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SEGMENTAL ANALYSIS AND DESIGN FUNDAMENTALS
TIME DEPENDENT EFFECTS

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ABSTRACT : Stress redistribution over the life span of segmentally erected prestressed concrete bridges has a significant impact on the long-term deflection and stresses of these structures. Yet due to the complexity of analysis, in many cases they are not effectively accounted for. Material and construction time dependent effects are the source of stress redistribution. The material time dependency consists of creep, shrinkage and aging of concrete, and stress relaxation in prestressing steel. The time difference in the application of dead, live and construction loading necessitated by the sequence of erection forms the construction time dependency. The two categories of time dependency are interrelated. Larger spans, younger concrete at loading, and shorter construction sequence generally result in an intensification of the time dependent effects.

This work presents the basic concepts of the time dependent parameters. It reviews the codes' approach to their quantification, and describes the procedures used in the analysis of segmentally constructed bridges in allowing for the time-dependent effects. The sensitivity of two typical bridge structures, one a precast prestressed girder with concrete topping, and the other a free cantilever construction to the selected values of the time dependent parameters is investigated. It is illustrated that through new analysis tools the effects of the time dependent parameters can now be readily accounted for.

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1 - INTRODUCTION

Time dependent concrete deformation is greatly amplified with early concrete loading coupled with high construction stresses normally exceeding the service level conditions. Control of stresses and deflections for short and long term considerations is achieved through the application of pre-. and post tensioning tendons. As a result, **creep, shrinkage**, and **aging** effects of concrete as well as **relaxation** of prestressing steel must be considered in the design and analysis of segmentally erected prestressed concrete bridges. Significant deviation from an accurate structural response will result if these factors are neglected or inaccurately predicted.

2 - TIME DEPENDENT COMPONENTS

Concrete time dependent behavior may be classified into three distinct categories namely creep, shrinkage and aging. Creep is a stress originated effect and is dependent, among other factors, on the entire load history of the concrete in place. Shrinkage is a non-stress originated factor. It is due primarily to reduction in water content and change in volume caused by carbonation. As concrete ages, its modulus of elasticity increases. Addition, or deletion of the same magnitude of loading at different ages results in different deformation in concrete. The impact of the creep component is by far the most complex to predict, since it is a function of the entire load history of the structure.

Prestressing time dependent behavior is due entirely to the relaxation phenomenon similar in nature to the creep component of concrete. The relaxation of prestressing steel is defined as the decrease in tendon force with time under constant deformation.

Creep, shrinkage and aging of concrete and relaxation of prestressing steel may be classified as time dependent material inherent parameters. The other class of time dependent parameters relates to construction scheduling and configuration of the structure. Segmentally erected prestressed concrete bridges generally follow a repetitive cycle in construction operations. The shorter is the cycle period, the younger is the concrete age at loading or prestressing, the higher will be the concrete time dependent effects due to creep, shrinkage and aging. This in turn, would lead to a higher degree of stress redistribution. Similarly, different designs, such as precast versus cant-in-place free cantilever construction, impact the behavior and response of the structure in the same fashion. Composite construction in segmental erection adds to the complexity in prediction of the structural response.

An account of the time dependent parameters and their allowance in design is given in Aalami (1993).

2.1 Aging of Concrete

Concrete strength and consequently its elastic modulus which governs the stiffness and the deflection of a structure are monotonically increasing parameters with time. As concrete ages, the rate of increase in strength decreases and the curve is asymptotic to an upper bound value. The following describes the relationships suggested in ACI [ACI, 1982] and commonly used for the variation of concrete strength and modulus of elasticity with time.

2.1-1 Concrete Strength f'c

The cylinder strength $f'_c(t)$ is computed using an equation of the form

$$f'c(t) = [t/(a+bt)]f'c(28) \qquad (2.1.1\text{-}1)$$

Where, $f'_c(28)$ is the 28-day strength; t, is the time in days after casting of the concrete; and, a, and, b, are constants which define the shape and the upper bound of the curve. The values of a and b depend on the type of cement and the curing method used for the specimen. For type I (common) cement and moist curing, the recommended values are a = 4.0 and b = 0.85.

2.1-2 Modulus of Elasticity $E_i(t)$

The time dependent elastic modulus $E_i(t)$ is computed as a function of the cylinder strength $f'_c(t)$ using the following formula:

$$E_i(t) = 33\, w^{1.5}\, [f'_c(t)]^{1/2} \qquad (2.1.2\text{-}1a)$$

Where, w, is the unit weight of concrete in pounds per cubic foot, and f'c and $E_i(t)$ are in psi.

2.2 Creep

Creep strain $\varepsilon^c(t)$ is a stress originated strain. It is defined as the increase in strain under sustained stress applied to a concrete specimen under constant humidity and temperature. Creep strain does not include the instantaneous elastic deformation. Its rate decreases to zero over time and is only partially recoverable Fig. 2.2-1. Structural response is significantly in error when creep effects are neglected. A larger creep is generally expected to yield higher deformations and significant stress redistribution.

Creep strain at any time under constant stress is computed using one of the many relationships available. The following ACI [ACI 1982] expression is in common use:

$$C(t) = K_s * K_H * K_h * K_\tau * \{(t - \tau)^{0.6}/[10 + (t - \tau)^{0.6}]\} * Cu \qquad (2.2\text{-}1)$$

Where,

t = observation time in days
τ = age at loading in days
$C(t)$ = Creep coefficient = (creep strain at time t)/(initial immediate strain)
C_u = ultimate creep coefficient determined by experiment
K_s, K_H, K_h, K_τ are coefficients accounting for the slump of mix, humidity, member thickness and age of loading. [ACI 1982]

Ultimate creep coefficients generally vary between 1.5 and 3.0.

2.3 Shrinkage

Shrinkage strain $\varepsilon^s(t)$ is a non-stress originated strain. It is defined as the deformation under neither load nor temperature change. Shrinkage of concrete is primarily due to loss of water upon drying (drying shrinkage) and volume change due to carbonation (carbonation shrinkage) Fig. 2.3-1. Shrinkage rate decreases to zero over time. Similar to creep time dependency, the structural response is significantly in error when shrinkage effects are neglected. Shrinkage will cause significant stress redistribution.

Several relationships are available to compute shrinkage strains. The following expression given in ACI [ACI, 1982] is commonly used.

$$\in^s(t) = K_s * K_H * K_h \{(t - to)^e/[f + (t - to)^e]\} * \in^s{}_u \qquad (2.3\text{-}1)$$

Where,

$\in^s(t)$ = shrinkage strain at observation time, t
$\in^s{}_u$ = ultimate shrinkage strain determined by experiment
t = observation time in days
to = age of curing in days
f, e = constants, determined from experiments [ACI 1982]
K_s, K_H, K_h are slump, member size and relative humidity correction factors [ACI 1982]

For normal concrete, the constant "f" is between 20 and 130, and "e" between 0.90 and 1.90. Ultimate shrinkage strains vary between 300 and 800 micro strains.

2.4 Loss of Stress in Prestressing

2.4.1 Relaxation in Prestressing

Prestressing time dependent behavior is due entirely to the relaxation phenomenon, similar in nature to the creep component of concrete. The relaxation of prestressing steel is defined as the decrease in tendon force with time under constant strains (Fig. 2.4-1). Most prestressing steel available is generally sold as either low relaxation or stress relieved material. Low relaxation prestressing steel experiences less reduction in force with time compared to the stress relieved counterpart.

The prestressing tendon steel is considered as a linear elastic material subject to time dependent strains due to relaxation. The following equation [Magura et al ,1964] is generally used for the calculation of tendon stress due to relaxation under constant strain.

$$f_s = \{1.0 - [(\log t)/c]*[(f_{si} / f_{sy}) - 0.55]\} f_{sy} \qquad (2.4.1.1)$$

Where,

f_s = steel stress at time t
f_{si} = initial steel stress
f_{sy} = 0.001 offset yield stress
c = constant (= 10 for stress relieved strand; = 45 for low relaxation strand)
t = time in hours after stressing

2.4.2 Other Time-Dependent Parameters

Prestressing steel may be indirectly subjected to the effect of other time dependent parameters, creep, shrinkage and aging of concrete. Further, strain in concrete due to changes in loading and temperature result in stress changes in prestressing steel, and thereby the deformation and stresses in the structure. Consequently, the prestressing force profile will also undergo a redistribution in force.

2.5 Construction Schedule

Creep, shrinkage and aging of concrete and relaxation of prestressing steel may be classified as time dependent material inherent parameters. Another class of time dependent parameters relates to construction scheduling, and selection of design scheme, such as span-by-span construction compared to the free cantilever method. Segmentally erected prestressed concrete bridges generally follow a repetitive cycle in construction operations. As will be demonstrated in the design examples presented herein, the duration of each construction cycle also affects the stresses and deformations of the completed structure. Time dependent material parameters coupled with design type selection such as cantilever construction will result in significant stress and deflection profile redistribution of dead and prestressing forces from the initial to the final structural configuration Fig. 2.5-1

3 - PROCEDURES TO ACCOUNT FOR TIME DEPENDENT EFFECTS

3.1 Determination of Material Model

3.1.1 Code Models

A - Characteristic Values (Cu, ε_s^u)

The ultimate creep Cu and ultimate shrinkage ε_s^u coefficients represented by points A in Fig. 3.1-1b are normally determined through laboratory testing of concrete specimens. These are the estimated maximum values of creep and shrinkage a concrete specimen can attain with time under sustained conditions. The conditions for which the ultimate creep and shrinkage are estimated are selected to be those of the projected prototype structure. Apart from the fact that each concrete mix and environmental condition leads to a different ultimate value (Fig. 3.1.-1b), relationships proposed in different design codes, such as ACI [1982] and CEB [1978] result to slightly different ultimate values for the same specimen and environmental condition.

B - Variations of time dependent parameters

In addition to a difference in the ultimate values for each of the parameters creep and shrinkage predicted by the different design codes, the path of reaching the ultimate values also differs among the design codes. Refer to Fig. 3.1.1a for the schematic illustration of this feature. Curves 1, 2 and 3 which represent different design codes are each scaled to a common ultimate value as shown with point A. It is clear that the different representations do not match after each is normalized such as to have a common ultimate characteristic value. Consequently, different analyses performed on identical structures with different code requirements for the same ultimate coefficients will generally yield different results since the solutions are generally based on the path history which is different among the different code representations.

C - Scaling of creep and shrinkage curves

If after adopting a design code for concrete model, it is observed that its application to similar structures consistently yields a lower or higher estimate of the time dependent response, the code values can be scaled as shown in Fig. 3.1-1c. The scaling retains the shape of the code defined path with time, but increases or decreased the time dependent values proportional to the scale factor.

