, March-April 1975, pp. 22-41.

8. Danon, J. R., and Gamble, W. L., "Time-Dependent Deformations and Losses in Concrete Bridges Built by the Cantilever Method," Report No. UILU-ENG-77-2002, Civil Engineering Studies, University of Illinois at Urbana-Champaign, Urbana, Illinois, January 1977, 169 pp.

9. Ghali, A., Sisodiya, R. G., and Tadros, G. S., "Displacements and Losses in Multistage Prestressed Members," *Journal of the Structural Division*, ASCE, V. 100, No. ST11, Proceedings Paper 10968, November 1974, pp. 2307-2322.

10. Thenoz, M., "Redistribution des Efforts Par Fluage dans les Ponts Construits par Encorbellement" (Redistribution of Internal Forces Due to Creep in Segmental Construction), 7th FIP Congress, Fédération Internationale de la Précontrainte, Contribution of French Techniques, New York, 1974, pp. 25-27.

11. Tadros, M. K., Ghali, A., and Dilger, W. H., "Time-Dependent Analysis of Composite Frames," *Journal of the Structural Division*, ASCE, V. 103, No. ST4, Proceedings Paper 12893, April 1977, pp. 871-884.

12. Magura, D. D., Sozen, M. A., and Siess, C. P., "A Study of Stress Relaxation in Prestressing Reinforcement," PCI JOURNAL, V. 9, No. 2, April 1964, pp. 13-26.

13. PCI Committee on Prestress Loss, "Recommendations for Estimating Prestress Losses," PCI JOURNAL, V. 20, No. 4, July-August 1975, pp. 43-75.

14. Brown, R. C., Jr., and Burns, N. H., "Computer Analysis of Segmentally Erected Bridges," *Journal of the Structural Division*, ASCE, V. 101, No. ST4, Proceedings Paper 11218, April 1975, pp. 761-778.

15. Tadros, M. K., Ghali, A., and Dilger, W. H., "Computer Program for Analysis of Stresses and Deformation in Segmental Construction, a User's Manual," Research Report, published by Departments of Civil Engineering, the University of Nebraska at Omaha, and the University of Calgary, Canada, 1979.

16. ACI Committee 209, Subcommittee 2, "Prediction of Creep, Shrinkage and Temperature Effects in Concrete Structures," SP-27, American Concrete Institute, Detroit, 1971, pp. 51-93.

17. *International Recommendations for the Calculation and Execution of Concrete Structures*, Comité Européen de Béton-Fédération Internationale de la Précontrainte, June 1970, Cement and Concrete Association, London, England.

18. Mathivat, J., "Récents Développements des Ponts à Voussoirs Prefabriqués" (Recent Developments in Precast Segmental Bridge Construction), International Association for Bridge and Structural Engineering, IABSE Surveys S-2/77, May 1977, 24 pp.

19. Mathivat, J., *Constructions par Encorbellement des Ponts en Béton Précontraint*, Editions Eyrolles, 1979, Paris, France, 340 pp.

APPENDIX I—NOTATION

A = cross-sectional area

E = modulus of elasticity

f = normal stress, positive when tensile; subscripts *pso*, *psy* and *psu* indicate initial, 0.1 offset and ultimate stress in prestressed steel, respectively

I = moment of inertia of section

i,j = time interval number, referred to in step-by-step calculation; when used with time-dependent parameters, they indicate time at middle of interval; $i - ½$ and $i + ½$ refer to beginning and end of interval i, respectively

Δ = used as prefix to indicate incremental value

ϵ = normal strain, positive when corresponds to extension

$s[i + ½, j - ½]$ = shrinkage of concrete during period $\{t[i + ½] - t[j - ½]\}$

t = time, in days, measured from reference date

$v[i + ½, j]$ = creep coefficient, ratio of creep strain at time $t[i + ½]$ to instantaneous strain caused by constant sustained stress applied at $t(j)$; subscript N is used to indicate normal creep, equals value of creep coefficient when $t[i + ½]$ = infinity and t_j = 28 days

Subscripts

c = concrete

e = effective modulus of elasticity [Eq. (6)]

ns = non-prestressed steel

ps = prestressed steel

r = reduced relaxation in shortened prestressed tendon

ro = intrinsic relaxation in constant-length tendon

Superscript $\bar{\ }$ = "initial" value defined in Eqs. (5) and (7)

Additional Notation Used in Flow Chart

DEL = nodal displacement increments

DFC = increment of stress resultant in concrete

DFPS = increment of stress resultant in prestressed steel

DFNS = increment of stress resultant in non-prestressed steel

DFRC = increment of internal force in concrete required to eliminate initial strains

DFRPS = increment of internal force in prestressed steel required to eliminate initial strains

