Stress Distribution of Prestressed Concrete Structures as Influenced by Time and Temperature

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Synopsis: It is well known that creep and shrinkage of concrete cause long term deflections of continuous prestressed concrete bridges. A large amount of research has been conducted in this area. While the bridges continue to deform with time, the moment distributions of the bridges remain essentially unchanged under the influence of creep if the bridge temperature is uniform. The change of stresses is caused by prestress losses due to the creep, shrinkage of concrete and the relaxation of steel. However, rate of creep of concrete increases with temperature. The seasonal and daily temperature variations of a prestressed concrete deck not only induce thermal stresses but also significant stress redistribution due to non-homogeneous creep rates in the concrete.

The paper presents some recent findings from Steady State analyses. The provision in the latest European code EN1992 (EC2) for thermal creep is discussed. The amount of moment redistribution will be quantified against a range of thermal creep rates. The results show that stresses could be significantly underestimated if temperature and thermal creep are not considered in design. The advancement of EC2 as well as its limitation in modelling the long term behaviour of prestressed concrete bridges are also discussed. The paper also addresses the problems that can be created when the structural continuity is created for the first time due to the remedial works after the bridge has been in service many years. Under these circumstances creep can have a major effect on both serviceability and safety.

Keywords: bridge; relaxation; steady state; thermal creep; thermal stress

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INTRODUCTION

The primary consequence of creep in concrete structures is the development of time-dependent deformations. There are however occasions when significant stress changes in time also occur. It is generally believed that creep is beneficial to the behaviour of statically indeterminate structures in that stresses and bending moments will be kept close to design values derived from equilibrium and compatibility considerations. This may be seen readily to be true for the two-span prestressed beam of Fig. 1a where the bending moments are changed from the normal design values by imposing a displacement at one support. The instantaneous effect is as shown in Fig. 1b. Thereafter creep returns the bending moments and stresses asymptotically towards the pre-support displacement imposition values, i.e. the original elastic solution. This response is not general and is a consequence of several factors, viz. the prestressing force is taken to be a follow-up load (assumed here not to change in time, i.e. there is no restraint to longitudinal deformations), the creep rate of concrete is homogeneous throughout the beam and the temperature everywhere is uniform. This behaviour is not general and may be seen to be a special case of a more general steady state theory (see following section for more details)[1-3] for non-uniformly heated structures in which creep becomes a non-homogeneous property that is governed by the local temperature. The non-homogeneity is caused by an increasing specific creep (creep strain per unit stress) with temperature. For such cases the long-term, or steady state solution, is no longer the same as the initial elastic solution but is unique to the temperature distribution within the structure.

In the following sections, the prediction of thermal creep is discussed, the influence of creep on structural performance is further described and examples are given to justify the belief that creep is not always beneficial to structural performance. In the final section some cases of significant importance are described in relation to the imposition of restraints, flexural and axial, late in the life of a structure. Previously safe and serviceable structures are shown to be able to adopt a new character and display intrinsically unsafe behaviour thereafter.

RESEARCH SIGNIFICANCE

While thermal stress caused by temperature fluctuations is considered in bridge structures, temperature effect on creep induced stress redistribution has not been received the same attention. The research examines the material creep properties as reported by researchers and in the latest European code EC2. Two analytical methods to evaluate the maximum moment and stress induced by quasi-permanent load and temperature fluctuations at steady state are proposed. The effect of different thermal creep properties of concrete is also studied. The results show significant increase in support moment of continuous prestressed concrete bridges. The paper also addresses the problems that can be created when the structural continuity is created for the first time due to the remedial works after the bridge has been in service many years. Under these circumstances creep can have a major effect on both serviceability and safety.