3.1.2 Laboratory Models

A - Material tests, their presentation, and interpretation

(i) Shrinkage
Shrinkage time dependent parameters may be obtained through laboratory testing of specimens built to specifications (Fig 3.1-2a) . A few readings are normally made for observation times extending a few weeks only beyond loading age. Extrapolation of these readings is necessary to determine shrinkage properties beyond the time range of the tests. The engineer specifies the loading ages and the observation times at which the shrinkage of the specimen is recorded. The expected analysis time range generally extends beyond the time limit of measured observations (point C on the graph is the last measured observation). To afford the extension of the results, a best fit curve through the readings is defined and is subsequently used to extrapolate the laboratory measurements to the anticipated time limit of the analysis (point D in Fig. 3.1-2). If it becomes necessary to analyze a structure for a period beyond the range of the observed, or extrapolated values, the last time period in the shrinkage time table is assumed to represent the ultimate shrinkage coefficient. For example, if the shrinkage table from the laboratory terminates on reading for 5 years, but a solution for 20 year is sought, the analysis presented herein assumes the last entry of the table to be the ultimate shrinkage coefficient. The analytical solution can be improved by first extrapolating the range of the table of shrinkage coefficients and then performing the response analysis.

A somewhat modified approach, as discussed in the following, is adopted for the creep parameter.

(ii) Creep

In using laboratory measured values to investigate the creep response of a structure three measurements are sought. These are the instantaneous response to the load, the changes in the specimen deformation, and the ultimate deformation with time of the specimen.

Single Loading Case

The simple case involves the generation of a creep table for the prediction of response of a structure to a single and instantaneous application of loading. An example of this case is the creep response of a precast prestressed girder in a bridge construction, when the topping slab is poured over a row of parallel girders, such as the schematic of Fig. 3.1-3. For creep computations, the load due to the weight of the topping is considered to have been applied instantaneously the applied loading remains on the precast prestressed girder indefinitely. The associated lab test results for the creep analysis of this girder are likely to be as follows.

(a) Refer to Fig. 3.1-2. The first prerequisite is that a concrete specimen (such as a standard cylinder) of the precast girder be available. This specimen must have been kept under the same environmental conditions (temperature, humidity) that the precast girder has experienced.

(b) The creep response of concrete depends on the age of concrete at the time the loading is applied. For this reason, at the same age that the prototype precast beam receives its topping, the test cylinder is subjected to an axial loading and its instantaneous shortening is measured. Line AB in Fig. 3.1-2b indicates the instantaneous shortening of the test cylinder. From the instantaneous shortening, using Hook's Law for uniaxial loading, the modulus of elasticity of the concrete at the time of loading is calculated.

(c) The loaded test specimen is kept under the same environmental condition as the prototype girder. From time to time, such as at days 1, 3, 5, 7, 14, 28, ... the shortening of the test specimen is measured. The creep strain on each of the observed days is calculated as follows.

Measured strain is:

$$\varepsilon = (L - Lo)/Lo \qquad (3.1.2\text{-}1)$$

Where

L = is the new length,
Lo = is the original length

Creep curves are normally normalized with respect to the stress used in their determination. Subsequently, the measurements recorded at the different observation times must be scaled by the stress applied to the specimen. The creep strain ε_c is therefore

$$\varepsilon_c = \varepsilon / f_c \qquad (3.1.2\text{-}2)$$

Where f_c is the axial stress on the test cylinder. It is equal to the applied loading divided by the cross-sectional area of the cylinder. As an example, for the observed day H, shown in Fig. 3.1-2b, the total strain is shown by the ordinate of point G. The creep strain to be used in the creep computations is the difference between the ordinates of points G and B.

(d) If the last observed lab measurement is point C on the chart, but it is intended to estimate the response of the structure for a longer time period (point D on the chart), a curve is approximated through the measured values B to C. The approximated curve is extrapolated to give the estimated creep strain for point D on the curve. The following relationship is generally used to obtain an approximate curve fit to the measured creep strains

$$C(t) = K[(t - \tau)^a/[b + (t - \tau)^a] \qquad (3.1.2\text{-}3)$$

Where,

t	= observation time in days
τ	= age at loading in days
$C(t)$	= Creep coefficient = (creep strain at time t)/(initial immediate strain)
K	= coefficient accounting for the slump of mix, humidity, and member thickness
a,b	= coefficients to be determined experimentally

Should additional values between the last laboratory reading (point C) and the assumed limit of the extrapolation day (point D) become necessary, the relationship for the approximated curve is used to generate assumed observation values such as point E in Fig. 3.1-2b.

Multiple Loading Case

A different scenario is when the long-term loading on the structure is not applied at one time. Consider the case of a free cantilever construction (described in detail in Section 4.2). As new segments are placed, the previously installed segments receive significant additional loading. In other words, the loading on a given segment does not remain constant. The procedure for the lab generated creep tables to be used for the creep computations is described with the aid of Fig. 3.1-2c.

(a) Several test cylinders from each segment of the bridge are cast and maintained in the same environmental condition as the prototype segment. In Fig. 3.1-2c, it is envisaged that the prototype segment will receive three significant loads applied at ages marked as A1, A2 and A3.

(b) In concert with the loading of the prototype, or as closely as practical, the test specimens are loaded with a convenient value. Each specimen is kept under observation and its creep strain determined following the procedure described in the preceding for the single loading case. The outcome of the observation will be a table containing: moduli of elasticity associated with each of the three loading times; table of observation days and the associated shrinkage values for each of the three specimens.

(c) Similar to the single case loading, the observed lab measurements can be extrapolated to generate data for prediction of the structure response beyond the last day of observation.

In actual construction, in many cases such as the free cantilever method, the creep response of a segment must be estimated before the segment is installed. Adjustments to vertical alignment (camber adjustment) are effected at the time of installation of segments. In such cases, two steps are suggested. First, prior to concreting a segment, specimens of the same mix as projected for the segment to be cast are cast, tested in the lab, and their observed response used to predict the creep of the prototype. Second, specimens from the actual segment are taken and tested to verify the validity of the first group. Should the two results be significantly different, a re-analysis is performed for the bridge under construction. Adjustments necessary to the predicted camber calculation are introduced on the alignment of subsequent segments.

Construction cycles oftentimes differ from the predicted schedule. A set of laboratory observations geared to specific loading dates, such as shown in Fig. 3.1-2c (days A1, A2, and A3) needs to be modified for the intermediate loading conditions. The modification is achieved through linear interpolation between the approximated curves of the given observation dates.

B - Relaxation Tests

Almost all prestressing wires come with the manufacturer's data on relaxation. The loss in prestressing depends primarily on the initial stress of the loaded strand wire. The loss in prestressing can vary by several times depending on the metallurgical composition and the manufacturing process of the wire.

To obtain the relaxation characteristics of a wire, the wire is stretched to a given strain. With constant temperature and keeping the initial strain fixed, the loss of force in the wire is measured after a given time (typically 1000 hours)

A set of laboratory data for relaxation test can look similar to the table shown below.

TABLE 3.1-1 EXAMPLE OF RELAXATION TEST RESULTS

	Ratio of initial to breaking stress	Loss in stress (%)	Hours
1	0.80	12	1000
2	0.70	8	1000
3	0.60	4.5	1000

The relaxation coefficient , c, can be computed from the following formula with the parameters substituted with values obtained from the previous table. The following expression is obtain through a rearrangement of equation 2.4.1-1.

$$c = [\log(t)/(1-f_s/f_{si})]^* [1.053(f_{si}/f_{pu})-0.55] \qquad (3.1.2\text{-}4)$$

In this relationship, c, is calculated from the laboratory test. It is then substituted in the relaxation formula Eq 2.4.1.1.

3.2 Implementation of Material Models in Practical Design

3.2.1 Stress History

Among the time dependent parameters of concrete, only creep effects are function of the entire stress history Fig. 3.2-1a. Creep strains are defined as the increase in strain value under a sustained applied stress. Subsequently, the creep component does not include any instantaneous deformation as shown in Fig. 3.2-1b. The creep rate decreases with time and is only partially recoverable with load removal. A compact solution for the prediction of creep strains for the entire stress history has been developed by Mokaddam (1969) and Kabir (1976). The method has since been successfully implemented and used by Kang and Scordelis (1980). The method is now the preferred procedure for modeling creep response. Creep response curves for a specimen subjected to an axial loading with reversal for each of the ACI and Hong Kong codes is presented in Fig. 3.2-1c for illustration. The creep component is partially recoverable for unloading as well.

3.2.2 Procedures for Implementation of Stress History

A - Creep history

The formulation of the creep equation is based on the assumptions that the principle of superposition is applicable as shown in Fig. 3.2-1b and that the creep component is a linear function in stress. The creep component may be expressed with the following integral

$$\varepsilon_c(t) = \int J(t,\tau)\, d\sigma(\tau) \qquad (3.2.2\text{-}1)$$

Where $J(t,\tau)$ is the creep compliance function and $d\sigma(\tau)$ is the increase in stress with loading age. The creep compliance function is substituted by an exponential series function of time, loading and retardation times. The material constitutive relationship is subsequently substituted in the recursive relationship to extract the creep component.

B - Relaxation in prestressing

In an actual bridge, strain in prestressing strand is not constant. Creep, shrinkage and external loads cause additional changes in strain over time. To incorporate the additional force variations over time, Hernandez and Gamble (1975) have suggested a procedure to compute the stress in strand subject to an applied strain history. It is based on the assumption that all non-relaxation changes in tendon force occur at the ends of the time steps (Fig. 3.2-2)

At time t_1, externally caused strain changes cause the tendon force to change to f_{s1}. To compute the stress relaxation δf_{r2} during the time interval δt_2, Eq. 2.4.1.1 previously defined is used to calculate a fictitious initial tendon force f_{si1} which would have relaxed by δf_{r1} to f_{s1} during δt_1. The stress f'_{s2} may then be found, assuming f_{si1} as the initial tendon force and above equation Eq. 2.4.1.1. This procedure is applied to each tendon segment during each time step to arrive at the total stress relaxation at time t_n.

3.2.3 Total Stress Losses in Prestressing

Based on Fig. 3.2-3, another important factor in modeling of prestressing is that the analysis should allow the strands to experience the deformation of the structure due to other time-dependent factors, such as creep. The representation of prestressing by equivalent forces, as is the case in load-balancing method and its variations, is not adequate. The method used for the time-dependent analysis is illustrated in Fig. 3.2-3.

