Computer Analyses of Time-Dependent Behavior of Continuous Precast Prestressed Bridges

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Synopsis: Design and construction of bridges composed of simple-span, pretensioned girders made continuous for composite dead and live loads have become widespread. The design of these structures in the United States has been generally based on the procedure outlined in "Design of Continuous Highway Bridges with Precast, Prestressed Concrete Girders," published by the Portland Cement Association (PCA) in 1969. Although existing bridges designed by this procedure are generally performing well, it is believed that this method may not accurately predict the true behavior of these structures.

One of the major uncertainties in the design of these structures is prediction of positive and negative moments in the cast-in-place connections at the piers. This uncertainty is due to the different loading and construction stages, time-dependent effects, and details used to make the connections.

To resolve such uncertainties, Construction Technology Laboratories, Inc. has conducted an analytical study under the sponsorship of the National Cooperative Highway Research Project 12-29. The objective was to develop guidelines for more rational design of the continuity connections.

This paper summarizes results of an extensive parametric study to consider the effects of (1) construction sequence including simple span behavior before and continuous behavior after casting the deck and diaphragms, (2) time-dependent behavior including concrete creep and shrinkage, and steel relaxation, (3) live load applied at any stage of service life, (4) cracking resulting from both positive and negative moment including "tension stiffened" stress-strain relationships for reinforcement, and (5) closing of cracks when combined dead load plus time-dependent moments are reversed by application of live loads.

Keywords: bridges (structures); computer programs; continuity (structural); cracking (fracturing); creep properties; girders; loads (forces); moments; precast concrete; prestressed concrete; pretensioning; shrinkage; stress relaxation; structural analysis

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INTRODUCTION

Background

Application of precast, prestressed girders to bridge construction started in the United States in the early 1950's. Use of pretensioned I-girders with cast-in-place concrete decks grew rapidly. Until the early 1960's, bridges built with pretensioned I-girders and cast-in-place concrete deck were designed as simply supported spans. However, longitudinal reinforcement placed in continuous deck slabs above the piers provided negative moment capacity. Therefore, these I-girders could be considered as partially continuous for negative moments at the piers. The degree of continuity depends on the time-dependent effects and the positive and negative moment connection details provided at the piers.

In a pretensioned member, prestress will usually cause the member to camber. If the member is simply supported, the ends of the member will tend to rotate, as shown in Fig. 1(a). When members are made continuous through the deck and pier diaphragms, the ends of the pretensioned girder are restrained from rotating. As a result, a positive restraint moment, as shown in Fig. 1(b) may occur at the pier. Positive moment also occurs at the piers when alternate spans have live loads. Reinforcement for positive moment connection is designed for the summation of positive moment due to time-dependent effects and live load application. Construction of the positive moment connection detail is generally expensive and time consuming.

In 1961, the Portland Cement Association (PCA) conducted an experimental research program on this type of bridge (1). The research program studied the influences of creep in the precast girders and differential shrinkage between the precast girders and the cast-in-place deck slab on continuity behavior after an extended period of time. As a result of these studies, procedures were developed for design of the positive moment connection and the negative moment reinforcement over supporting piers (2).

There are several uncertainties associated with the PCA procedures. Some of the uncertainty stems from the simplifying assumptions made in the PCA procedures. One assumption is that girder concrete and deck concrete have the same creep and shrinkage properties. This would not generally be the case, particularly if the sequence of construction results in significantly different ages between the girder, diaphragm, and deck concrete. Different concrete mixes and curing conditions for girder, diaphragm, and deck concretes also cause differences in creep and shrinkage properties. Also, for the PCA simplified analyses, the continuity connections are considered to have zero length and to be fully rigid. Full continuity is assumed in calculation of live-load positive and negative moments. The actual connections have finite lengths and rotational stiffnesses. The moment of inertia of the reinforced concrete section at the connection after cracking from either positive or negative moment will be significantly lower than the prestressed girder section. In addition, when positive restraint moment from time-dependent effects causes cracking in the diaphragm concrete, these cracks must close before the full section becomes effective for negative live load moment.

