Problems in Designing Prestressed Segmental Concrete Bridges

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ABSTRACT

Prestressed segmental concrete bridge construction offers unique advantages over conventional methods of bridge construction. As a result, this relatively new method has gained wide popularity in the construction industry. However, as with any innovative technique, there are some design problems associated with these bridges that require attention. Experience has indicated that damage may occur at local details in these bridges. Some design problems that can be solved with the use of advanced analysis methods to obtain a better understanding of bridge behavior are discussed. The finite-element method, in conjunction with representative material models, is ideally suited for providing a better understanding of the structural behavior and for obtaining accurate response predictions. Two representative examples are analyzed and discussed to demonstrate how to use these procedures in the design process. Response predictions obtained from the application of these advanced analysis methods to segmental concrete bridges can be used as a basis for improving designs. Further examples are given, and finally the importance and implementation of such methods in routine design are briefly discussed.

A damaged local detail in a precast segmental bridge can adversely affect the stability or safety of the bridge as a whole. Therefore, special attention should be given to the design of these details. However, in some cases standard design methods may not sufficiently cover all critical conditions in these details. The objective of this paper is to emphasize that these problems can be solved with the use of advanced analysis methods during the design process.

As a result of the need for efficient, economical, and maintenance-free structural systems, structural designers optimize their designs by using structural components and materials as efficiently as possible. Precast segmental bridge technology is an example of such a development, by which construction costs have been reduced in many cases. These bridges consist of small prefabricated segments that are assembled and prestressed on site (1,2).

Despite the advantages associated with the structural efficiency and low maintenance of the structures, damages have been observed in some field applications. Little information is available on the long-term performance of these structures. A few examples of previous problems on these bridges are joints that failed, spalling and honeycombing of concrete, instabilities during construction, and various problems at local details.

In the following sections typical structural problems that have been encountered in precast segmental bridges are discussed. Two specific problems are then analyzed and discussed to illustrate the use of advanced design methods. Finally, the implications for future designs are examined.

STRUCTURAL PROBLEMS IN PRECAST SEGMENTAL BRIDGES

A large number of precast segmental bridges have been constructed over the past 10 years, and on the whole the construction method has proved to be economically viable. Nevertheless, a number of engineering problems that are unique to these types of bridges have been encountered (2). Some problems that can be resolved in the design stage are briefly discussed. Other problems that usually occur during construction are not covered, but they have been discussed elsewhere (2).

Concrete has spalled off or cracked on several of these bridges. Gerwick (3) and Lin and Redfield (4) reported that concrete was crushed at prestressing tendon anchors in the Columbia River crossing bridges. A large number of prestressing ducts in thin slabs also caused spalling on these structures. Casey (5) indicated that an overly complex cross section made it difficult to compact the concrete, and this led to honeycombing. Expensive remedial measures then offset much of the cost savings. In a box-girder bridge with external tendons, severe cracks developed in the anchor diaphragm, which resulted in closure of the structure to traffic (6). Additional vertical prestressing and extensive grouting were necessary to prevent further crack growth. A similar example is analyzed and discussed in more detail in the next section of this paper.

Cracks and thermal deformations occurred on a number of bridges while manufacturing and curing the segments. For example, large cracks developed in the segment wings of the Zilwaukee Bridge after curing, but the majority of these cracks closed after transverse prestressing was applied. Shrinkage and thermal expansion and contraction probably caused these cracks (7). The remaining large cracks were filled with epoxy, and additional prestressing was added to limit further cracking during operational use.

Creep and shrinkage deformations of concrete significantly affect the deflections of the structure. Thus the shapes of the segments have to be adjusted so that large secondary moments are not introduced when two adjacent sections of the bridge are joined together. This problem is especially troublesome when the two parts do not deflect by the same amount. Several research projects have focused on the time-dependent behavior of segmentally constructed bridges, and a number of numerical methods have been developed to predict the creep, shrinkage, and elastic deflections during the life of the bridge (8-11). These methods have been used with varying degrees of success. On the majority of bridges the actual deflections compared favorably with the predicted values, but discrepancies occurred on some projects (12). In cast-in-place bridges these differences can be remedied while casting the new segment. However, shims or special wet segments have to be used for precast bridges, but these methods may not provide a satisfactory solution.

Shear keys on the contact faces between segments provide shear strength and ensure that adjacent segments can be positioned accurately during construction. Epoxy bonding agents are usually applied on these surfaces to protect the prestressing tendons from corrosion, to provide additional shear strength, and to assist in placing the segments while erecting the bridge. On a number of earlier bridges the epoxy failed to reach the required strength before additional segments were added, and some of these shear keys were crushed (5,12,13).

Analysis of Local Distress

The large costs that are involved in constructing prestressed segmental bridges require that precautions be taken to prevent damages in these bridges, such as damaged bearings and delaminations and cracks in the concrete. Cracks should be prevented from forming or kept to a minimum because they might result in corrosion of the reinforcement and deterioration of the concrete. The complex geometries and material behaviors of these bridges differ substantially from those of normal building-type structures, and therefore conventional techniques may not apply in designing details.