Fig 3.2-3 shows a concrete segment represented as a finite element and the portion of a tendon, tendon segment, associated with it. The ends of the tendon segment and the frame nodes (centroidal points at ends of concrete segment are considered fixed) on the end planes of concrete segment are rigidly connected. The tendon is modeled as a truss element with the tendon and concrete segment end points displacements related according to the assumption of plane sections remains plane. Thus, impact of deformations due to creep, shrinkage, concrete aging strain, temperature, and applied loading are all accounted for in the tendon through its immediate elastic and long-term relaxation characteristics. The **long-term stress losses** are implicit in the modeling.

4 - SENSIVITY OF STRUCTURE TO TIME DEPENDENT EFFECTS

The following illustrates the sensitivity of two typical structural systems to the selection of the time dependent parameters. The sensivity is measured through changes in the structure's short -. and long term deformation, stresses, and moments. The extent of moment redistribution is itself an indication of a structure's sensivity to that structure's time dependent parameters.

The studies reported herein were conducted using the ADAPT-ABI software [ADAPT, 1996]

4.1 Precast Prestressed Girder With Post-Tensioning and Composite Topping

4.1.1 Geometry and Support Conditions

The structure selected is a simply supported composite concrete beam consisting of a precast prestressed bridge girder and a cast in place slab. In addition to pretensioned strands, the girder is post-tensioning with both straight and profiled tendons. The objective of the post-tensioning is to better control he final girder's camber and stresses.

The girder's span is 152.33 ft (46.43 m) (Fig. 4.1-1). The cross section of the beam corresponds to AASHTO Type IV, an I-shape section with 54 in (1370 mm) depth and flanges of 22 (560 mm) and 26 (660 mm) in at the top and the bottom, respectively. It has a solid section at each end with a constant web equal to the width of the top flange (22 in = 560 mm). These solid sections have a length of 40.5 inch (1030 mm) and a tapered transition of 12 inch (310 mm).

The topping is a 7.5 in (191 mm) slab 79.2 in (2012 mm) in width. The slab is cast after the beam has been installed at its final location.

Both the prestressing and the post-tensioning tendons are made up of 7 wire strands. Their layout and arrangement within the section is shown in Fig. 4.1-1.

The supports allow for the shortening of the beam along its length.

TABLE 4.1-1 SECTION PROPERTIES OF THE COMPOSITE BEAM COMPONENTS

SECTION	AREA	INERTIA	Y_t	Y_b
Solid End	1137 in^2	289599 in^4	28.11 in	25.89 in
Midspan	789 in^2	260740 in^4	29.27 in	24.73 in
Topping	594 in^2	2784.4 in^4	3.75 in	3.75 in

Note : 1 in = 25.4 mm

Legend: Y_t = distance of the centroid to the top fiber

Y_b = distance of the centroid to the bottom fiber

4.1.2 Material Properties

4.1.2.1 Concrete

- f'c beam = 14500 psi
- f'c topping = 8000 psi
- Unit weight = 150 pcf
- Alpha = 0.000006 /degree F (Coefficient of thermal expansion)
- Creep Characteristics = based on ACI -206 (AASHTO)
- Creep Coefficient = 1.8
- Shrinkage Characteristics = based on ACI -206 (AASHTO)
- Shrinkage Coefficient = 0.00048

4.1.2.2 Nonprestressed Steel

- Es = 29000 ksi
- Percentage of reinforcement in concrete = 0.1%

4.1.2.3 Prestressing Steel

A - Strand Properties

- As = 0.217 in^2 (140.0 mm^2) 7 wire strand, 1/2" nominal diameter
- Strand type = low relaxation (c=45)
- fpu = 270 ksi
- Esp = 28000 ksi
- Alpha = 0.000006 /degrees F
  (Coefficient of thermal expansion)

B - System Parameters

- Coefficient of angular friction for prestress = 0.0 /radian
- Coefficient of wobble friction for prestress = 0 / in
- Coefficient of angular friction for post-tensioning = 0.25/radian
- Coefficient of wobble friction for post-tensioning = 0.0003 / in (1.18E-5/mm)

C - Stressing

- fpi for prestress = 0.75 fpu
  = 202.5 ksi (14.25 kg/cm^2)
- Anchor set for prestress = 0 in
- fpi for post-tensioning = 0.80 fpu
  = 216 ksi (15.2 kg/cm^2)
- Anchor set for post-tensioning = 3/8 in (9.5 mm)

4.1.3 Discussion of results

A parametric study is conducted to investigate the sensitivity of the simply supported prestressed beam response to variations in time dependent factors creep and shrinkage. In order to single out the impact of the time dependent parameters studied, in each analysis all the time dependent parameters, except the one being studied, are suppressed. The ranges of the parameters investigated are:

- For creep: Cu = 0 to 4
- For Shrinkage ϵ^s_u = 0 to 1000 micro strain

The effects of ultimate creep coefficient and ultimate shrinkage coefficient on the beam midspan deflection, midspan top and bottom stresses and prestress losses are shown in Fig. 4.1-2 and 4.1-3 respectively.

The midspan deflection plot is normalized with respect to the 28 day instantaneous dead loading deflection, in which the dead weight of the completed structure is assumed to have been applied instantaneously on day 28. The values in the figure are negative, since the beam's response to an instantaneous applied weight is downward deflection, whereas the prototype's deflection is upward on account of prestressing.

The midspan stresses are normalized with respect to the square root of the 28 day concrete strength (f'c) for convenient comparison with allowable stress values.

The percentage loss in prestressing stress is plotted for a representative prestressing wire. For this purpose, the lowest prestressing strand in the girder's cross section is selected as the representative strand.

The Figures 4.1-2 and 4.1-3 clearly highlight the sensitivity of the beam to the time dependent effects. Except for the normalized midspan deflection, with variations in creep for which the curve is asymptotic to an upper bound value, deflection, stresses and prestress losses vary essentially linearly with changes in creep coefficient. The designer should be cautioned that the effects of time dependent factors are not monotonic in the sense that these effects might amplify or attenuate the response for a given parameter. The variation of midspan deflection with the shrinkage coefficient (Fig. 4.1-3a) is an ideal illustration. Observe that with increase in shrinkage coefficient, not only the deflection is reduced, but it leads to a reversal. This is due to the fact that changes in deflection due to shrinkage included stress loss in prestressing are not consistent with change in deflection due to selfweight of the beam with increased shrinkage, if the beam were not prestressed.

4.2 Cast-in-place free cantilever segmental bridge

4.2.1 Geometry and Support Conditions

The bridge selected is a 145 m long, three span symmetrical structure to be constructed using the balanced cantilever construction method. The spans are 40 m, 65 m and 40 m. The overall elevation and plan geometry of the bridge are given in Fig. 4.2-1 and 4.2-2. The bridge has a mild curvature in plan. The spans are of variable cross section. The span to depth ratio is 27 for midspan and 16.25 at the face of the central supports. The deck is rigidly connected to the two central piers and rests on rollers over the end supports.

The superstructure is made up of a single box section Fig. 4.2-2 with a diaphragm over each of the piers and abutments. The width of the bridge is 11.5 m featuring a 2.50 m overhang on each side. The deck has a high, but constant superelevation of 6.5%. The webs sweep inward with a steep slope of 1 to 2.

The construction is broken into twelve segments stemming from each central pier, a pier table consisting of the diaphragm together with a half segment on one side and a full segment on the other side, a short 2 m closure strip at the mid-span and a 6 m approach segment at each abutment. The segment lengths vary between 4 to 5 meters. The maximum and minimum segment weights are: 85 tons and 79 tons respectively.

There are 12 internal top tendons in each web, one associated with each of the segments. In addition there are four pairs of bottom tendons, three pairs for the continuity of central span and one each at the end of the side spans.

At any given construction stage, only one segment is cast. Since the pier table is designed as 1.5 segments, at any given construction stage the bridge will be off balance by the loading of only one half segment.

4.2.2 Material Properties

4.2.2.1 Concrete

- f'c = 350 kg/cm^2
- f'ci Minimum = 280 kg/cm^2 at stressing
- Unit weight = 2400 kg/m^3
- Alfa = $1E10^{-5}$ /degrees centigrade (Coefficient of thermal expansion)
- Creep Characteristics = based on ACI - 206
- Creep coefficient = 2.5
- Shrinkage Characteristic = based on ACI - 206
- Shrinkage coefficient = 0.000400

4.2.2.2 Nonprestressed Steel

- Es = $2.04E10^6$ kg/cm^2
- Percentage of reinforcement in concrete = 2%

4.2.2.3 Prestressing Steel

Prestressing is provided by grouted internal tendons

A - Strand Properties

- 12.7 mm nominal strands; each 98.7 mm^2 in area
- Strand type = low relaxation (c=45)
- fpu = 19,000 kg/cm^2

- fpy = 16,000 kg/cm^2
- Esp = 1.97E6 kg/cm^2
- Alfa = $1E10^{-5}$ /degrees centigrade (Coefficient of thermal expansion)

B - System Parameters

- Coefficient of angular friction = 0.25 /radian
- Coefficient of wobble friction = 7E-6 /cm

C - Stressing

- fpi = 15,200 kg/cm^2 (stress at jacking)
- Anchor set = 8 mm

A similar parametric study detailed in the previous case is performed for the current structure. The response of this bridge to variations in time dependent factors creep, shrinkage and construction cycle is investigated. The range of variation selected for the parameters are:

- For Creep — Cu = 0 to 4
- For Shrinkage — $\in^{s}_{u}$ = 0 to 1000 micro-strain
- For construction cycle — Cycle = 7 to 21 days

In each solution obtained, only one of the time dependent variables is changed. The remainder are kept zero, except for the construction cycle which is assumed 14 days for the reference condition

The effects of ultimate creep coefficient, ultimate shrinkage coefficient and the length of construction cycle on the midspan and pier moments are shown in Fig. 4.2-3. In each location, the moments in Fig. 4.2-3 are normalized with respect to the corresponding moments after 20 years at the same location to monitor the extent of moment redistribution. Figs. 4.2-4 and 4.2-5 show the moments and the camber of the superstructure for different time dependent parameters.

The high sensitivity of the bridge superstructure to the time dependent effects is clearly highlighted in Fig. 4.2-3 through 4.2-5. Contrary to the previous example of a simply supported case, the response of the structure in this case is not nearly linear. The maximum observed change is approximately 25% for midspan moments due to increase in ultimate creep coefficient. Another important factor in segmental erection, particularly the free cantilever construction is the camber control. Camber is highly sensitive to changes in creep, shrinkage and construction cycle parameters particularly for locations away from the pier. Higher camber requirement is noted for lower construction cycle period, since concrete segments are loaded at younger ages. Hence, they are subjected to higher creep and shrinkage, as well as loss in prestressing forces.

4.3 Remarks on AASHTO code and sensitivity to time-dependent parameters

The following observations are made from the preceding discussions, which are in general agreement with the recommendations in AASHTO [AASHTO, 1989].

