Evaluation and Verification of Time-Dependent Deformations in Posttensioned Box-Girder Bridges

HENRY G. RUSSELL, KWOK-NAM SHIU, WILLIAM L. GAMBLE, AND VERNON L. MARSHALL

A 76.2-m (250-ft) span, constant-depth precast concrete posttensioned box-girder bridge was instrumented to measure longitudinal strain caused by applied loads, posttensioning, creep, and shrinkage. Material properties of the concrete used in the instrumented span were obtained. Variation of compressive strength, elastic modulus, coefficient of thermal expansion, creep, and shrinkage with age of concrete were measured for specimens cured indoors and outdoors. Time-dependent deformations were calculated by using a step-by-step numerical procedure that used detailed construction records and material properties. Comparisons between measured and calculated deformations showed best agreement when concrete properties were based on specimens cured in an outdoor environment. However, good agreement was also obtained when concrete data from laboratory specimens were used. This indicates that time-dependent deformations of box girders can be predicted by using the step-by-step numerical procedure with the physical properties of laboratory specimens.

Three posttensioned box-girder bridges have been instrumented by the Construction Technology Laboratories of the Portland Cement Association to measure long-term deformations. The three bridges are Denny Creek Bridge in Washington, Kishwaukee River Bridge in Illinois, and Linn Cove Viaduct in North Carolina. Although the specific objectives of each project have been different, the overall objective has been to obtain information to verify design and analysis procedures for time-dependent effects in posttensioned box-girder bridges. This paper concentrates on the research program that was performed in connection with the Kishwaukee River Bridge. The specific objective of the project was to verify a step-by-step numerical procedure for calculating time-dependent deformations in segmental box-girder bridges constructed by the cantilever method.

HIGHLIGHTS

Kishwaukee River Bridge, which is located near Rockford, Illinois, consists of twin constant-depth precast concrete posttensioned box-girder bridges. Each bridge has five spans that measure 51.8 m (170 ft) for the end spans and 76.2 m (250 ft) for the three interior spans. The bridge was constructed by using the balanced-cantilever method with a launching girder. A cast-in-place section was used to complete each span.

Three cross sections of one bridge span were instrumented during construction. Instrumentation was installed to measure longitudinal strains caused by applied loads, posttensioning, creep, and shrinkage. Instrumentation was installed before the precast segments were cast.

In parallel with the field investigation, properties of the concrete used in the instrumented segments were measured from concrete cylinders obtained in the field. Variation of compressive strength, elastic modulus, coefficient of thermal expansion, creep, and shrinkage with time were measured. Specimens were cured under both constant temperature and humidity and under field conditions.

Deformations of the instrumented segments were calculated by using a step-by-step numerical procedure developed at the University of Illinois, Urbana. The procedure accounted for important parameters that influence the time-dependent behavior of segmental bridges.

Comparisons were made between calculated deformations and those measured on the structure. Good agreement was obtained for calculations that used properties of the concrete that was stored outside.

KISHWAUKEE RIVER BRIDGE

The Kishwaukee River Bridge, shown in Figure 1, is located in Winnebago County, about four miles south of Rockford, Illinois. When completed, it will carry four lanes of US-51, which serves as a major north-south highway. The Kishwaukee River Bridge consists of two identical parallel bridges spanning about 30 m (100 ft) above the heavily wooded Kishwaukee River Valley. Each bridge has three main spans of 76.2 m (250 ft) and two end spans of 51.8 m (170 ft) for a total length of 332 m (1090 ft). Arrangement and elevation of the spans are shown in Figure 2.

The Kishwaukee River Bridge is a continuous single-cell box girder made with precast concrete segments. Overall cross-sectional dimensions of a midspan segment are shown in Figure 3. Except for those over the piers, each segment has a length of 2.1 m (7 ft). Pier segments have a diaphragm, a thicker soffit, and are shorter in length. Each main 76.2-m span is made up of 34 precast segments with 1 cast-in-place closure segment at midspan.

Posttensioning ducts are located in the top and bottom slab of the box section. The top slab is prestressed both transversely and longitudinally. Matching alignment keys are located in the webs and the top slab to facilitate erection.