THERMAL CREEP

It is known that the rate of creep is dependent on the temperature. Hannant[4], Ross and England[5], Browne[6] and Hansen[7] carried out experimental tests on concrete under sustained load and temperature up to 80°C. They concluded that the rate of creep increases with temperature. This extra creep occurs at temperature higher than 20°C is usually referred to as "thermal creep". Illston[8] identified transitional thermal creep which he described as an irrecoverable component of concrete creep brought about by heating concrete to a higher temperature for the first time while under load ("virgin heating"). The latest version of European Code EN1992:Part 1:2004 (EC2 for Buildings)[9] permits a constant creep coefficient for ambient temperatures between -40°C and +40°C. However, EN1992:Part 3:2006 (EC2 for Liquid retaining and containment structures)[10] recognises the effect of temperature on creep by providing a creep coefficient multiplier to factor the creep coefficient obtained at 20°C. If load is applied before heating, the code also provides a formula to calculate the Transitional Thermal Creep strain. These provisions are not included in the current British codes in the UK and represent an advancement of providing a reference for design engineers.

The influence of temperature on creep coefficient as reported by various researchers and by EN1992:Part3:2006 is summarised in Fig. 2 in the form of a creep coefficient multiplier. It is clearly shown in this figure that there is a wide range of thermal creep strain relating to different aggregates and cement types used. England[5] and Hannant[4] reported gravel aggregates in their tests whilst limestone was used by Browne[6].

The effect of the Transitional Thermal Creep is to increase deformation of the structure and to cause moment redistribution during the heating phase whenever the temperatures are not uniform. However, this strain component will not influence the long term steady state of the structure and hence will not be further discussed in the paper.

INFLUENCE OF TEMPERATURE CHANGE TO CONCRETE BRIDGE DECK

The temperature of a bridge deck changes daily and seasonally and is between two extreme limits which is usually non-linearly distributed throughout the depth of the deck. Thermal strains created by non-linear temperature changes are not compatible with plane sections beam theory. They therefore give rise to self-equilibrating stresses throughout the depth of the deck. Stresses of this type exist in addition to any other stresses which result from external constraints if the bridge deck is statically indeterminate. The total of these stresses are usually referred to as thermal stresses.

If concrete stress is within the linear creep range ($\sigma_c<0.45f_{ck}(t)$ according to EC2), specific creep is independent to concrete stress but dependent on temperature, i.e. thermal creep. The non-uniform temperature distribution in a bridge deck therefore generates non-homogeneous creep response. If the new temperature state is sustained, stresses in concrete will be redistributed because (i) the creep of concrete will progressively eliminate the thermal stresses and (ii) a new stress state will develop eventually to re-establish a compatible set of creep rates i.e. a Steady State of stress is reached.

STEADY STATE AND MOMENT REDISTRIBUTION

The Steady State is defined as the state when creep is continuing with no change in stress. The existence of a steady state of stress is supported by theoretical arguments and experimental measurements on non-uniformly heated statically indeterminate prestressed concrete beams[11,12]. Fig. 3 shows details of one test in which the bending moments and stresses changed in time while the beam was subjected to sustained loads and temperature crossfall. The recorded time variation of the support loads provided the bending moment information, while internal stress changes were monitored by recording strains on a set of elastic elements mounted within the constant bending moment section of the beam.

The experiment was conducted to determine the magnitude of the steady state support reactions while the supports were maintained co-linear. At a time (125 days) when the steady state was considered to be being approached fairly closely (from below in Fig. 3) a check was performed by imposing support displacements to increase the support reaction values to values estimated to be above the true steady state values. This produced the expected result, *viz.* the reaction values now reduced with time, thus approaching the steady state from above. A final check was performed (at 185 days) by returning the support displacements to the original co-linear form. This resulted in the steady state again being approached from below, thus confirming that the structure did in fact have a preferred long-term state for reaction values and bending moments. These in combination have been defined as defining the steady state solution to the problem for the given mechanical loads and specified temperature distribution.

Other important features of this experiment were (a) the steady state bending moment and stresses were considerably different from the initial elastic solution, both before and after heating. (b) the internal states of stress showed linear variation over the beam depth initially but became significantly non-linear as the steady state was approached. The 'recorded' stresses from the experiment showed a very close correlation with predictions from the steady state stress theory[1], Fig. 3.

When concrete creep properties are not certain during the design stage, the steady state of a concrete structure which is sensitive to creep should be considered as one of the limiting states for design checking.