F = nodal forces

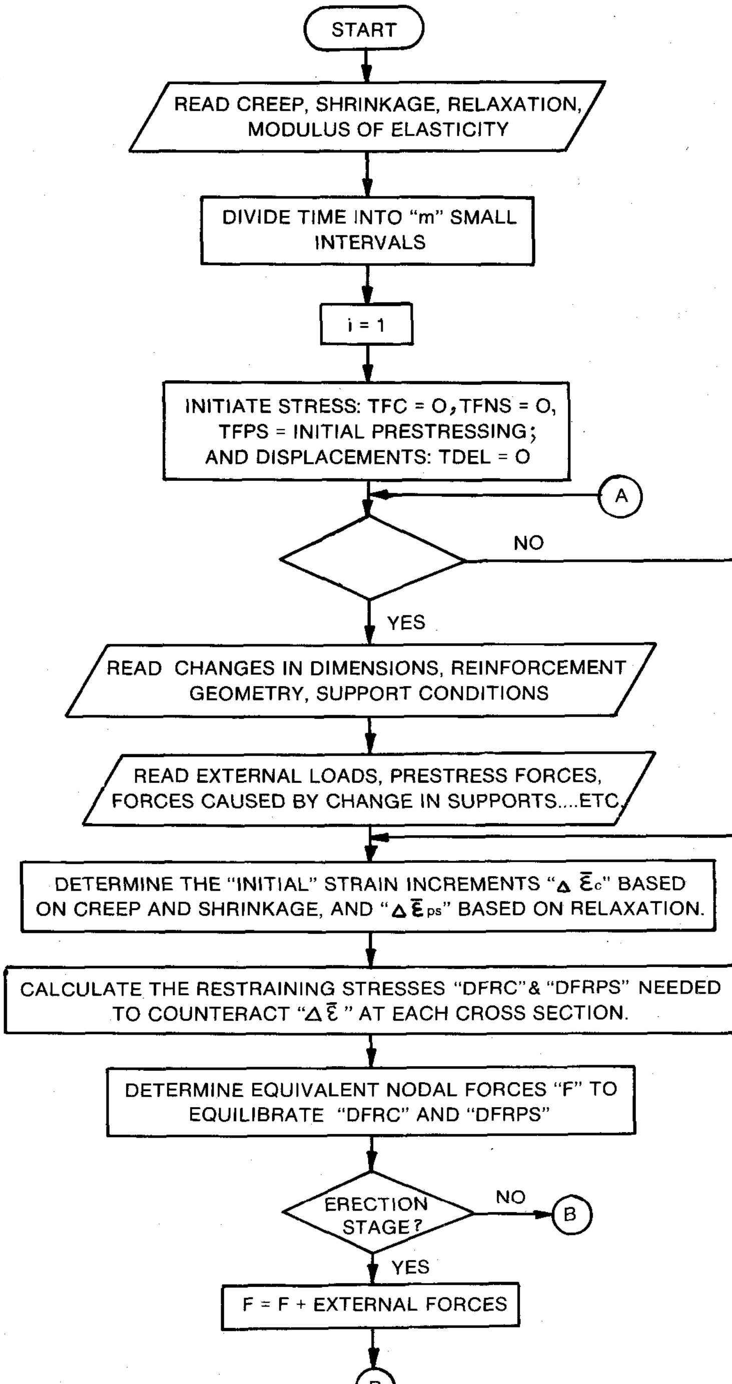
TDEL = total displacements at end of intermediate interval "i"

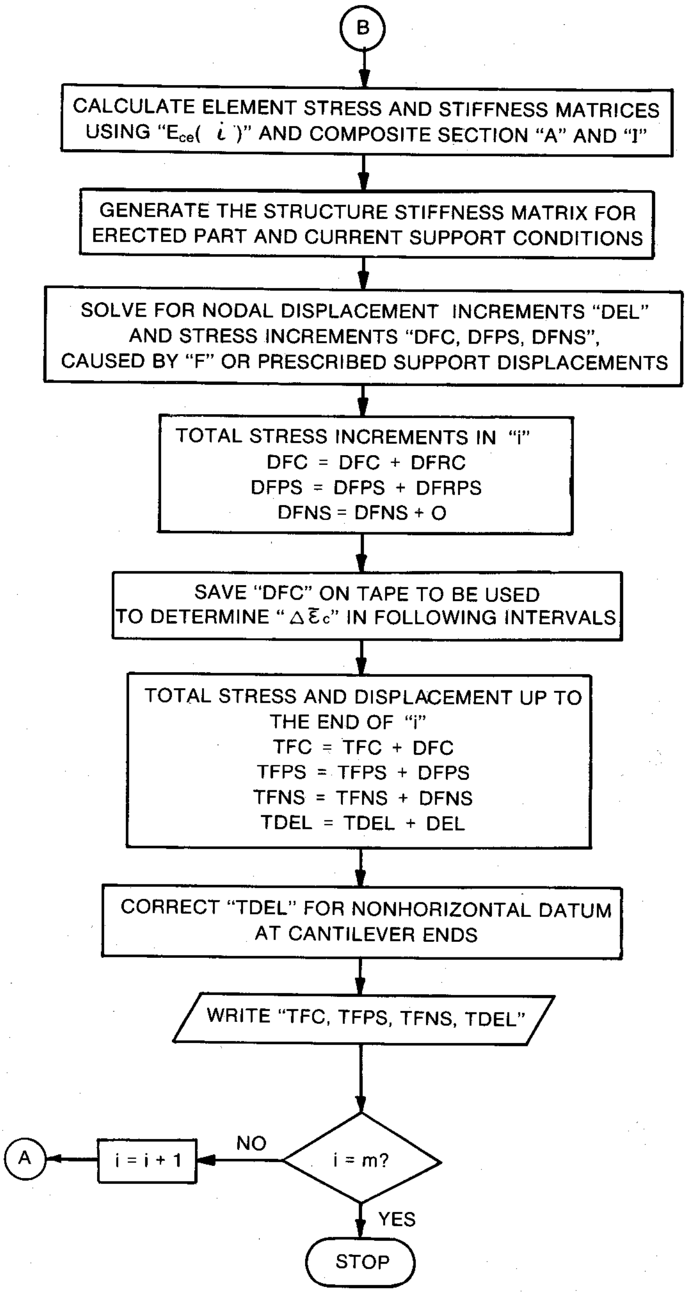
TFC = total stress resultant in concrete at end of interval "i"

TFPS = total stress resultant in prestressed steel at end of interval i

TFNS = total stress resultant in non-prestressed steel at end of interval i

APPENDIX II—FLOW CHART





Discussion of this paper is invited.
Please forward your comments to
PCI Headquarters by Jan. 1, 1980.