One example is the design of anchor blocks, where standard design methods are based on the behavior of rectangular beams. If the geometry differs from this form, the calculated stress distributions will be incorrect and the design might be unconservative. Material properties also play an important role in the behavior of a structure. The engineer should use realistic and representative values and models for the strength, stiffness, and time-dependent properties of the materials. Often empirical data are not sufficiently accurate for complex structures. If there is any doubt about the characteristics of the concrete, representative tests should be performed to ensure that the design data are consistent with the material that will finally be used in the bridge. Further, it might be necessary to make additional adjustments during construction if the concrete properties change.

Two selected problems are considered in the following subsections to demonstrate how refined methods can be used to more closely approximate the actual deformations and stress distributions in a bridge. The objective is to indicate that these methods can provide realistic answers to complex problems in bridge design. The effect of sharp deviations in the prestressing tendons are first investigated. Then the creep and shrinkage deformations under dead loads and prestressing forces are calculated.

The specific details assumed in these analyses do not refer to any particular structure. The objective of this presentation is to provide an understanding of the problems in general, and to describe methods that can identify and clarify these problems during the design stages.

Spalling and Cracking of Concrete at Prestressing Tendon Locations

Stress concentrations generally occur at positions where prestressing tendons are sharply curved or at anchor locations. An analysis is presented of a torsion diaphragm that also serves as an anchor block for prestressing tendons. This diaphragm is located above the supports of a multispan continuous bridge. The tendons are sharply curved in this anchor diaphragm, and the concern is whether cracks might develop in this region.

Analysis

The anchor region of the bridge in Figure 1 is analyzed to determine whether cracks might develop from the stress concentrations around the tendons. The finite-element method is used for this purpose. No details will be given regarding this method, and readers are referred to the available texts on this topic (14,15). What assumptions have been made in these analyses, and how the prestressing forces have been incorporated, are discussed. The results of these analyses are discussed later in this subsection.

Two different two-dimensional plane stress analyses are performed on the concrete box girder. These analyses are for an assumed cross-sectional detail at the anchor diaphragm, as shown in Figure 2, and for a corresponding long section of the anchor region, as shown in Figure 3. These sections are assumed to be symmetrical about their centerlines. Figures 4 and 5 show the finite-element meshes and the boundary conditions that are adopted to represent the symmetry.

Concrete is a nonlinear material in which deformations are not proportional to applied loads and strains are not proportional to the applied stresses (16). This is especially true at high compressive stresses that are close to the failure strength of the concrete. But in tension the stress-strain response is approximately linear. The purpose of the present analysis is to predict the formation of

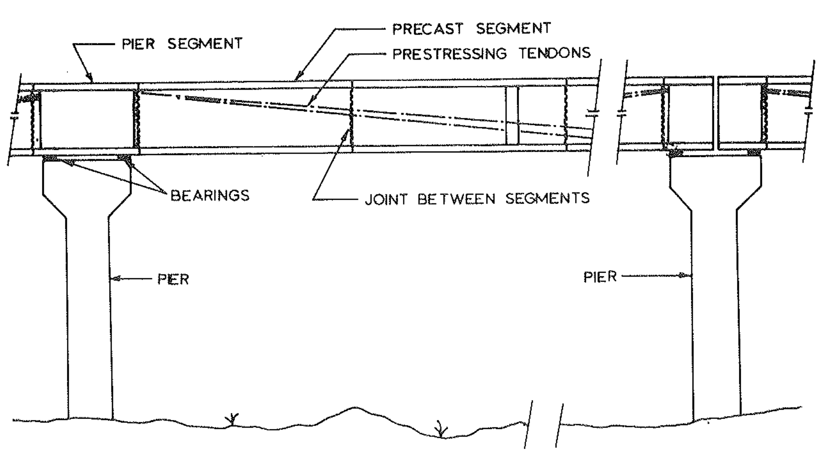

FIGURE 1 Typical span.

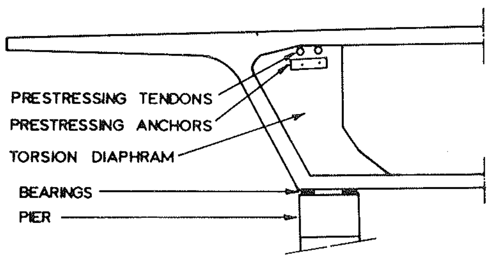

FIGURE 2 Details of cross section at support.

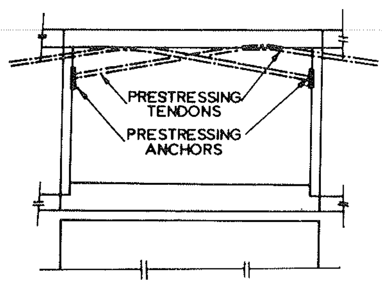

FIGURE 3 Details of long section through pier segment.