STEADY STATE THERMAL CREEP ANALYSIS AT SUSTAINED TEMPERATURE

In the case that a new temperature is sustained following the initial temperature under which the structure is completed, a thermal creep analysis at sustained temperature can be performed to examine the moment and stress distribution at steady state. If the structure is subjected to a cyclic temperature fluctuation throughout its design life such as bridges, a detailed analysis accounting for the temperature cycles as described in following section can be performed.

The thermal creep analysis adopted here is based on the Rate of Creep method. The total strain rate is expressed as

$$\dot{\varepsilon} = \frac{\dot{\sigma}}{E} + \sigma f(T) \qquad [1]$$

where the dot notation indicates differentiation with respect to the normalised creep, c*, which is termed as pseudo-time, t*. The normalised creep is

$$c^* = \varepsilon_{creep} / T / f(T) \qquad [2]$$

where, T is the temperature of concrete in ^{o}C when the creep strain, ε_{creep} , occurs. f(T) is a temperature function to collapse all specific creep curves at various temperatures into one curve. f(T) is usually in the form of

$$f(T) = T + a \qquad [3]$$

a is a constant dependent on the concrete material. Refer to the thermal creep strains results in Fig. 2. a = 70^{o}C for Browne, a = 62^{o}C for EN1992:3, a = 10^{o}C for Ross & England, a = 0^{o}C for Hansen and a = -5^{o}C for Hannant.

For beam plane sections remaining plane, at steady state, for an element at y from the sectional centroidal axis, the strain rate is

$$\dot{\varepsilon}_{ss} = \dot{\bar{\varepsilon}} + \dot{\psi} y = \sigma_{ss} f(T(y)) \qquad [4]$$

where $\dot{\psi}$ is curvature rate

$\dot{\bar{\varepsilon}}$ is axial strain rate

the subscript "ss" represents value at steady state

Apply equilibrium,

$$P = \int_{yb}^{yt} \sigma_{ss}\, b\, dy \qquad [5]$$

$$M_{ss} = \int_{yb}^{yt} \sigma_{ss}\, b\, y\, dy \qquad [6]$$

where P is prestress force and M_{ss} is bending moment at steady state.

By combining the equations,

$$P = \dot{\bar{\varepsilon}} I_1 + \dot{\psi} I_2 \qquad [7]$$

$$M_{ss} = \dot{\bar{\varepsilon}} I_2 + \dot{\psi} I_3 \qquad [8]$$

and

$$\dot{\bar{\varepsilon}} = p_1 M_{ss} + q_1 P \qquad [9]$$

$$\dot{\psi} = p_2 M_{ss} + q_2 P \qquad [10]$$

where $I_1 = \int \frac{b}{f(T(y))} dy$; $I_2 = \int \frac{b\,y}{f(T(y))} dy$; $I_3 = \int \frac{b\,y^2}{f(T(y))} dy$

and $p_1 = -I_2/S$; $p_2 = I_1/S$; $q_1 = I_3/S$; $q_2 = -I_2/S$
$S = I_1 I_3 - I_2^2$

After establishing the rate of curvature, $\dot{\psi}$, a flexibility analysis can be employed using Virtual Power principle[13]. The general form is

$$\int \dot{\psi} [m_i] ds = 0 \qquad [11]$$

Where m_i is the moment distribution of the released structure with a unit moment applied at the support i, the location of the ith redundancy.

The outcome of this analysis yields the continuity moment over the intermediate support in the example of Fig. 4. With this information, the steady state stress distribution throughout the beam can be evaluated after back substitutions of $\dot{\bar{\varepsilon}}$ and $\dot{\psi}$.

STEADY STATE THERMAL CREEP ANALYSIS FOR CYCLIC TEMPERATURE FLUCTUATIONS

At steady state, the relationship of stress distribution at temperature profile T1 (lower temperature) and that at temperature profile T2 (after heating) is defined as

$$\sigma_2 = \sigma_1 + \sigma_\alpha \qquad [12]$$

where σ_α is thermal stress, developed due to temperature change from T1 to T2.