NCHRP Project 12-29

Since the PCA research was completed over 20 years ago, there have been significant advancements in understanding of material behavior and in methods of analysis. Therefore, CTL is conducting NCHRP Project 12-29 to investigate the behavior of precast, prestressed girders made continuous and to develop guidelines for determining design moments commensurate with the degree of continuity developed at the piers.

From a questionnaire survey carried out as a first task to Project 12-29, it was learned that there is significant variation in application of this type of bridge design by various state Departments of Transportation. This is particularly true with respect to the positive moment design of the continuity connection. Current practice includes:

1. Designing and providing positive moment reinforcement or using standard details at the pier connection and considering negative moment continuity for reduction of live load positive moment near midspan.
2. Designing and providing positive moment reinforcement or using standard details but ignoring negative moment continuity for reduction of live load positive moment near midspan.
3. Providing no positive moment reinforcement but considering negative moment continuity for reduction of live load positive moment near midspan.
4. Providing no positive moment reinforcement and ignoring negative moment continuity for reduction of live load positive moment near midspan.

Another initial task of Project 12-29 was to develop and verify analytical procedures used to determine the degree of continuity and resulting moments. Descriptions of the computer program used in the project and its capabilities are given in the next section. Comparisons of analytical results and test observations for 1/2-scale bridges are given in Ref. 3.

The next major task of Project 12-29 was to carry out extensive computer analyses for a parametric study of the effects of various amounts of continuity reinforcement, different construction sequences, and variation in time-dependent material properties for concrete and prestressing steel in bridges with a range of girder type, span, and spacing. This paper summarizes the results of these computer analyses. The primary factor used to evaluate and compare results was the response to live load applied at various stages of service life. Implications of parametric study results on design procedures for this type of bridge are also discussed in the paper.

COMPUTER ANALYSES OF TIME-DEPENDENT RESPONSE

Program Description

Time-dependent deformations and restraint moments induced in multispan bridges built of prestressed girders made continuous were studied using a modified version of computer program PBEAM (4). This program, developed by C. Suttikan, is capable of analyzing composite prestressed concrete structures of any cross-sectional shape having one axis of symmetry. The program accounts for the effects of nonlinearity of stress-strain responses of materials and variations with time of strength, stiffness, creep, and shrinkage of concrete, and relaxation of steel. A step-by-step method is used in the time-dependent analysis with a tangent stiffness method implemented for solving nonlinear response.

Precast, prestressed bridge girders with composite cast-in-place decks are modeled using a discrete element method as developed by Hays and Matlock (5). Element deformations and forces are estimated by analyzing stress-strain relationships of a series of rectangular fibers distributed over the depth of the cross-section. It is assumed that strain in each fiber is constant at the centroidal axis of the fiber and strain distribution varies linearly through the depth of the section. For each time step, equilibrium at each element is maintained by determining the time dependent stress corresponding to the level of strain in each fiber. Stress multiplied by area is summed over all fibers and force equilibrium is checked. If necessary, the strain distribution is adjusted and the process is repeated until forces balance.

The A.C.I. Committee 209 (6) procedures to estimate the time variation of compressive strength, creep, and shrinkage of concrete and relaxation of prestressing steel are used in the PBEAM computer program. The rate of creep method and method of superposition are available in the program to account for the effect of concrete creep on the distribution of stresses in a section. The method of superposition was used for the studies conducted in this project.

The PBEAM computer program allows unlimited flexibility in analyzing various construction sequences and live load applications. The analysis accounts for construction sequence such that the simple span behavior of the girder before casting of deck and diaphragm, and continuous behavior thereafter are correctly modeled. Casting of deck and diaphragm can be done at any girder age and in any sequence. Live load can also be applied at any stage of service life and in any configuration. The program can therefore be used to investigate behavior under a wide variety of conditions likely to be encountered in actual use of this type of bridge.