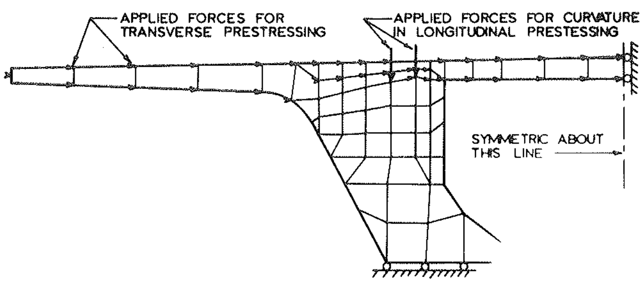

FIGURE 4 Finite-element mesh for cross-section analysis.

cracks in the diaphragm caused by the tensile stresses that occur at the locations where the prestressing tendons are sharply curved. Because tensile cracks form in concrete at relatively low stresses, a linear analysis provides sufficiently accurate answers.

Reinforcing steel introduces additional stiffness and strength in concrete, and in the finite-element analysis of a reinforced-concrete structure the effect of reinforcement may be conveniently modeled with the use of the smeared-reinforcement concept (17). The finite-element analysis program used for the present study includes this capability. But light reinforcement usually included in the torsion diaphragm under consideration does not significantly affect the crack predictions, and thus it may be ignored.

In both cases a thin section of the anchor block is analyzed so that the stresses do not change significantly through the thickness. Thus it is not necessary to perform a three-dimensional analysis; a two-dimensional (plane stress) approach is considered to be adequate.

For the purpose of this example, a bridge is considered that is prestressed in both the longitudinal and the transverse directions. Transverse prestressing is applied in the top slab of the box section by bonded pretensioned reinforcement. Prestressing in the longitudinal direction is applied

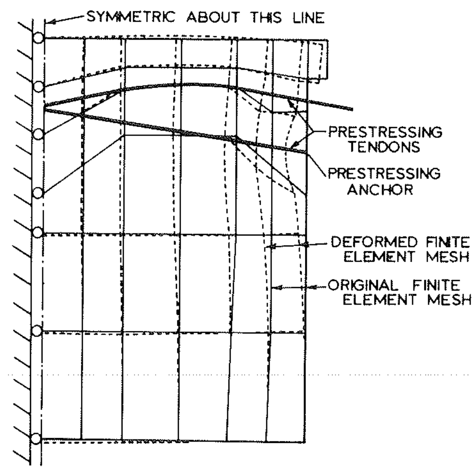

FIGURE 5 Finite element for long section analysis.

with posttensioned bonded tendons. The longitudinal prestressing tendons are sharply curved in the anchor block. This introduces large vertical forces on the cross section, which are taken into account in both analyses.

In the analysis the transverse prestressing forces are applied as nodal forces on the finite-element nodes. Equivalent forces are calculated to represent the prestressing as accurately as possible. Because the pretensioned tendons are fully bonded to the concrete, it can be assumed that the total prestressing force is transmitted to the concrete by uniformly distributed bond stresses. These forces act along the tendon, and all forces should therefore be applied along that line. This restriction would dictate where the finite-element nodes should be placed. As an alternative, equivalent nodal forces can be calculated to represent the effect of the prestressing force at the boundary of each finite element. Displacement interpolation functions for the finite elements provide the basis for allocating the representative forces to the two or three nodes on each boundary (14).

The vertical forces that result from the curved longitudinal prestressing tendons are applied as distributed forces on the nodes. The displacement interpolation functions are again used to convert these distributed loads to equivalent nodal loads.

Experimental results indicate that the failure strength of concrete is greatly influenced by the stress level. Kupfer et al. (18) tested concrete under biaxial compression and compression and tension. They found that under biaxial compression, the compressive strength of the concrete exceeds the usual uniaxial strength. However, when concrete is subjected to tensile and compressive stresses in two perpendicular directions, the tensile strength of the material is less than the tensile strength under uniaxial stresses. These phenomena are best represented by the failure envelope in Figure 6a. This envelope represents the biaxial stress combinations under which concrete fails. In Figure 6a the two principal stresses at a point (σ_1 and σ_2) are normalized with respect to the uniaxial compressive strength of the concrete. If the stresses lie outside the failure envelope, it can be expected that the concrete will have failed. The mode of failure depends on the type of stresses. If both stresses are compressive, the concrete will be considered crushed; if one or both of the stresses are tensile, the concrete will be cracked.

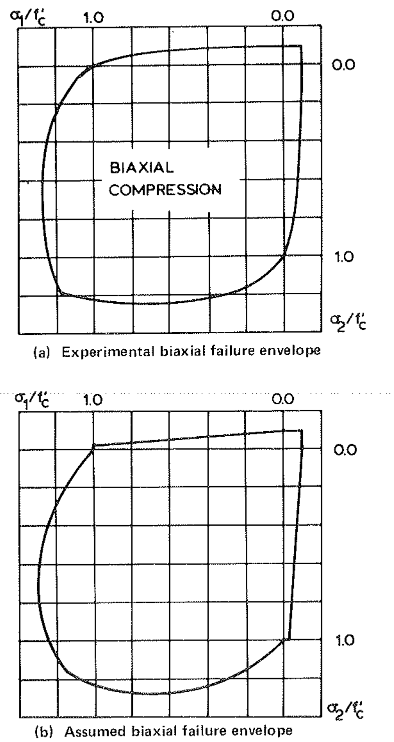

FIGURE 6 Failure envelopes of biaxially loaded concrete.