The steady state strain rates at T1 and T2 states are expressed as

$$\dot{\varepsilon}_1 = \sigma_1 f(T1) \quad \text{and} \quad \dot{\varepsilon}_2 = \sigma_2 f(T2) \qquad [13]$$

The average compatible strain rates in a complete temperature cycle are

$$\dot{\varepsilon}_{AV} = \frac{\dot{\varepsilon}_1 k + \dot{\varepsilon}_2}{1+k} = \sigma_2 T_{AV} - \sigma_\alpha \frac{k\,f(T1)}{1+k} \qquad [14]$$

where, k is the ratio of the pseudo-time at T1 to that at T2 and T_{AV} is the average value of the temperature function f(T) in a temperature cycle expressed as

$$T_{AV} = \frac{kT1 + T2}{1+k} \qquad [15]$$

Hence, σ_2 can be expressed as

$$\sigma_2 = \frac{1}{f(T_{AV})}\left[\dot{\varepsilon} + \dot{\psi} y + \sigma_\alpha \frac{k\, f(T1)}{1+k}\right] \qquad [16]$$

Apply equilibrium,

$$P = \int_{yb}^{yt} \sigma_2\, b\, dy \qquad [17]$$

$$M = \int_{yb}^{yt} \sigma_2\, b\, y\, dy \qquad [18]$$

By combining the equations,

$$P = \dot{\varepsilon}\, I_1 + \dot{\psi} I_2 + I_4 \qquad [19]$$

$$M = \dot{\varepsilon}\, I_2 + \dot{\psi} I_3 + I_5 \qquad [20]$$

and

$$\dot{\varepsilon} = p_1 M + q_1 P + r_1 \qquad [21]$$

$$\dot{\psi} = p_2 M + q_2 P + r_2 \qquad [22]$$

where $I_1 = \int \frac{b}{f(T_{AV}(y))} dy$; $I_2 = \int \frac{b\, y}{f(T_{AV}(y))} dy$; $I_3 = \int \frac{b\, y^2}{f(T_{AV}(y))} dy$

and $p_1 = -I_2/S$; $p_2 = I_1/S$; $q_1 = I_3/S$; $q_2 = -I_2/S$;

$r_1 = (I_2 I_5 - I_2 I_4)/S$; $r_2 = (I_2 I_4 - I_1 I_5)/S$; $S = I_1 I_3 - I_2^2$

By using the Virtual Power concept and the flexibility method, the steady state continuity moment and section stresses can be found.

AN EXAMPLE OF A TWO-SPAN CONTINUOUS BEAM

An axially prestressed continuous beam of two spans subjected to two temperature crossfall profiles, one linear and one non-linear, has been studied. The details of the beam and the temperature profiles are shown on Fig. 4. Analysis was carried out based on the detailed list in Table 1.

Two types of analysis have been conducted. The simpler one is to assume the temperature of the beam being sustained at an average temperature until the steady state. The other one is to account explicitly for the real nature of temperature fluctuations between two temperature states until the steady state is reached. Case 4 is the simplified approach of the Case 6 for linear temperature crossfall. Case 5 is the simplified approach of the Case 7 for non-linear temperature crossfall. In all cases, the initial temperature crossfall is T1 (uniform at 20°C) and the temperature function for Rate of Creep analysis is f(T) = T+10°C.

RESULTS AND DISCUSSION

Increase in hogging moment at support

An increasing support moment at Steady State is generally observed for non-uniform temperature crossfall. Figs. 5 and 6 show the moment distributions for the linear and non-linear temperature crossfall cases respectively. They show similar behaviour. The initial support moment caused by the quasi-permanent load is 500kNm. The application of temperature crossfall T3 reduces this initial support moment to 125kNm. At steady state, the maximum support moment occurs when the beam temperature reverts to T1. The two figures also show that both methods of (i) cyclic temperature analysis and (ii) sustained average temperature analysis give extremely similar maximum support moment at T1 which are found as 803 kNm and 795 kNm respectively, 60% larger than that of initial elastic solution under mechanical load only. Therefore, (i) the thermal creep effect on the change of support moment cannot be ignored; and (ii) a sustained average temperature analysis is recommended for practical use because it is a simpler calculation.