The impact of variations in time dependent effects is significant and should be treated carefully. Different design parameters have different sensitivity levels hence should be considered separately. Attenuation, amplification and even reversal in response is observed for each design parameter. Selection of a higher creep or shrinkage coefficients does not necessary lead to more conservative design. Higher creep, shrinkage and construction cycle period lead to higher moment redistribution and prestressing loss.

5 - CONCLUSIONS

From the parametric study of the response of two typical structures, namely a precast prestressed bridge girder with topping slab, and a free cantilever construction, it is concluded that the deformation and stresses of these structures to the time dependent parameters are sensitive to the values of the time dependent parameters selected. The results reinforce the importance of the accurate estimate of the time dependent coefficients, and their careful implementation in design.

The conclusions arrived are based on the study of two prototype structures. More analyses are needed to arrive at general conclusions and provide specific design recommendations.

In summary, accurate prediction of material and construction time dependent parameters appear to be critical for a successful control of geometry and stresses in segmentally erected prestressed concrete bridges. Current technological advances allow full recognition and implementation of time dependent parameters for construction of special types of structures in which camber, alignment and stress control are required. In this context, AASHTO recognizes the need for special treatment of segmentally designed bridge structures. An explicit methodological approach and recommendations are still lacking. It is hoped that future AASHTO publications will address these issues.

6 - REFERENCES

Aalami, Bijan, O., "Design and Analysis of Segmentally Constructed Brides," Proceedings, CONCET'93, International Conference on Engineering and Technology, Kuala Lumpur, Malaysia, May 25-27, 1993, pp. TS2-1,16, (1993).

AASHTO, "*Guide specifications for design and construction of segmental concrete bridges,*" American Association of State Highway and Transportation Officials, Washington, D.C., pp. 118 (1989)

ACI Committee 209, "*Prediction or creep, shrinkage and temperature effects in concrete structures,*" ACI-209R-82, American Concrete Institute, (1982)

ADAPT-ABI , " *ADAPT software for analysis and design of incrementally constructed bridges*", ADAPT, 1733 Woodside Road, No 220, Redwood City, Ca 94061, (1996)

CEB-FIP, "*CEB-FIP model code for concrete structures,*" Comite Euro-International de Beton, Lewis Brooks, 2 Blagdon Rd, New Malden, Surrey, KT3 4AD, England (1978)

Hernandez, H. D. and Gamble, E. L., "Time dependent prestress losses in prestressing concrete construction," Structural Research Series N0 417, Civil Engineering Studies, University of Illinois, Urbana, May (1975)

Kabir, A. F., "*Nonlinear analysis of reinforced concrete panels, slabs and loading,*" University of California at Berkeley, SESM Report No 76-6 (1976)

Kasti, F. A., "*Nonlinear Material and Time Dependent Analysis of Segmentally Erected Reinforced and Prestressed Concrete Composite Three Dimensional Frames*", Structural Engineering and Structural Mechanics Report No. UCB/SESM-90/03, Dept. of Civil Engineering, University of California, Berkeley, pp. 248 (1990)

Ketchum, M. A., "*Redistribution of stresses in segmentally erected prestressed concrete bridges,*" Structural Engineering and Structural Mechanics Report No UCB/SESM-86/07, Dept of Civil Engineering, University of California, Berkeley, pp. 226 (1986)

Mokaddam, M. A., " *Behavior of concrete under variable temperature and loading*", Interim Report to Oakridge National Laboratory, Reactor Division, Oakridge, Tennessee (1969)

PCI Committee on Segmental Construction, "*Recommended practice for segmental construction in prestressed concrete,*" PCI, pp.22 - 41, (1975a)

PCI Bridge Committee , "*Tentative design and recommendation specifications for precast segmental box girder bridges,*" PCI Journal, pp. 34-42, Jul-Aug (1975b)

Tadors, M.K., Ghali, A. and Dilger, W., "*Time-dependent prestress loss and deflection in prestressed concrete members,*" PCI Journal, Vol. 20, No. 3, May-June (1975)

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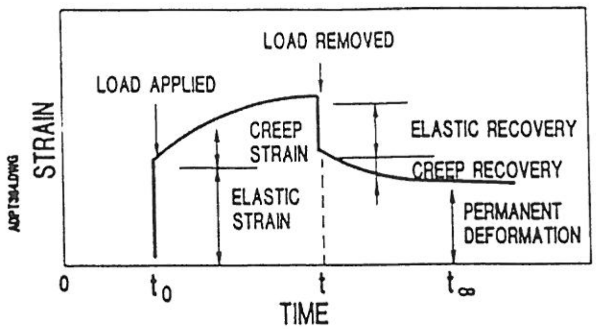
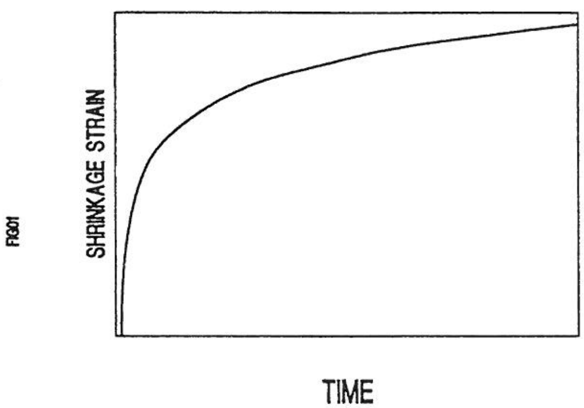


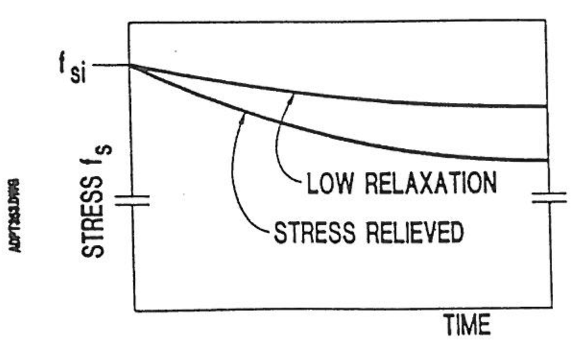
CREEP STRAIN RESPONSE OF CONCRETE

FIGURE 2.2-1



VARIATION OF SHRINKAGE STRAIN WITH TIME

FIGURE 2.3-1



STRESS LOSS IN PRESTRESSING
DUE TO RELAXATION

FIGURE 2.4-1

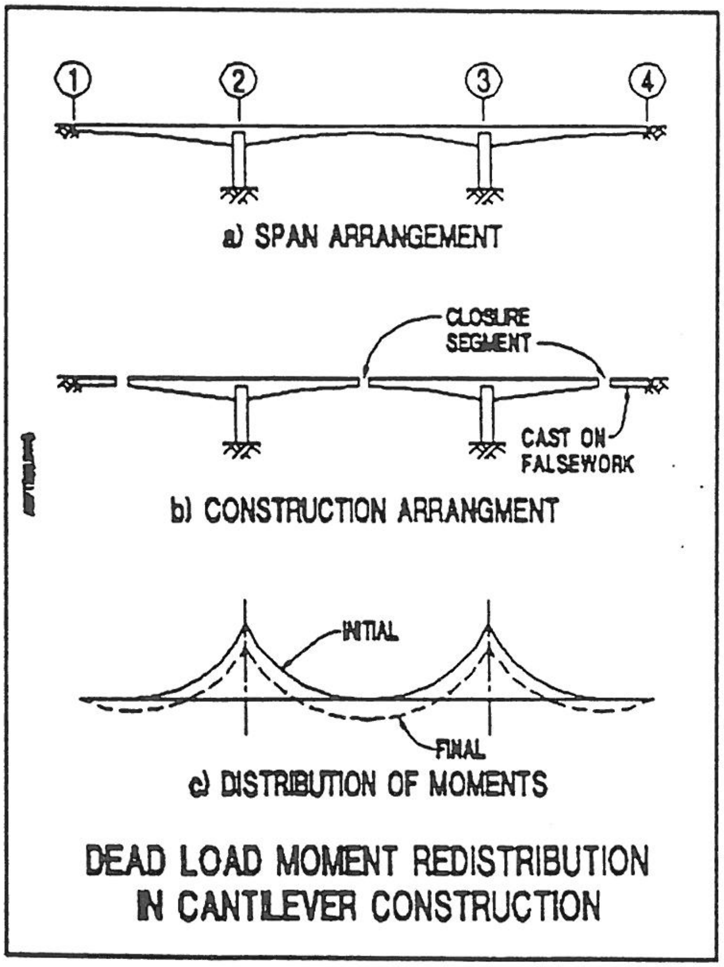
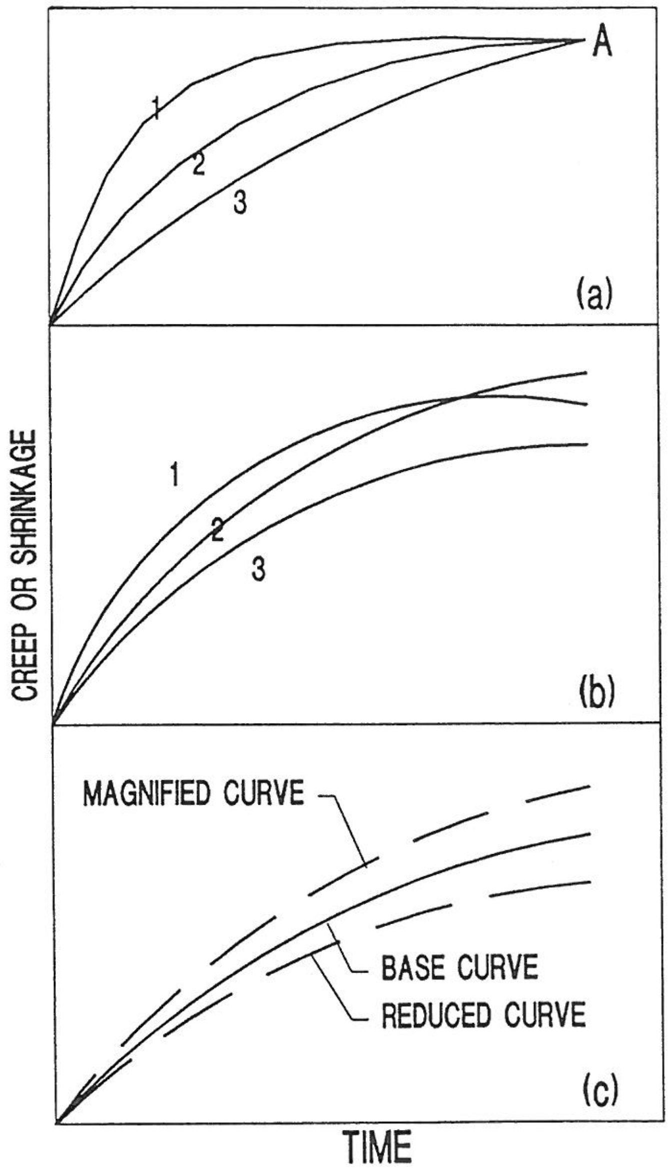
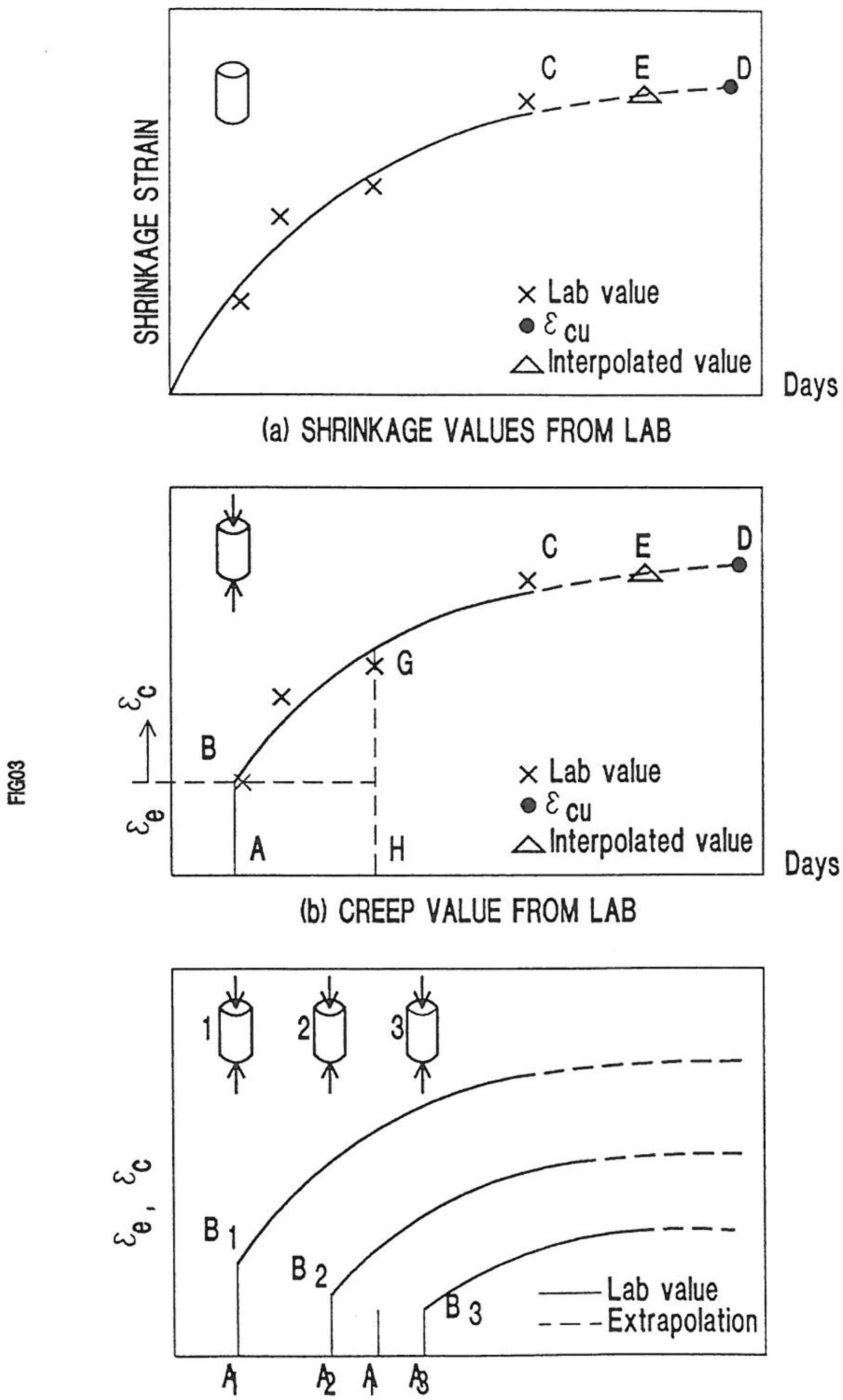