Several numerical methods have been proposed to represent these experimental results (17,19). A simplified model is used for this analysis (20). It consists of the criteria for the biaxial compression region (21), a bilinear model in the tension-compression region, and a constant strength in the biaxial tension region. This postulated failure envelope is shown in Figure 6b.

Therefore, the proposed strategy to determine whether cracks might form in the concrete is to analyze the details with the use of a linear elastic method, and then to compare the predicted stresses with the established failure criterion for concrete.

Discussion of Results

The results of the finite-element analyses are presented in Figures 5, 7, and 8. The deformed shape (indicated by broken lines in Figure 5) is exaggerated to indicate the effect more clearly. Note the effect of the boundary conditions, as well as the influence of the prestressing forces on the deformation. Figure 7 shows the maximum principle stress contours. In this figure full lines represent tensile stresses and broken lines represent compressive stresses. It is particularly important to notice the large tensile stresses above the prestressing tendons in the cross section.

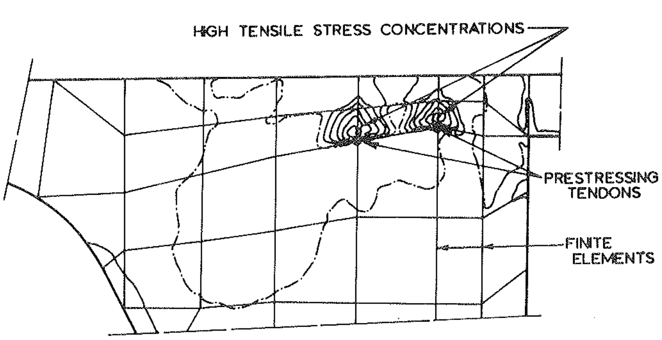

FIGURE 7 Results of cross-section analysis.

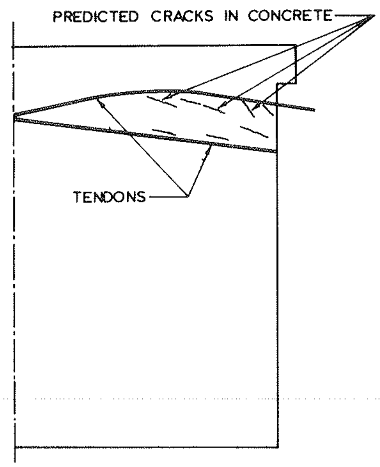

FIGURE 8 Predicted crack location in long section.

The directions of the principal stresses and their relative magnitudes have been obtained from the finite-element analyses. Of particular interest are those locations where large tensile stresses occur in conjunction with the compressive stresses. This is the case at positions above and behind the tendons anchors (see Figure 8). If such an analysis is performed while designing a bridge, the engineer can take precautions to prevent these cracks from forming or to prevent them from extending if they do form.

Creep in Concrete

The time-dependent behavior of segmentally constructed precast and cast-in-place bridges is of great importance in the design and construction of these structures. Sections of a structure can be deformed or overstressed if the time-dependent deformations are not predicted correctly. Therefore, it is necessary to use accurate models for the structure as well as for the material so that they represent the actual bridge. These deformations are used to calculate the prestressing forces and to determine the initial shape of the bridge.

A number of numerical models are available to calculate these long-term deformations. The methods of the American Concrete Institute (ACI) (22) and the Comite Europeen du Beton (CEB) (23) are widely used. However, these standard methods for predicting creep and shrinkage have been questioned for their accuracy. Bazant and Panula (24) recently suggested a model for better predictions. Also, attention should be paid to variations in the local climatic, material, and environmental conditions in computing long-term deformations.

A method for predicting the long-term shortening of a multispan bridge with finite elements is described. The method offers advantages in (a) computing accurate material models for long-term deformations, and (b) allowing flexibility for better structural representation.

It has been shown that the long-term deformation of concrete can be predicted more accurately if some tests are performed on the concrete (24). Therefore, it is possible to reduce the uncertainty in the creep and shrinkage predictions. The method described in this subsection is well suited for incorporating such test results in the design of such a structure.

Numerical Model for Creep

Bazant (25) and Anderson (26) proposed a method for calculating long-term creep and shrinkage deformations with the finite-element method. The method for the uniaxial creep of concrete is based on the following assumptions.

1. The total deformation of the concrete at time t is the sum of those deformations that result from the applied stress [$\epsilon_\sigma(t)$] and those deformations that are independent of the applied stress [$\epsilon^0(t)$]:

$$\epsilon(t) = \epsilon_\sigma(t) + \epsilon^0(t) \tag{1}$$

2. The stress-related deformations can be divided into the instantaneous and the long-term deformations, such that

$$\epsilon_\sigma(t) = \left\{[1/E(t')] + C(t;t')\right\}\sigma = \sigma\, J(t;t') \tag{2}$$

where

$\sigma/E(t')$ = instantaneous deformation at age t',
$\sigma C(t;t')$ = creep deformation, and
$J(t;t')$ = total time-dependent compliance.