Influence of Self equilibrating stresses

Figs. 7 and 8 show the stress distributions of the support section for the two temperature crossfalls. The non-homogeneous creep rates causes non-linear sectional stresses leading to a set of self-equilibrating stresses which is additional to the one caused by non-linear temperature crossfall. The self-equilibrating stresses of the non-linear temperature increase the tensile stress at the top of the section from that calculated simply from the bending moment. In this case, the bending moment alone is not adequate to define the design limit for continuous bridges. For the span section, Fig. 9 shows that the creep effect is beneficial to reduce the maximum compressive stress at the top but is not desirable in increasing the tensile stress at the bottom. The change of stresses due to creep is considerably smaller in the span section than that in the support section.

Influence of thermal creep property of concrete

Here, the thermal creep of concrete is defined by the temperature function f(T) = T + a, where a is a material parameter. The larger the value of "a", the less increase in creep rate due to temperature. Fig. 10 shows that for a = 120^{o}C (rate of creep appears not to increase with temperature), the maximum support moment still increases to a value 22% and 40% larger than its initial (loading only) elastic value for the non-linear and linear temperature crossfall cases respectively. The current EC2 Part 3 implies a value of 62^{o}C for "a". Based on this assumption, EC2 Part 1 which assumes creep rate to be temperature independent up to 40^{o}C is supported by the results in Fig. 10. However, when a <40^{o}C, the support moment increases drastically and an accurate knowledge of the parameter "a" becomes important.

CHANGE OF STRUCTURAL FORM

There are often good reasons that necessitate changing the structural form of an existing structure. These can relate to sequential construction or the need to satisfy different requirements from the original design, but more frequently they are brought about during the retrofitting of structures after a period of service in order to improve serviceability or extend service life. Examples include the making of simply supported beams continuous over internal supports and the provision of additional prestress late in the life of a structure. Creep of concrete can play a dominating role in defining structural behaviour and safety after such changes have been made even though it may not have been important beforehand.

When the statically determinate, or indeterminate, form of a reinforced concrete or prestressed concrete structure is changed in this manner, time-dependent changes to the support actions and internal stresses and moments can be expected. These changes may be unimportant in the short term but can create seriously changed behaviour in the longer term with the possibility of violating the original design parameters. Special care is therefore necessary to examine in detail the nature and possible extent of the changes, in time, and to determine whether they are likely to impair future serviceability and/or safety.

In analysing problems of this type it is frequently impossible to obtain closed form solutions and one must resort to numerical step by step techniques. It is also important to adopt a representation for creep that is appropriate for allowing significant stress redistribution to occur with time. In this context formulations based on the effective modulus concept are inappropriate since they are most suitable to those cases in which stresses show little or no change in time. Both 'superposition' and 'effective modulus' methods overestimate creep recovery under a falling stress regime. As a consequence they underestimate the magnitude of stress changes in time. 'Rate of flow'[16] which distinguishes explicitly between non-recoverable flow and delayed elastic strain is a highly recommended approach. A simplification of this method is the 'rate of creep' approach. This combines the delayed elastic and flow strains in a strain rate formulation which produces results in which stress changes are overestimated. This method thus constitutes a safe engineering approach. Equation 1 defines the total strain rate, with respect to time, in relation to the elastic and creep properties of concrete. It is used in this section to investigate two cases of change of structural form.

In the first, a rotational restraint is imposed as a result of making two simply supported prestressed concrete beams continuous over the common support. In the second, a segmentally constructed prestressed unit of varying cross-sectional area is studied following the imposition of overall displacement restraint. This example serves to highlight some of the features introduced into the post-repair behaviour of the Koror-Babeldaob bridge[14] in the republic of Palau in 1996, as described later. This bridge collapsed six weeks after the remedial work was completed.

(i) Rotational restraint.
Fig. 11 shows two simply supported prestressed concrete beams made continuous over the common support after the two beams were subjected to their design loading; a uniformly distributed load in this case. Here the joining of the two beams creates a rotational restraint over the common support and leads to a time-dependent build up of bending moment there. Fig. 12 gives the details. The long-term steady state support bending moment is of the same magnitude as would have been predicted from an elastic calculation, for the same loads and prestress, and of continuous construction initially. The deformed shape, however, would be different.