1
2
3
4
a) SPAN ARRANGEMENT
CLOSURE SEGMENT
CAST ON FALSEWORK
b) CONSTRUCTION ARRANGMENT
INITIAL
FINAL
c) DISTRIBUTION OF MOMENTS
DEAD LOAD MOMENT REDISTRIBUTION
IN CANTILEVER CONSTRUCTION

FIGURE 2.5-1



TREATMENTS OF CREEP AND SHRINKAGE CURVES IN DESIGN

FIGURE 3.1-1



(a) SHRINKAGE VALUES FROM LAB

(b) CREEP VALUE FROM LAB

(c) MULTIPLE SPECIMEN CREEP VALUES FROM LAB

PRESENTATION OF LAB VALUES

FIGURE 3.1-2

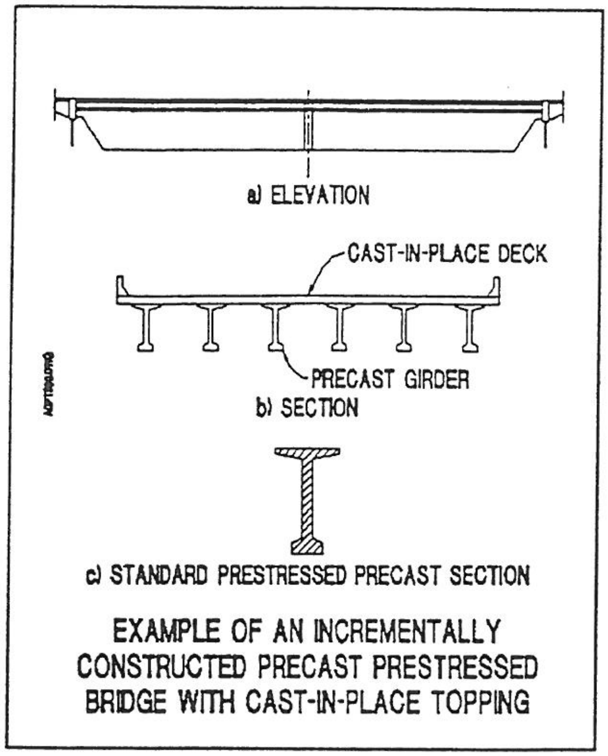


FIGURE 3.1-3

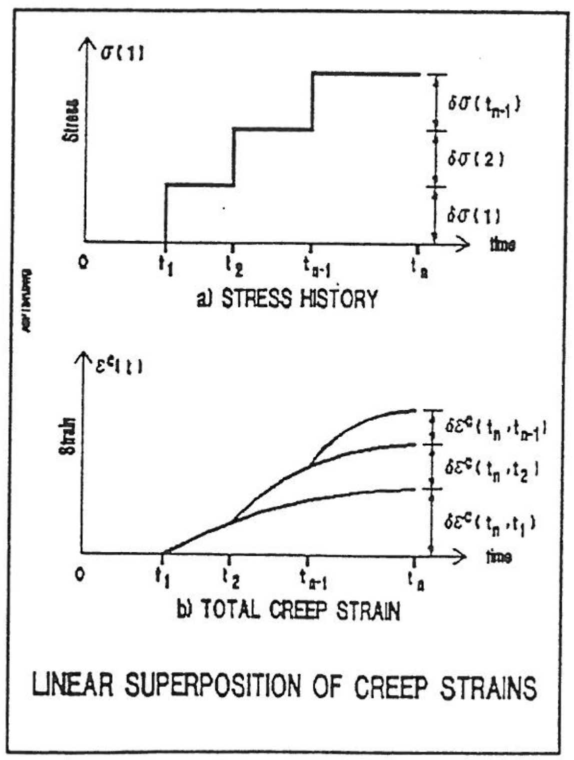
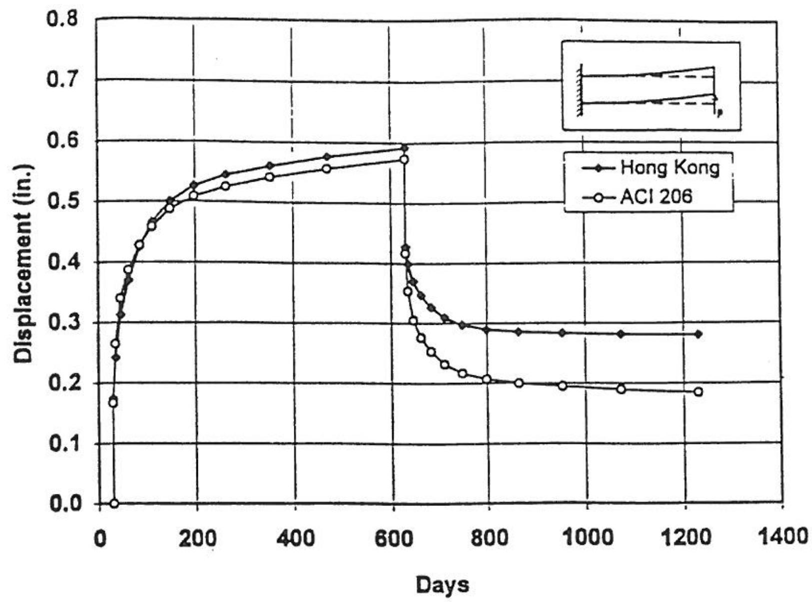


FIGURE 3.2-1



Deflection Under Sustained Load

FIGURE 3.2-1 (c)