The program's capabilities also allow a realistic analysis of the influence of diaphragm cracking on the behavior of this type of bridge. Depending on stress level and time-dependent material properties, the program accounts for cracking of girder and/or deck concrete under positive or negative moments. For each time step of the analysis, the program stores the stress-strain relationship in every fiber of each element. These stored conditions serve as the starting point for the behavior calculated for the succeeding time step. In this way, the program can follow both crack development and crack closing. This analytically models the situation shown experimentally in Ref. 1, in which cracks, which had opened at the bottom of the diaphragm under the influence of positive restraint moments, were closed by the negative moment induced upon application of live load. With initial application of live load, the diaphragm crack is open and girders behave essentially as simply-supported beams. With increasing live load and rotation at the diaphragm, the bottom crack closes and negative moment continuity becomes effective. The amount of rotation needed to close the crack is dependent on the creep and shrinkage properties of both the girder and deck concrete, the ages of the two concretes at the time of live load, the amount of restraint provided by the positive moment reinforcement in the diaphragm, and the girder type, span length and spacing. Therefore, the degree of negative moment continuity is dependent on all these parameters. With proper incremental application of live load, PBEAM correctly models the change in negative moment stiffness that accompanies closing of the diaphragm cracks, thereby providing an analytical tool to evaluate the effects of all these parameters.

A "tension stiffened" effective stress-strain relationship for top and bottom reinforcement at supports was also used in PBEAM in order to model behavior of reinforcing steel at cracked sections (7). These features allow PBEAM to accurately predict the changes in continuity at supports due to cracking of diaphragm and deck concrete as well as closing of cracks upon reversal of moments from application of live load.

Parametric Study

The program PBEAM was used to evaluate the effects of variations in several parameters on the behavior of bridges constructed of prestressed girders made continuous. The basic model used for the parametric study consisted of a bridge of four equal length spans, as shown in Fig. 2. Characteristics of the model which were varied included:

1. Girder type
2. Span length
3. Girder spacing
4. Positive moment continuity reinforcement
5. Time-dependent material properties
   - Concrete: Compressive strength
     - Creep coefficient
     - Shrinkage strain
   - Prestressing steel: Relaxation characteristics
6. Construction sequence
7. Girder age at application of live load

Values for these parameters were chosen to reflect the range of current practice as indicated by responses to the questionnaire. Following are brief discussions of the variables used in the parametric study.

Four girder types were used in the parametric study, AASHTO Types IV and VI, Modified Bulb Tee BT72/6 (8), and a box section. From questionnaire responses, AASHTO standard sections were most commonly used. The AASHTO-IV and AASHTO-VI sections were chosen to represent girders used for medium to long span bridges. The AASHTO-IV girder was used with span lengths of 70 and 100 ft (21 and 30 m) in the parametric study. The AASHTO-VI girder was used with span lengths of 100 and 130 ft (30 and 40 m) in the parametric study. These AASHTO standard girders have heavy cross-sections for their span length capabilities. Questionnaire responses indicated that a variety of lighter girder sections are in current use, particularly for longer spans. The BT72/6 section, from Ref. 8, was chosen to represent girders with a lighter, more efficient cross section. This girder was used with span lengths of 100 and 130 ft (30 and 40 m) in the parametric study. The girder spacing used for these three girders throughout the majority of parametric study analyses was 8 ft (2.4 m). Preliminary studies indicated that girder spacings varying between 4.5 and 8 ft (1.4 and 2.4 m) had a minor influence on bridge behavior. The box section which was used in the parametric study was a 48 in. (1200 mm) wide by 54 in. (1400 mm) deep section with side wall thicknesses of 5 in. (130 mm) and top and bottom wall thicknesses of 3.5 in. and 6.5 in. (90 and 170 mm), respectively. The box girder span length and girder spacing used in the parametric study were 100 and 11.7 ft (30 and 3.6 m), respectively. For AASHTO-IV, AASHTO-VI, and BT72/6 Sections, deck dimensions were 92 by 6.5 in (2340 by 170 mm). For the box girder section, deck dimensions were 140 by 9 in. (3560 by 230 mm).