(ii) Axial restraint.
The influence of imposing an overall axial displacement restraint on a prestressed concrete unit of sequential construction and varying cross-sectional area is investigated. This example may be considered as a simplification to some features of the retrofitting carried out in 1996 to the Koror-Babeldaob prestressed concrete box girder cantilever bridge in the Republic of Palau. In this segmentally constructed bridge the piers to the two cantilever arms were founded on piled foundations with raked piles[15], Fig. 13. The retrofitting was to overcome sagging at the cantilever tips at an age of 19 years and was brought about by several measures as indicated in Fig. 14. Firstly exterior prestressing tendons placed inside the box section, and passing over two deviator beams in each cantilever arm were used to lift the central section after first making it continuous at the crown hinge. At this time the two ends of the cantilevers were also subjected to axial jacking forces to add extra compression to compensate for earlier stress losses due to creep. From this time the bridge acted as continuous between the two piled abutments and was therefore also subjected to axial restraint. The 'hogging' moments in the central region would have enhanced the restraining forces by reason of deflecting the bridge profile further from the straight line shortest distance between the two abutments. This bridge collapsed six weeks after the remedial works had been completed.

Although the definitive reasons for the collapse have not been published, this example serves to demonstrate that the remedial work changed the behaviour of the bridge in a substantial way, from that associated with normal prestressed concrete in which the prestress acts as a 'follow-up' loading with little change in time, to a dramatically altered behaviour for which the prestressed segments lost stress rapidly due to the overall longitudinal displacement boundary constraint imposed, in addition to the rotational constraint at the crown section. The mixed boundary conditions of prestress forces locally on each segment and overall displacement control on axial deformations thus led to the dramatic behavioural changes, with a rapid destressing of the concrete that is not normally associated with prestressed concrete. The changes described here refer to the influence of creep only. Inclusion of shrinkage will bring about an even more rapid loss of stress than shown in the example of Fig. 11.

The figures and discussion of the next section describe the detailed nature of the behavioural changes caused by the overall displacement control imposed.

The example of Fig. 15 addresses the axial length change restraint in a simplified manner, while retaining the important geometrical features of the bridge, *viz.* the parabolic variation of cross-sectional area between the crown section and the abutments. Similar to the bridge, each segment of Fig. 15 is prestressed locally to a uniform stress (the same in each element) by a common proportion of prestressing steel. *(The prestressing tendons are assumed to be fully bonded to the surrounding concrete.)* Thereafter a range of different conditions has been studied; all relate to changes of concrete and steel stresses. They are:

(i) The normal prestressed concrete situation, i.e. no restraint to axial shortening.

(ii) The imposition of an overall length change restraint, to simulate at least approximately the making of the actual bridge monolithic at the crown hinge.

(iii) The application of an axial jacking force before applying overall displacement restraint between the two ends. This is to simulate the crown jacking forces applied to the actual bridge before monolithic construction was created.

(iv) The application of a short term temperature change to determine the distribution of thermal stresses in the axially restrained condition. These stresses could be expected to apply as a short-term variation to the time-varying stresses caused by creep, whenever a temperature change took place. Their magnitude would also relate to the temperature at which the bridge was made continuous at the crown section.

(v) The imposition of overall length change restraint for elements of the same cross-sectional area. This comparison allows the importance of the influence of the variable geometry to be assessed.

Table 2 sets out the details of the individual cases studied.

DISCUSSION

Fig. 1 shows the typical response of introducing moment continuity late in the life of a structure. At uniform temperature the steady state solution for the time-dependent moment variations is the same as the normal elastic solution for the same continuity applied initially. When temperatures are sustained but not uniform a different steady state solution is reached that is uniquely related to the form of the temperature distribution and the prestress applied.

Figs. 16 to 20 allow several important observations to be made for the example of Fig. 15 when conflicting boundary conditions (namely restriction to free axial movements) are imposed on a prestressed structure.

Overall displacement restraint is responsible for an immediate departure from normal prestressed behaviour; causing concrete to undergo a rapid loss of prestress. Comparison of Cases (A) and (E) in Fig. 16(a) show the very significant difference in concrete stress as creep occurs. In Case (E) the theoretical limit to the concrete stress is zero. The steel stress however shows no variation over the same period, while steel force is progressively transferred to the boundary constraints.