Threaded 32-mm (1.25-in) diameter posttensioning bars were used as longitudinal tendons. Mechanical couplers were used to connect bars. Each span was constructed by cantilevering out from the main piers. A launching girder was used to position the segments. Temporary longitudinal posttensioning was applied to hold segments together during construction. A cast-in-place segment closed each span so that all spans were continuous for live load.

FIELD MEASUREMENTS

Three segments in the southbound lane bridge of a

Figure 1. Kishwaukee River Bridge.

Figure 2. Elevation and plan of Kishwaukee River Bridge.

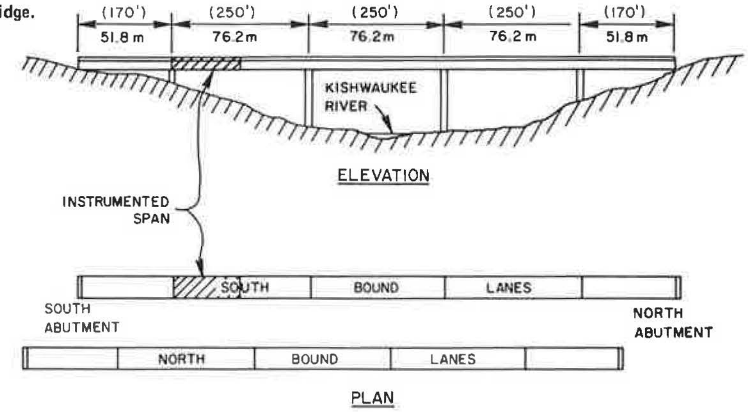


Figure 3. Segment dimensions and locations of strain measurements.

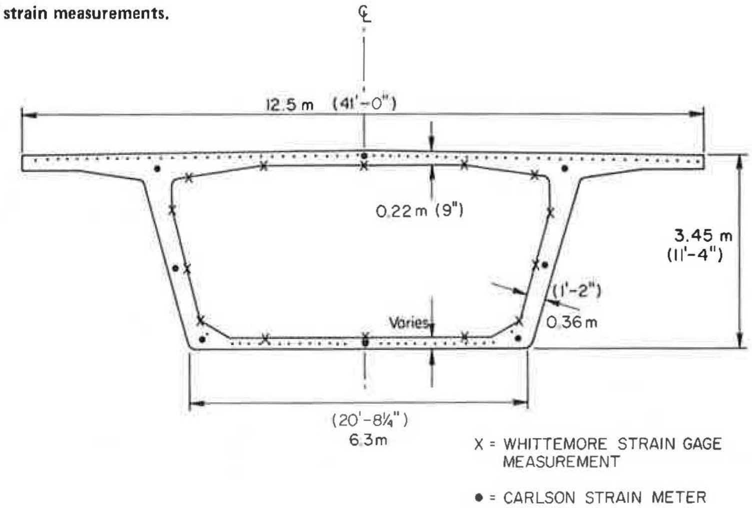
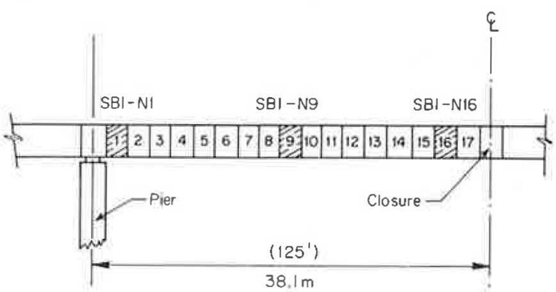


Figure 4. Location of instrumented segments.



76.2-m span (shown in Figure 2) were instrumented. Segments were designated as SB1-N1, SB1-N9, and SB1-N16. The segments were located next to the pier support, at quarter span, and near the center of the span, respectively. Locations of instrumented segments are shown in Figure 4.

Measurements of longitudinal strains, vertical deflections relative to the pier, and surface temperature of concrete were made in each instrumented segment. Installation of field instrumentation was completed in 1978.