Bazant indicated that a time-step integration method can be developed from these basic assumptions to calculate the time-dependent deformations under varying stress conditions. This material law can be written as

$$\Delta\epsilon = (\Delta\sigma/E'') + \Delta\epsilon'' + \Delta\epsilon^0 \tag{3}$$

where

$\Delta\epsilon$ = change in strain during a time step,
$\Delta\sigma$ = change in stress during the time step,
$1/E''$ = time-dependent compliance,
$\Delta\epsilon''$ = incremental inelastic (creep) strain, and
$\Delta\epsilon^0$ = non-stress-related strain increment.

This model for creep has been incorporated in a finite-element program for the analysis of plane stress and plane strain specimens.

Finite-Element Model of Prestressed Span

The center span of a five-span continuous bridge with posttensioned tendons is analyzed to determine time-dependent deformations. It is assumed for the purpose of this analysis that the tendons are located outside the concrete and that they are only attached to the concrete at the anchors and at two intermediate diaphragms, where the tendons change direction (see Figure 1). The main objective is to calculate the shortening of this span as a result of creep and shrinkage in the concrete so that the expansion joints and bearings can be designed accordingly. Therefore, because the end spans are far removed from this span, and because only long-term effects at the expansion joints will be calculated, it can be assumed that the ends of this span are prevented from rotating.

Only dead-load effects are considered, and because these do not introduce torsional or any other nonplanar effects, plane stress finite elements can be used to model the concrete box girder. Also, it is assumed that the shear lag effect in the flanges of the box girder will not affect the long-term deformations, and that this effect can therefore be ignored in this analysis. Elements of different thicknesses are used. These correspond to the total thickness of concrete on the particular level, and no special provision is made for the hollow box. One-dimensional elements with axial stiffness are used to represent prestressing tendons. Initial strains in the tendons introduce the prestressing forces in the bridge. The advantage of using this approach, rather than applying equivalent external forces at finite-element nodes, is that prestressing losses caused by concrete creep and shrinkage are taken into account through deformations in the bridge. The additional stiffness of the tendons is also taken into account.

The initial strains in the prestressing tendons are calculated as follows. The bridge span is analyzed with linear elastic material properties that represent the stiffness of the concrete at the time of stressing. Unit forces are consecutively applied at the finite-element tendon anchors. The force P_{ji} in tendon section i caused by the unit force at anchor j is obtained from these analyses:

$$f_i = f_i^0 - (P_{1i}f_1^0 + P_{2i}f_2^0 + \ldots + P_{ni}f_n^0) \tag{4}$$

if there are n tendons. These forces, with the additional forces introduced by the dead loads, can be written as

$$[f] = \left\{[I] - [P]\right\}[f^0] + [f^{DL}] \tag{5}$$

where

$[f]$ = vector that contains the prestressing forces in the tendons (f_i) at the time of stressing,
$[I]$ = n by n identity matrix,
$[P]$ = stress influence matrix with entries P_{ji},
$[f^0]$ = initial prestressing force vector, and
$[f^{DL}]$ = vector that contains the forces in the tendons from the dead loads.

The initial tendon forces can then be obtained from

$$[f^0] = \left\{[I] - [P]\right\}^{-1}[f - f^{DL}] \tag{6}$$

The corresponding initial strains are calculated from these initial stresses.

Bazant (25) suggested that the total time-dependent compliance in Equation 2 can be expressed as a Dirichlet expansion such that

$$J(t;t') = a_0 + b_0(t')^{-n_0} + \sum_{m=1}^{N}[a_m + b_m(t')^{-n_m}]\left\{1-\exp[-(t-t')/\tau_m]\right\} \tag{7}$$

where a_i, b_i, and n_i are experimentally determined coefficients, and τ_i are retardation times. Also, the shrinkage strain can be expressed as

$$\epsilon_{sh}(t) = \epsilon_{sh\infty}\, k_d\, \sqrt{(t-t')/[(t-t')+\tau_\alpha]} \tag{8}$$

where $\epsilon_{sh\infty}$, k_d, and τ_α are coefficients that depend on the concrete properties.

Note that the particular method does not merely require one single creep coefficient, but several coefficients (as shown in Equation 7) are needed to fully represent the time-dependent behavior of the concrete. It is important that the correct coefficients are used in this model. These coefficients can be obtained either from experimental data or by calibrating Equation 7 to available creep curves.

In this analysis data from previous tests are used to illustrate the method (27,28). These coefficients are adjusted for the particular concrete compressive strength of approximately 5,000 psi.

The finite-element mesh and the deformed shapes

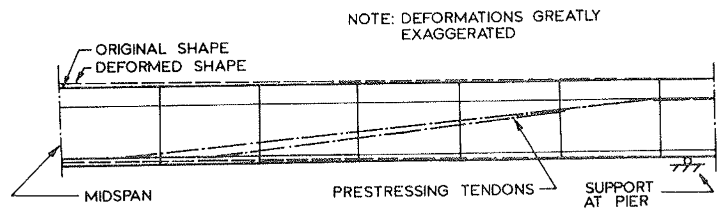

FIGURE 9 Configuration used for creep analysis.