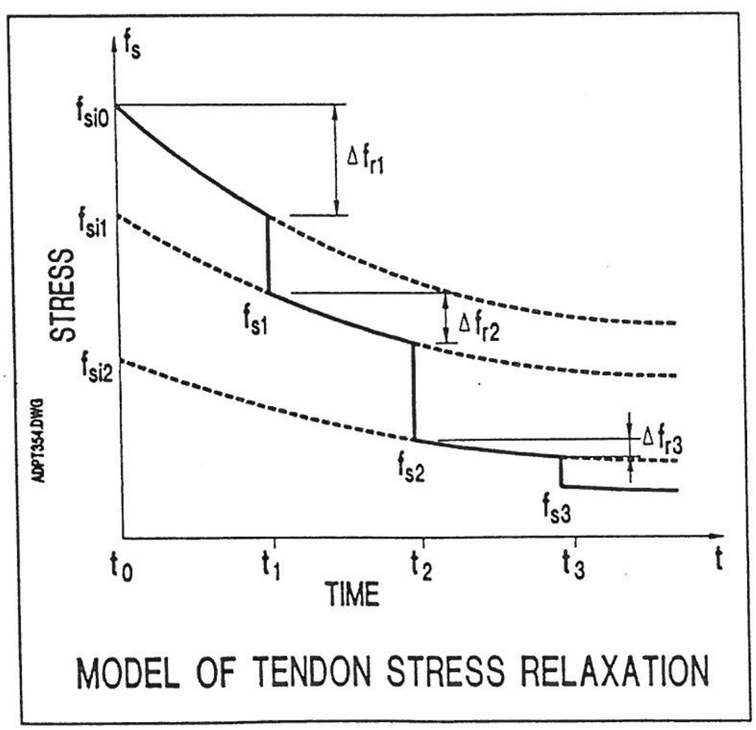


FIGURE 3.2-2

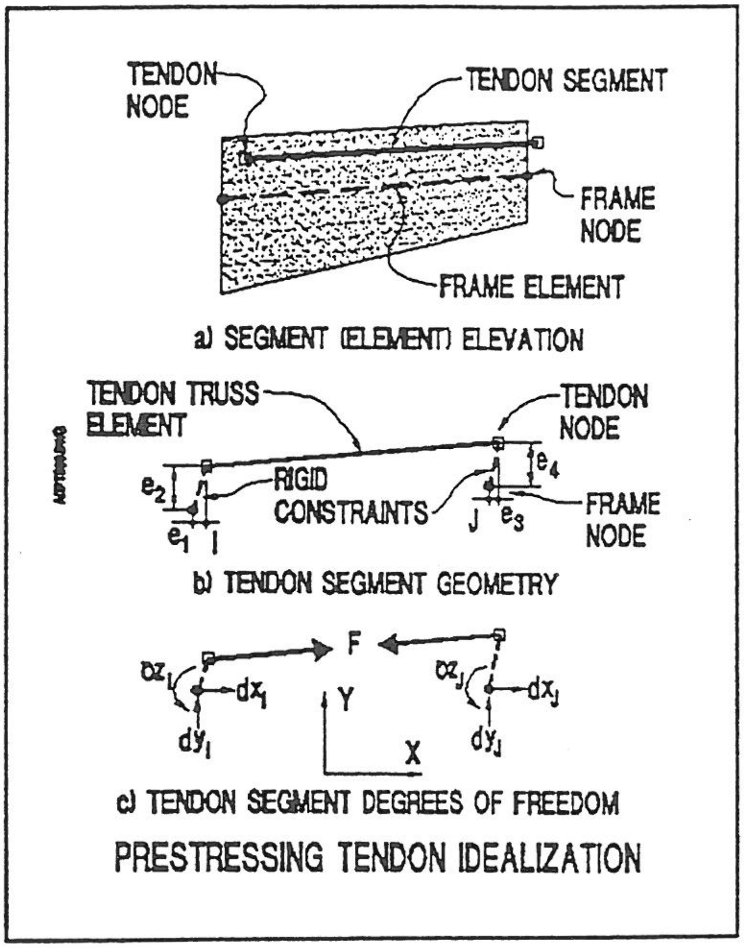

TENDON NODE
TENDON SEGMENT
FRAME NODE
FRAME ELEMENT
a) SEGMENT (ELEMENT) ELEVATION
TENDON TRUSS ELEMENT
TENDON NODE
RIGID CONSTRAINTS
FRAME NODE
b) TENDON SEGMENT GEOMETRY
F
Y
X
c) TENDON SEGMENT DEGREES OF FREEDOM
PRESTRESSING TENDON IDEALIZATION