Longitudinal strains were measured with a Whittemore mechanical strain gage (1) and 24 Carlson strain meters. Locations of Whittemore gage measurements and installed Carlson strain meters are indicated in Figure 3. The Whittemore strain gage measured the surface strain of concrete. The Carlson meter readings gave internal concrete strains and temperatures. Both Carlson strain meters and Whittemore strain gage readings yielded consistent and similar results. In this paper, only data from the Carlson strain meters are discussed.

MATERIAL PROPERTIES

Properties of the concrete used in the instrumented segments were determined from tests of concrete cylinders made at the precasting plant. Thirty-five 152x305-mm (6x12-in) concrete cylinders were used for each bridge segment. Cylinders were steam-cured alongside the segments before shipping to Construction Technology Laboratories for tests. After arrival, the cylinders were cured either under a constant temperature of 23°C (73°F) and 50 percent relative humidity or under outdoor conditions.

Physical property tests were conducted to determine the variation of compressive strength (ASTM C39-72), modulus of elasticity, Poisson's ratio, and coefficient of thermal expansion with time. Physical properties of the concrete at different ages are summarized in Table 1.

Creep tests of cylinders cured under a constant temperature of 23°C and 50 percent relative humidity were initiated at three different concrete ages. All creep cylinders were subjected to a constant stress of 13.8 MPa (2000 psi). Tests were conducted according to ASTM C512-74. Similar creep tests for cylinders cured outside were started at a concrete age of 28 days. Whenever creep readings were made,

Table 1. Concrete properties.

Segment	Curing Environment	Age (days)	Compressive Strength, f'_c (MPa)	Modulus of Elasticity, E (MPa)	Poisson's Ratio	Coefficient of Thermal Expansion (millionths/°C)
SB1-N1	Controlled	28	39.2	29 800	0.15	9.63
		180	43.0	31 000	0.15	9.86
		360	40.9	30 400	0.13	-
	Outdoor	28	39.6	29 000	0.15	-
		180	44.5	31 200	0.15	-
SB1-N9	Controlled	28	42.7	30 800	0.16	9.86
		90	41.9	31 900	0.15	10.26
		180	44.5	32 800	0.15	10.13
		360	41.1	31 200	0.14	-
	Outdoor	28	41.9	31 800	0.14	-
		90	44.9	32 500	0.15	-
		180	44.5	31 600	0.14	-
SB1-N16	Controlled	28	39.6	30 500	0.15	10.17
		90	42.3	32 800	0.15	10.53
		180	43.4	30 400	0.15	10.49
		360	41.9	31 200	0.14	-
	Outdoor	28	32.2	28 900	0.14	-
		180	42.8	29 200	0.15	-

Note: 1 MPa = 145 psi, t°C = (t°F − 32)/1.8.

Figure 5. Specific creep versus time.

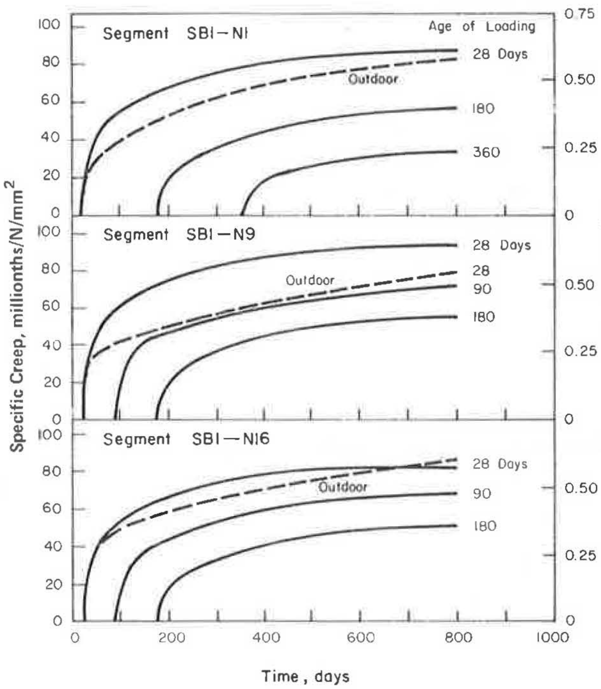
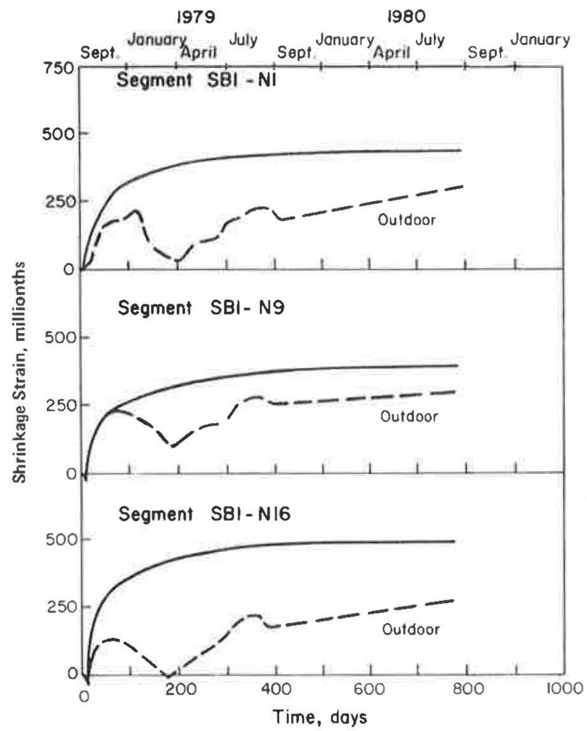