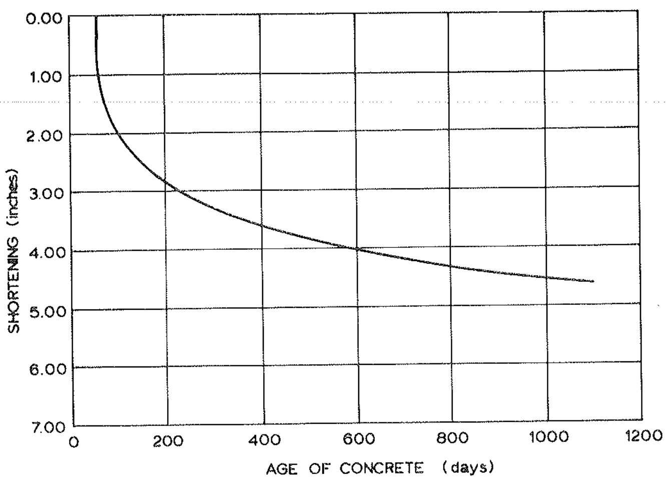

FIGURE 10 Long-term shortening.

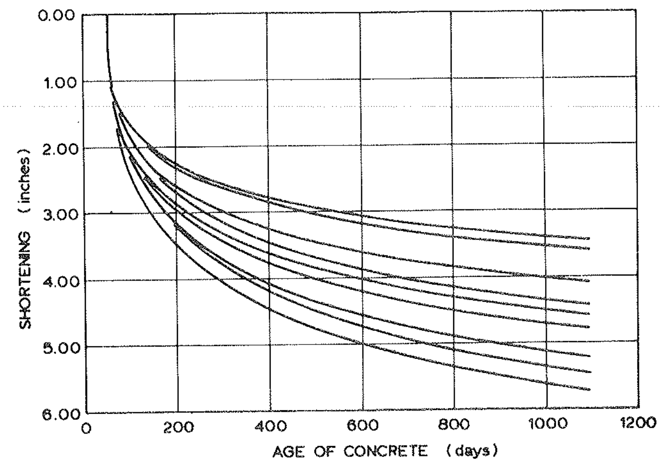

FIGURE 11 Variability in the shortening.

are shown in Figure 9, and the shortening of the bridge is shown in Figure 10. Note that the deformations initially increased rapidly, but then the rate of change in deformations decreased. These deformations only take elastic, creep, and shrinkage effects into account, but short-term effects of temperature changes and horizontal traffic loads should also be considered in the final calculations.

Variability of Concrete Properties

Because of variations in the way concrete is batched and cured, and because of variations in aggregate properties, composition of different mixes, and environmental conditions, concrete from different batches have different stiffness, creep, and shrinkage properties. This is so even for concretes with similar strengths. Therefore, it is not possible to accurately predict these properties with empirical expressions. Because of these uncertain properties the structural response is also uncertain. An uncertainty analysis should be performed to explore how the response of the structure is affected by the uncertainty in the material properties.

The present case study is extended in this subsection to illustrate how the shortening of the bridge is affected by the variability in the long-term concrete properties. It has been found that the predicted stiffness, creep, and shrinkage properties have standard deviations of 20, 22, and 33 percent, respectively, with respect to the average material properties (24). To study the behavior of the bridge under these variable conditions, a number of analyses can be performed with different material properties. For this example, three different values are assigned to each of the material properties. In terms of the average (predicted) value (α) and its standard deviation (σ_α), these three values are ($\alpha - \sigma_\alpha$), α, and ($\alpha + \sigma_\alpha$). The predicted response for nine selected cases are presented in Figure 11, where different combinations of each of the three material properties (stiffness, creep, and shrinkage) are used.

Note the wide range in the predicted shortening. It is clear that it is not possible to accurately predict the shortening of the bridge; the bearings and other supporting structures can also not be designed to close tolerances, but this uncertainty should be taken into account.

The finite-element method, as previously described, is ideally suited for this analysis, where the material properties have to be incorporated in the actual design. The example indicates that it is important to determine whether the deformations are sensitive to variations in the material properties.

APPLICATION OF REPRESENTATIVE CONCRETE MODELS TO BRIDGE ANALYSIS

Current design codes incorporate many safety margins that have been built up through many years of experience. However, design conditions only reflect the sometimes limited experience of previous designs and experiments, and usually it is not possible to extend these standards beyond those limits on which they are based. Therefore, in applying innovative design concepts that do not adhere to these limits, alternative methods have to be explored to verify the designs.

Research has been conducted on the fundamental behavior of concrete when it is subjected to various load and environmental conditions. This information reflects the actual material response, whereas current concrete design codes tend to reflect the properties of members or structural systems. Although these research results may not be used directly in the design process, finite-element programs have been developed in which they have been incorporated. As was illustrated in the previous examples, these methods can be used to study the nonstandard details to ensure that the material strengths are not exceeded.