One of the main priorities of the parametric study was to determine the effect on bridge behavior of varying amounts of positive moment reinforcement at supports. For each analyzed combination of parameters, positive moment reinforcement was set equal to one or more of 0.2, 3.6, or 7.2 sq in. (1.3, 23, or 46 sq cm). These values represent a range of reinforcement equivalent to one No. 4 bar up to twelve No. 7 bars. The 0.2 sq in. (1.3 sq cm) reinforcement is intended to represent an unreinforced section. A small amount of reinforcement had to be used in the PBEAM program to obtain a satisfactory numerical solution. Throughout the remainder of this paper, bridges analyzed with 3.6 or 7.2 sq in. (23 or 46 sq cm) of positive moment steel will be referred to as reinforced. Bridges analyzed with 0.2 sq in. (1.3 sq cm) will be referred to as unreinforced.

Preliminary PBEAM results indicated that variations in deck and girder concrete compressive strength had a minor effect on bridge behavior. Therefore, for the majority of parametric study analyses, girder and deck concrete compressive strengths were 6500 and 4000 psi (45 and 28 MPa), respectively. Two ultimate creep coefficients were used throughout the parametric study. A value of 3.25 represented the high end of the range and 1.625 was used for the low end value. Variation in ultimate shrinkage strain was found to be less influential than variation in ultimate creep coefficient. As a result, a value of 600 millionths was used for most PBEAM runs for the ultimate shrinkage strain for girder and deck concrete. For most parametric study runs, relaxation characteristics for stress-relieved strand were used. Preliminary PBEAM results indicated that the difference in bridge behavior between stress-relieved and low-relaxation strand was insignificant.

From responses to the questionnaire, it was learned that current bridge construction practices include a number of aspects. Girder age at the start of bridge construction varies from about 10 days up to about 300 days. The majority of respondents indicated that construction started at girder ages less than 90 days. The sequence of construction of deck and diaphragm fell into three general categories:

1. Casting the deck and diaphragms simultaneously at various girder ages.
2. Casting the deck approximately 7 to 10 days before the diaphragm at various girder ages.
3. Casting the diaphragm approximately 7 to 10 days before the deck at various girder ages.

For purposes of the parametric study, the majority of PBEAM runs were conducted assuming the deck and diaphragm were constructed simultaneously. The girder age at which continuity was established for these runs was either 17 or 67 days. Additional runs were conducted with girder age at continuity up to 320 days. Several runs were also done to investigate behavior when the deck was cast 7 days before the diaphragm and vice versa.

To evaluate the behavior of bridges under service conditions, bridge response was analyzed for the live load configuration shown in Fig. 2. The pattern of live load was chosen because it is symmetric about the central support and produces the maximum negative moment at the central support. The magnitude of the loads are based on the AASHTO HS20-44 lane load (9) with a girder spacing of 8 ft (2.4 m), two girders per lane of traffic, and an impact factor for span length equal to 130 ft (40 m). For purposes of comparison, the magnitude of load was not changed to account for different impact factors for 70 and 100 ft (21 and 30 m) spans. For bridges with girder age at continuity of 17 days, live load was applied at 650 days. For bridges with girder age at continuity of 67 days or greater, live loads were applied approximately 30 days after continuity was established. For several runs with continuity at 67 days, live loads were applied at 650 days.

CONTINUITY CONNECTION EFFECTS FOR POSITIVE MOMENT

The effects of providing varying amounts of positive moment connection steel on typical bridge behavior when positive support restraint moments develop are illustrated in Figs. 3 and 4. The results shown are from PBEAM analyses for AASHTO-VI girders with 130 ft (40 m) span lengths and ultimate creep coefficient of 3.25. The girder age at which continuity was established was 17 days and live load was applied at 650 days. The results shown in Figs. 3 and 4 and the discussion following are typical of all girder types, span lengths, and ultimate creep coefficient combinations analyzed in the parametric study for continuity age of 17 days and live load applied at 650 days. In these cases, a large proportion of girder prestress creep occurs after continuity is established, causing positive restraint moments to develop. Also, negative restraint moments caused by differential shrinkage between deck and girder concrete are lessened due to the fact that a relatively small amount of girder shrinkage has occurred before the deck is cast. The effects of varying amounts of positive moment reinforcement on support moments and midspan moments under these conditions are described below.