Figure 6. Shrinkage versus time.



companion shrinkage readings of concrete were taken.

Variation of specific creep of concrete with time for each segment is shown in Figure 5. Specific creep is defined as the amount of creep strain under unit stress in millionths per newton per square millimeter. Concrete shrinkage was excluded from the creep strain readings. Different curves in the figure represent specific creep of concrete loaded at different ages. For outdoor specimens, creep readings were adjusted back to 23°C for comparison. Specific creep of outdoor specimens was found to be lower than for those cured under the controlled environment. This relation is shown in Figure 5.

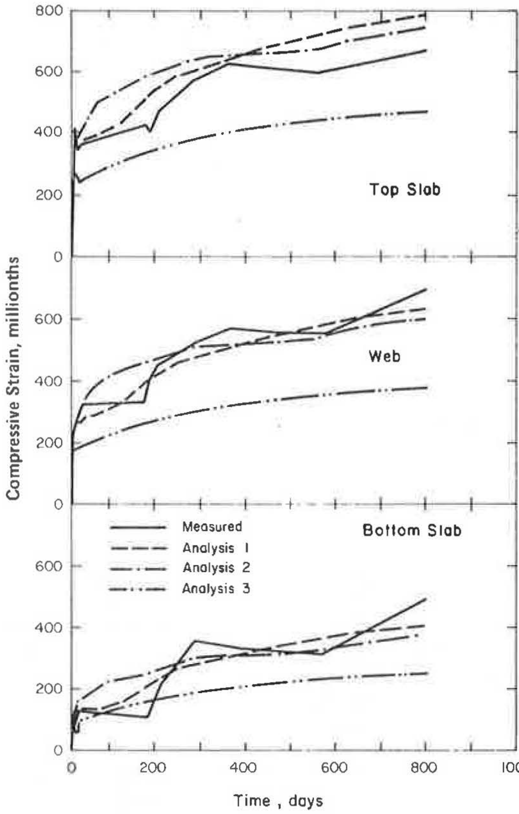
Shrinkage measurements of concrete cylinders began seven days after casting. A comparison of the data for specimens cured at constant temperature and under the outdoor environment is shown in Figure 6.

CALCULATED DEFORMATIONS

The time-dependent deformations of the bridge were calculated by using a step-by-step numerical procedure (2). Total shortening was considered to include instantaneous deformation, shrinkage deformation, and creep deformation. The analysis accounted for effects of concrete material properties, relaxation of prestressing steel, member thickness, elastic recovery, age of loading, and creep of concrete under a variable stress history. Size correction factors were based on Comite Europeen du Benton (CEB) recommendations (3). The actual casting and erection schedule of the segments was followed. Further details are given elsewhere (2).