Apart from these cases, several more examples can be given where standard building codes would not be directly applicable. Simplifying assumptions are often implicitly incorporated in design rules and can be justified for smaller members. In larger applications these simplifications may introduce inaccuracies that might not be compensated for by the redundancies in the structure. A few examples are discussed in the following paragraphs.

Various theories have been proposed for designing shear and torsional reinforcement in beams, but tests indicate that current design expressions may be unconservative for certain conditions (16). More advanced material behavior should therefore be applied in the analysis of deeper beams and box girders. In these instances the members resemble plane stress conditions for which biaxial properties should be considered to more accurately incorporate the failure and shear transfer behavior of concrete.

Current design methods should be improved in accounting for the effects introduced in statically indeterminate structures by prestressing and thermal gradients at ultimate loads. Approximations would introduce inaccuracies that can lead to cracking and crushing under design conditions. The use of incorrect creep and shrinkage data in the design can likewise cause inaccuracies in deformation predictions, which would result in distress.

Local details also deserve special attention, especially where high stress concentrations can be expected. Tendon anchors, abrupt directional changes in prestressing tendons, supports, joints, holes, and cavities can also introduce high stress concentrations. Again, current design methods should be improved for these cases.

SUMMARY

In this paper a brief discussion is presented of the problems that have occurred on some precast segmental bridges. Some of these problems may be attributed to conventional design methods that may not correctly incorporate the complex behavior of these bridges. Two typical problems are analyzed and discussed to show how the finite-element method can be used in conjunction with a representative material model to study specific details. These material models can provide more accurate information on the serviceability and ultimate limit strength of these structures. Therefore, such methods should be incorporated in the design process of these bridges.

REFERENCES

1. W. Podolny and J. Muller. Prestressed Concrete Segmental Bridges. Wiley, New York, 1982.
2. D.J.W. Wium and O. Buyukozturk. Precast Segmental Bridges--Status and Future Directions. Journal of Civil Engineering for Practicing and Design Engineers, Vol. 3, No. 1, 1984, pp. 59-79.
3. B.C. Gerwick. Construction Problems and Solutions: I-205 Columbia River Bridge. Proc., Segmental Concrete Bridge Conference, Prestressed Concrete Institute, Kansas City, Mo., 1982.
4. T.Y. Lin and C. Redfield. Some Design Issues for American Constructors. Proc., Segmental Concrete Bridge Conference, Prestressed Concrete Institute, Kansas City, Mo., 1982.
5. G.J. Casey. Zilwaukee Bridge Construction. Proc., Segmental Concrete Bridge Conference, Prestressed Concrete Institute, Kansas City, Mo., 1982.
6. R.J. Woodward. Cracks in a Concrete Bridge. Concrete, Vol. 17, No. 7, 1983, pp. 40-45.
7. Early Decisions and Delays on the Zilwaukee Michigan Bridge Project. U.S. General Accounting Office, 1983.
8. S. Kashima and J.E. Breen. Construction and Load Tests of a Segmentally Precast Box Girder Bridge Model. Res. Report 121-5. Center for Highway Research, University of Texas, Austin, 1975.
9. K.N. Shiu, J.I. Daniel, and H.G. Russel. Time-Dependent Behavior of Segmental Cantilever Concrete Bridges. Final Report. Illinois Department of Transportation, Springfield (in preparation).
10. M.K. Tadros, A. Ghali, and W.H. Dilger. Long-Term Stresses and Deformations of Segmental Bridges. Prestressed Concrete Institute Journal, Vol. 24, No. 4, 1974, pp. 67-87.
11. S.F. van Zyl and A.C. Scordelis. Analysis of Curved, Prestressed Segmental Bridges. ASCE, Journal of the Structural Division, Vol. 105, No. ST11, 1979, pp. 2399-2417.
12. A.C. Harwood. I-205 Columbia River Bridge Design and Construction. Proc., Segmental Concrete Bridge Conference, Prestressed Concrete Institute, Kansas City, Mo., 1982.
13. E.A. Lamberson and J.M. Barker. Kishwaukee River Bridges. Concrete International, Vol. 3, No. 3, 1981, pp. 93-101.
14. K.J. Bathe. Finite Element Procedures in Engineering Analysis. Prentice-Hall, Englewood Cliffs, N.J., 1982.
15. O.C. Zienkiewics. The Finite Element Method. McGraw-Hill, New York, 1982.
16. R. Park and T. Paulay. Reinforced Concrete Structures. Wiley, New York, 1975.
17. Finite Element Analysis of Reinforced Concrete. ASCE, New York, 1982.
18. H. Kupfer, H.K. Hilsdorf, and H. Rüsch. Behavior of Concrete Under Biaxial Stresses. Journal of the American Concrete Institute, Vol. 66, No. 8, 1969, pp. 656-666.
19. W.F. Chen. Plasticity in Reinforced Concrete. McGraw-Hill, New York, 1981.
20. O. Buyukozturk and T.-M. Tseng. Behavior of Concrete in Biaxial Cyclic Compression. ASCE, Journal of Structural Engineering, Vol. 110, No. 3, March 1984, pp. 461-476.
21. K.J. Willam and E.P. Warnke. Constitutive Model for the Triaxial Behavior of Concrete. Paper III-1. Presented at Seminar on Concrete Structures Subjected to Triaxial Stress, International Association of Bridge and Structural Engineers, Bergamo, Italy, May 17-19, 1974.
22. Building Code Requirements for Reinforced Concrete. Report ACI-318-77. American Concrete Institute, Detroit, 1977.
23. CEB-FIP Model Code for Concrete Structure. Federation International da la Precontrainte, Wexham Springs, Slough, England, 1978.
24. Z.P. Bazant and L. Panula. Creep and Shrinkage Characterization for Analyzing Prestressed Concrete Structures. Prestressed Concrete Institute Journal, Vol. 25, No. 3, 1980, pp. 86-122.
25. Z.P. Bazant. Mathematical Models for Creep and Shrinkage of Concrete. In Creep and Shrinkage in Concrete Structures (Z.P. Bazant and F.H. Wittman, eds.), Wiley, New York, 1982, pp. 163-256.
26. C.A. Anderson. Numerical Creep Analysis of Structures. In Creep and Shrinkage in Concrete Structures (Z.P. Bazant and F.H. Wittman, eds.), Wiley, New York, 1982, pp. 259-303.