The variation of cross-sectional area with position creates another important feature. The restraint to overall length change results in an interchange between creep and elastic strains within the segments collectively, but not uniformly. The elastic strain rates, and hence the stress changes, are at all times distributed in the inverse ratio of the segment stiffnesses. This causes additional, and rapidly developing, stress relief (beyond that for relaxation at constant strain) thus causing tension in some of the more slender segments. Fig. 16(b), Case (B), shows this effect for each segment following the application of the length restraint while Fig. 16(c), Case (C), shows a similar effect to exist even after a compressive axial jacking force of P = 35 MN was applied before the length restraint was imposed. The more stocky segments (e.g. segment 10) experience behaviour that is more closely associated with normal prestress behaviour (compare Case (A) of Fig. 16(a)).

Comparison of Fig. 17(a), Case (A), with Fig. 17(b) , Case (B), reveal the dramatic changes to concrete stress that result from the combined influence of axial restraint and varying cross-sectional area, on a segment by segment basis, as creep develops. The separate influence of cross-sectional area may be seen by comparing Case (B), Fig. 17(b), with Case (E), Fig. 18.

The benefits of the axial jacking force applied before imposing the length restraint may be seen by comparing Case (C), Fig. 19, with Case (B) of Fig. 17(b). It is of interest to note here that whereas the initial effect of the jacking was to increase the stress in segment 1 by more than 5 MN/m^2 with the development of creep this difference progressively reduced to around 1 MN/m^2 and both solutions eventually revealed the development of tensile stresses. Clearly the most important parameter, controlling change of concrete stress, is that of the overall length restraint that was imposed.

Finally the segment stresses corresponding to a short-term uniform temperature change of 10^{o}C are displayed in Fig. 19, Case (D). Again, the highest stresses are seen to concentrate towards the more slender segments as a result of the non-uniform stiffness throughout the set of segments.

CONCLUSIONS

Concrete creep can be detrimental to structural performance, not only from the standpoint of serviceability, but from safety considerations also. Non-uniform temperatures cause differential creep and stress redistribution to occur in prestressed concrete structures. The consequences often lead to violations of the original design unless explicit recognition is paid to the long-term changes of stress as part of the design criteria. The self-equilibrating stresses due to thermal creep and thermal stresses of non-linear temperature crossfall will increase the maximum stress over the support section from that calculated from the bending moment. Research presented here shows that if the material constant "a" of the temperature function is less than 40^{o}C, the thermal creep will cause significant stress redistribution (see Fig. 10) and its effect on prestressed concrete structures should not be ignored. The thermal creep multiplier as specified in EC2 Part 3 implies "a = 62^{o}C" which is less than those from other researchers. If it is valid, the creep effect to the steady state stresses at the low end of the bridge temperature range should still be checked. A support moment of 40% larger than that of a elastic solution is still possible.

Imposed constraints will lead to stress changes in time. Flexural restraints, common to the creation of moment continuity, should be incorporated in the initial design even though they may be imposed late in the service life of a structure. Displacement restraints in the direction of the prestress should be avoided because they generate a conflict of boundary conditions between the follow-up loading required of the prestress from the tendons and the overall displacement constraints required for the structure. The result can be a rapid and serious time-dependent reduction in concrete prestress, leading to failure in tension, or more likely in shear as the result of removing the pre-compression required of a safe design.

The time-dependent stress reductions depicted in the example of Fig. 6, where tensile stresses are shown to develop in some prestressed concrete segments simply as a result of introducing an overall length restraint, suggests that a full and detailed analysis of the Koror-Babeldaob bridge is warranted to determine the most likely true effects of the dual restraints imposed on the structure by the remedial works. The overall length restraint would bring about similar stress reductions with time as displayed in the example of Fig. 6, while the external prestressing tendons introduced to cause uplift over the central region of the span (already made continuous at the crown joint) would exacerbate the rate of stress decay in the concrete by reason of trying to increase the length of the bridge profile. Failure of this bridge was inevitable eventually.

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