FIGURE 3.2-3

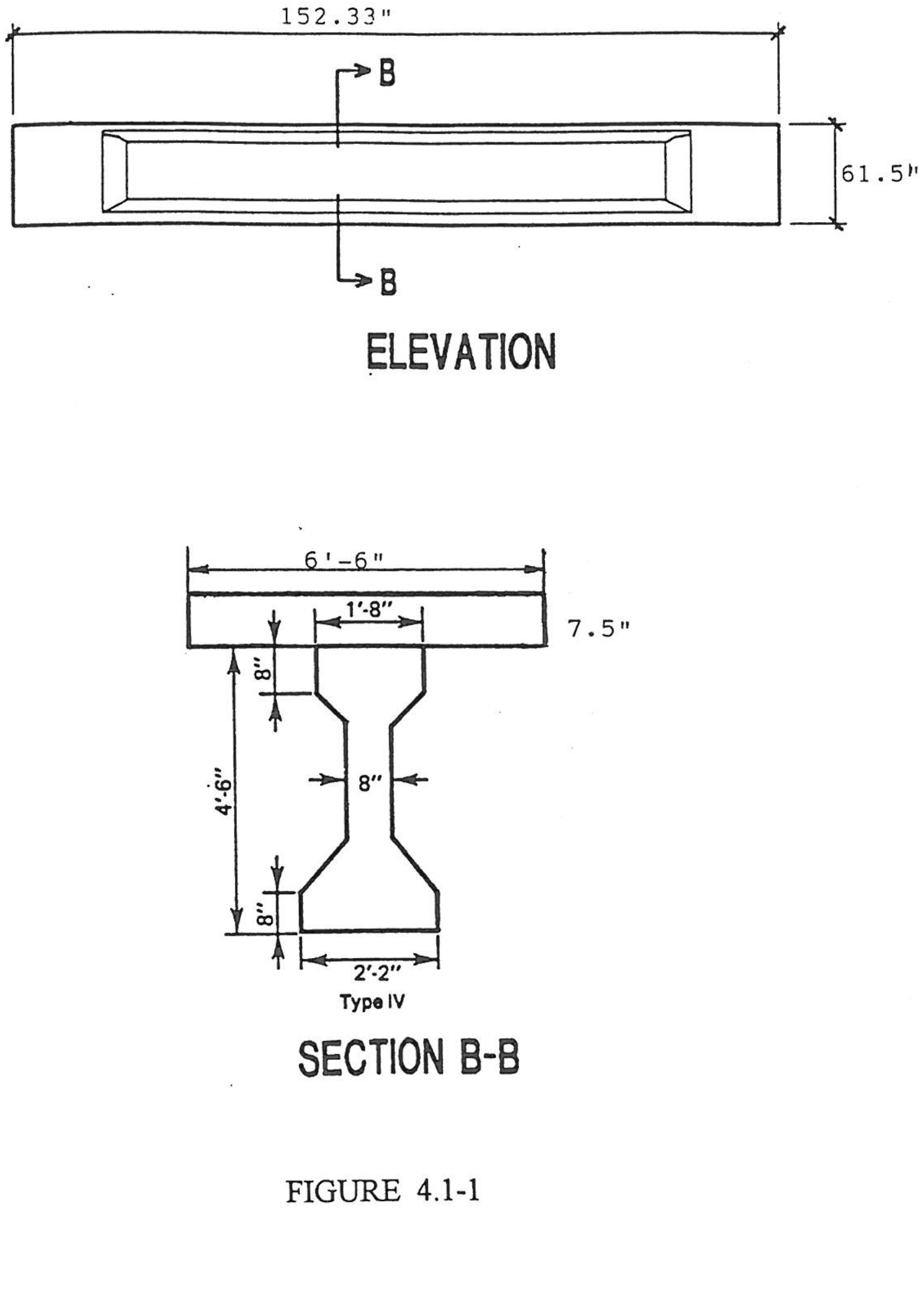


FIGURE 4.1-1

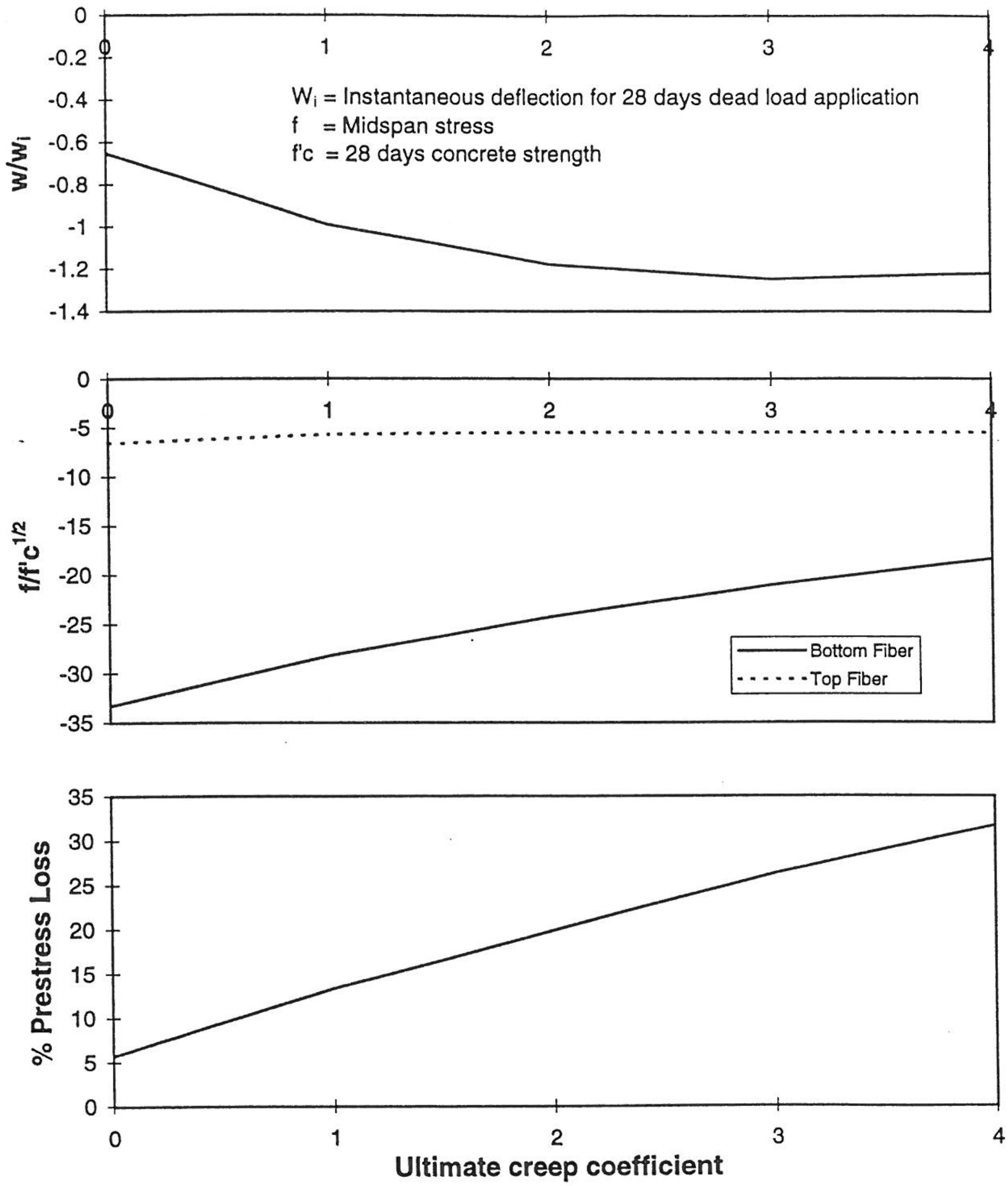


Figure 4.1.2
Sensitivity with respect to variation in ultimate creep coefficient

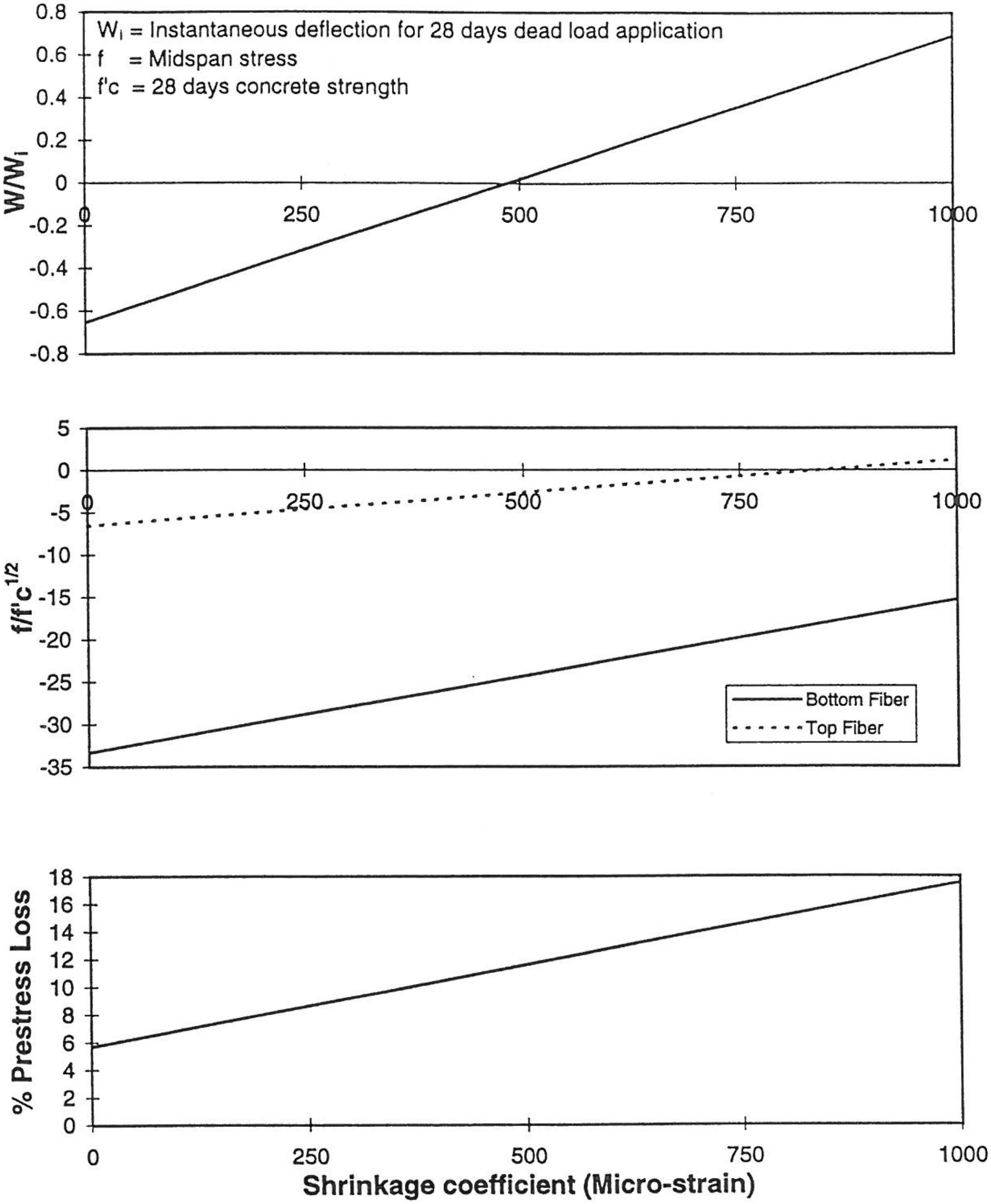
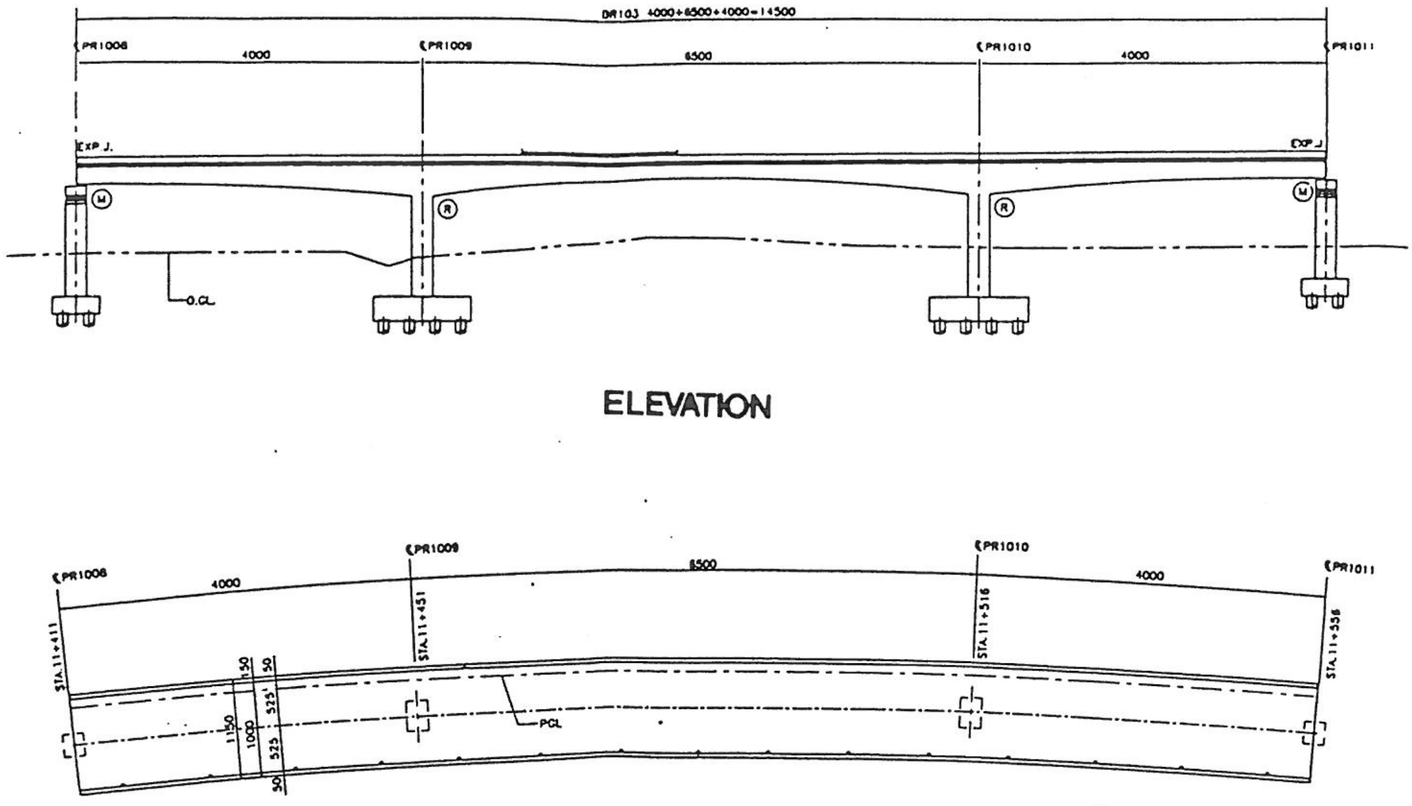
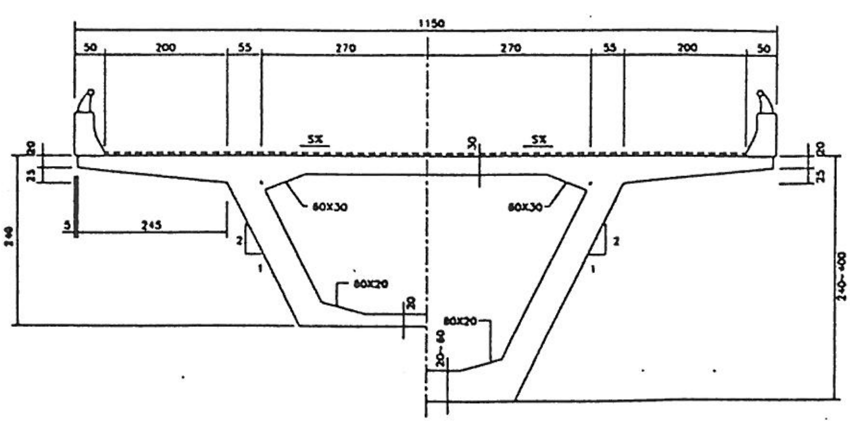