27. Z.P. Bazant and S.T. Wu. Dirichlet Series Creep Function for Aging Concrete. ASCE, Journal of the Engineering Mechanics Division, Vol. 99, No. EM2, 1973, pp. 367-387.
28. R.D. Browne. Properties of Concrete in Reactor Vessels. Paper 13. Proc., Conference on Prestressed Concrete Reactor Pressure Vessels, Institution of Civil Engineering, London, England, 1967, pp. 11-113.

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Edge-Stiffening Effect of New Jersey Barrier Walls on Cantilever Slabs

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ABSTRACT

The policy of using New Jersey type barrier walls along all major highways has been endorsed by the Ontario Ministry of Transportation and Communications. Consequently, new bridges and deck rehabilitations have massive barrier walls along outside edges of bridge decks. These barriers act as edge stiffening for cantilever slabs and have a significant effect on the distribution of live load on cantilever slabs. The current code specifications for the design of concrete cantilever slabs were established for slabs with rigid supports and with no edge stiffening. These specifications are conservative when edge stiffening is present. The load distribution of a typical cantilever slab supported by an exterior longitudinal girder was investigated by using three-dimensional finite elements. The study considered various edge-stiffening conditions and varying flexibility of longitudinal deck support. The results are compared with the methods given in the Ontario Highway Bridge Design Code and with other simplified methods. As a result of the enhanced load distribution, a significant potential saving in the quantity of cantilever reinforcing steel and in the cost of deck rehabilitation can be realized.

During the past few decades the nature of highways and vehicular traffic has changed rapidly. The highway system has developed with the objectives of providing for increased traffic volumes, increased truck loads, faster speeds, and greater safety. The vehicular traffic, in particular truck traffic, has dramatically changed in size and weight. As the heavy trucks have become more numerous and traffic has become more congested, a greater need to confine out-of-control trucks has arisen. Consequently, the nature of the restraining elements, which are designed to keep trucks within their right-of-way, has also changed.

Initially, the railings, parapet walls, or barrier walls were of simple form, consisting of a post and railing type. The initial use of wood gave way to the stronger materials of steel and concrete. However, as truck size increased, the post and railing type were not sufficient to resist collision loads and could not redirect out-of-control trucks back onto the highway. Consequently, the province of Ontario adopted a standard barrier wall that consisted of a continuous reinforced-concrete barrier wall. The typical barrier wall used for controlled-access highways is shown in Figure 1. This barrier wall is 450 mm wide at the base and just more than 1 m in height, with a total mass of 760 kg/m. Also shown is a barrier wall with railing that is used for roads with pedestrians. These massive barrier walls are considered a restraint mechanism and are used to redirect traffic, but not in a structural sense.

In Ontario a popular form of bridge construction is the concrete slab on longitudinal concrete or steel girders. Economically, it is advantageous to minimize the number of girders; consequently, the use of a cantilever slab is common. A typical cross section of a recently designed continuous steel box-girder bridge is shown in Figure 2. The design of the cantilever is governed by (a) dead loads, (b) vertical live loads, and (c) horizontal collision loads. The dead-load effects are secondary compared to the live-load effects. The ratio of factored live-load effect to factored dead-load effect is approximately 2.5 to 3.5 for a cantilever span of 1.5 m.

Current design specifications do not take into account the presence of these massive barrier walls. The design specifications have not kept pace with the development of the barrier walls and their structural effect on the design of the supporting slab. The effect of the presence of continuous concrete barrier walls on the design of the supporting cantilever slab is investigated. The presence of barrier walls affects only the distribution of vertical live loads and collision loads. Dead-load effects are not altered by the presence of barrier walls.

CODE SPECIFICATIONS FOR CANTILEVER SLABS

Current codes have been developed so that the canti-