Moments at Supports

Time-dependent restraint moments at the central support of the four span bridge up to day 650 are shown in the left side of Fig. 3. The right side of Fig. 3 indicates moments upon incremental application of live load at day 650. The three curves on the graph are from PBEAM analyses with three quantities of positive moment reinforcement at supports, A_s. With positive moment reinforcement provided, A_s equal to 3.6 or 7.2 sq in. (23 or 46 sq cm), positive restraint moments accumulate over time. At an age of 650 days, restraint moment at the central support is equal to about 8000 kip-in (900 kN-m). With a small amount of positive reinforcement, A_s equal to 0.2 sq in. (1.3 sq cm), positive restraint moments are negligible. With a small amount of provided reinforcement, the girder end rotation causes cracks to develop in the bottom of the diaphragm. The rotational stiffness of the diaphragm is therefore reduced and restraints against girder end rotation decrease. When a substantial amount of reinforcement is provided, girder end rotation is more effectively restrained.

Figure 3 also shows response of central support moments to application of live load at 650 days. Live load was applied in three increments to reach 25, 50, and 100% of the load. It can be seen that the behavior of the reinforced and unreinforced sections differ. For analyses of bridges with reinforced diaphragms, application of live load causes immediate decrease of central support moment. At 100% live load, the moments are negative. In general, moments decrease linearly up to 100% of live load. For the analysis of the unreinforced diaphragm bridge, application of live load causes only a slight decrease in central support moment. This results from the fact that girder end rotation due to the applied live load is not restrained because of the low rotational stiffness of the cracked diaphragm. The positive moment crack at the bottom of the diaphragm must close prior to inducing negative moment at the continuity connection.

Mid-Span Moments

Time-dependent interior span midspan moments up to day 650 are shown in the left side of Fig. 4. The right side of Fig. 4 indicates midspan moments due to incremental application of live load at day 650. The three curves on the graph are from PBEAM analyses with three quantities of positive moment reinforcement at supports. With provided positive moment reinforcement at the diaphragms, midspan moments increase due to the positive restraint moments which develop at the supports. On the other hand, with unreinforced diaphragms, midspan moments remain essentially constant through day 650. Since the girder ends are virtually unrestrained, small restraint moments develop at supports, and midspan moments remain approximately equal to the composite section simple span dead load moment of 44500 kip-in. (5030 kN-m). The difference in midspan moments at 650 days between the unreinforced and reinforced diaphragm analyses is due to the difference in restraint moments at supports.

Upon application of live load at day 650, midspan moments increase in amounts dependent on the degree of continuity at supports. For analyses of bridges with reinforced diaphragms, support regions have relatively high rotational stiffnesses. As a result, continuous sections at supports resist some of the applied live load and the increase in midspan moment is reduced. For the analysis with negligible positive reinforcement, support regions have very small rotational stiffnesses because of diaphragm cracking. Therefore, essentially all of the applied live load is resisted by bending of the composite girder and midspan moments increase substantially until girder end rotation results in closing of diaphragm cracks.

An important feature of the data shown in Fig. 4 is that the resultant moment, defined as the final moment after application of live load, is the same regardless of the amount of positive moment reinforcement provided. The resultant moment is equal to 66000 kip-in (7460 kN-m) for the bridge analyzed in Fig. 4. The average change in midspan positive moment with application of live load for the reinforced diaphragm analyses is 11000 kip-in. (1240 kN-m). For the unreinforced diaphragm analysis, change in midspan moment with application of live