Figure 4.1.3
Sensitivity with respect to variation in shrinkage coefficient



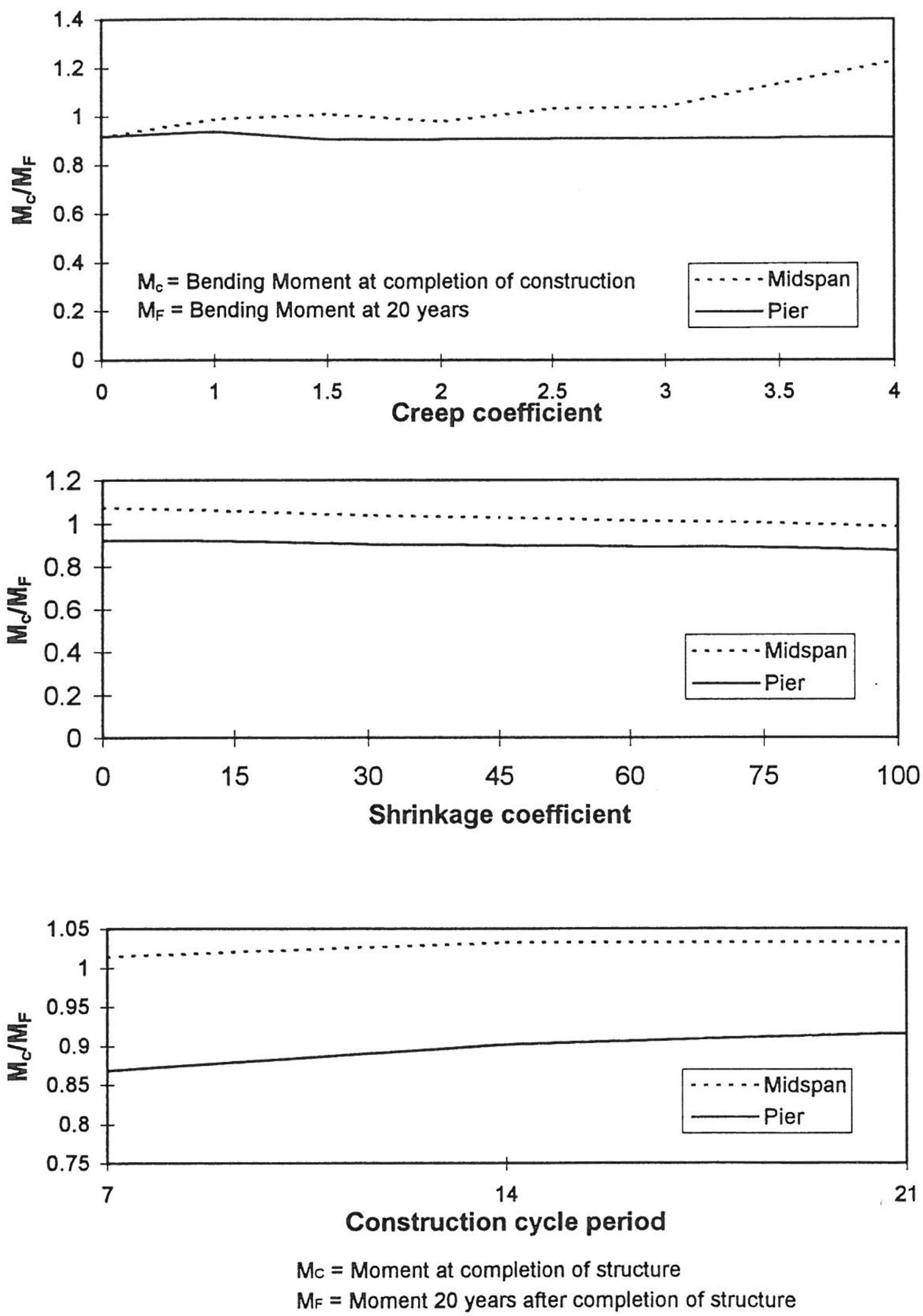
FREE CANTILEVER BRIDGE (cm)

FIGURE 4.2-1



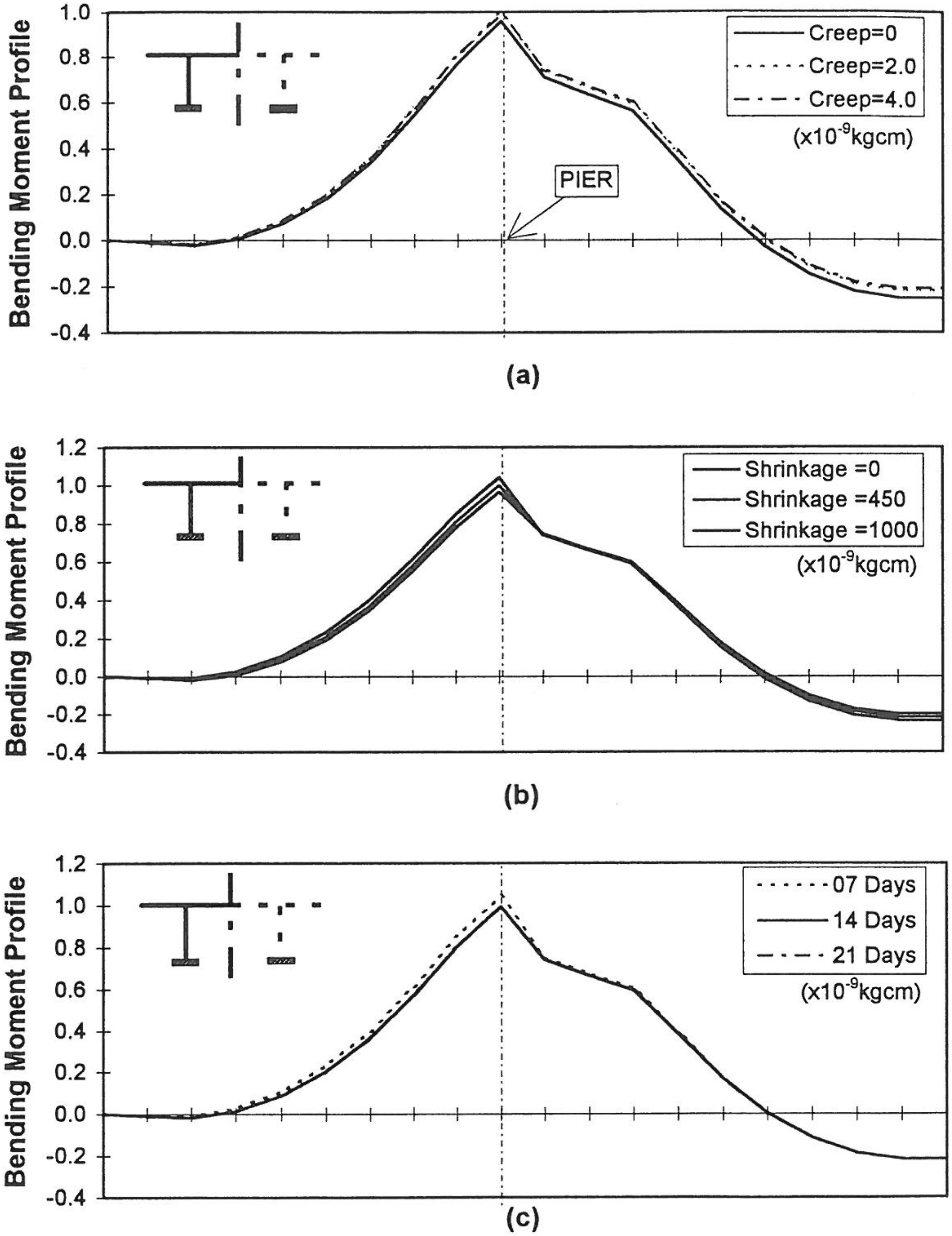
CROSS SECTION GEOMETRY

FIGURE 4.2-2 (cm)



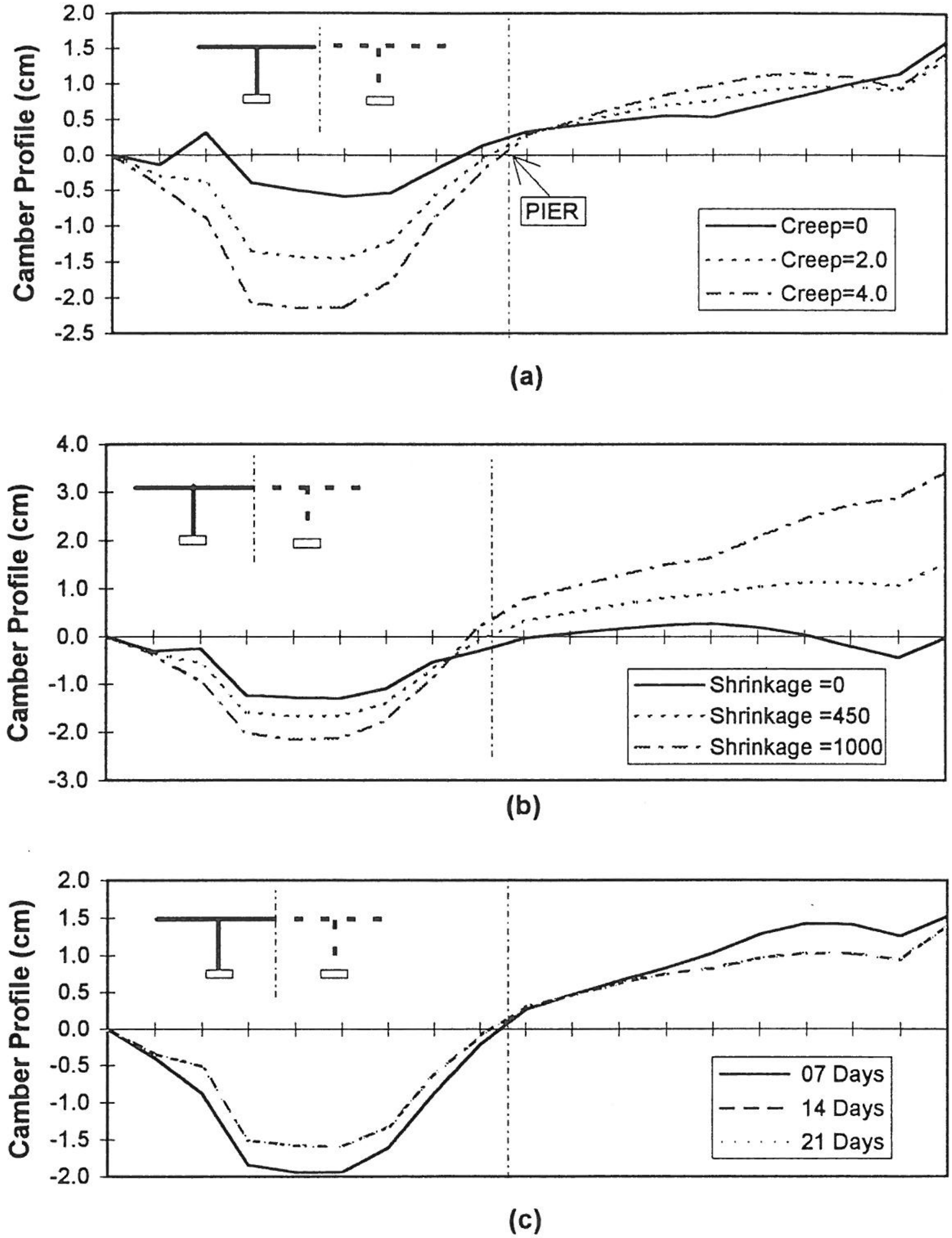
SENSITIVITY WITH RESPECT TO VARIATION IN CREEP, SHRINKAGE AND CONSTRUCTION CYCLE

FIGURE 4.2.3



20 YEARS SENSITIVITY WITH RESPECT TO:
(a) CREEP, (b) SHRINKAGE (MICRO-STRAIN), AND (c) CONSTRUCTION CYCLE (DAYS)

FIGURE 4.2.4



20 YEARS SENSITIVITY WITH RESPECT TO:
(a) CREEP, (b) SHRINKAGE (MICRO-STRAIN), AND (c) CONSTRUCTION CYCLE (DAYS)

FIGURE 4.2.5