Analyses were performed for three different sets of material properties. In analysis 1, experimen-

Figure 7. Comparison of measured and calculated strains for segment SB1-N1.



tally determined properties of the concrete specimens stored outdoors were used. Analysis 2 was performed by using material properties determined from the laboratory-cured specimens. For analysis 3, the recommendations of CEB (3) were used to generate material properties. Relative humidity was assumed to be 50 and 80 percent for creep and shrinkage, respectively (4).

COMPARISON OF RESULTS

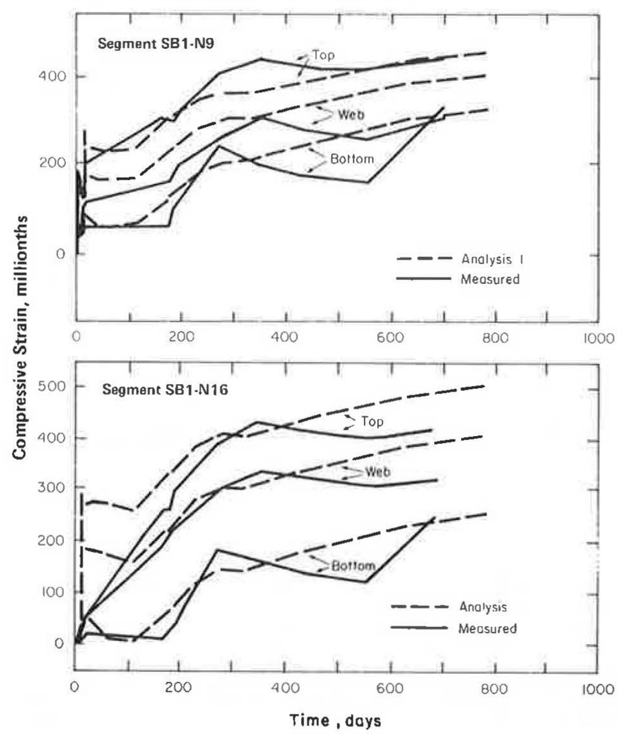
Calculated strains were compared with measured strains to determine the effectiveness of the analytical procedure. Detailed comparisons are presented elsewhere (2). Comparisons of measured strains with strains calculated from the three analyses for segment SB1-N1 are shown in Figure 7. The best agreement was obtained with analysis 1, which used the concrete material properties of specimens stored outdoors. Good comparison was obtained with analysis 2, which used material properties of laboratory-cured concrete cylinders. In analysis 3, calculated strains were consistently smaller than the measured values.

The comparison of measured and calculated strains by using analysis 1 for segments SB1-N9 and SB1-N16 is shown in Figure 8. Good agreement was obtained. Similarly, good agreement between measured and calculated strains was obtained for analysis 2. This shows that time-dependent deformation of the box girder can be adequately estimated by using material properties obtained from laboratory-cured concrete cylinders.

CONCLUSIONS

Comparisons of calculated and measured deformations on the Kishwaukee River Bridge were made. Excellent

Figure 8. Comparison of measured and calculated strains.



agreement was obtained when concrete material properties were based on specimens stored outdoors. Reasonable comparison was also obtained when material properties based on laboratory-cured specimens were used. The analytical procedure therefore provides a suitable method for predicting time-dependent deformations in posttensioned box-girder bridges. Time-dependent behavior of box girders can be estimated by using standard laboratory data.

ACKNOWLEDGMENT

Work described in this paper was performed as part of an agreement between the Portland Cement Association and the Illinois Department of Transportation (DOT) in cooperation with the Federal Highway Administration (FHWA). The analytical work was performed at the University of Illinois, Urbana, under an agreement with the Portland Cement Association. Richard Taylor of the Illinois DOT coordinated the project for the State of Illinois. Members of the advisory committee who reviewed the progress of the project were Robert Appleman, Gayle Lane, Richard Scarr, Donald R. Schwartz, Colin Strang, Richard Taylor, and John Ross. Their contributions to the project are appreciated.

The contents of this report reflect our views, and we are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Illinois DOT or FHWA. This report does not constitute a standard, specification, or regulation.

REFERENCES

1. N.W. Hanson, T.T.C. Hsu, O.A. Kurvits, and A.H. Mattock. Facilities and Test Methods of PCA Structural Laboratory--Improvements 1960-65. Portland Cement Association, Skokie, IL, PCA Development Bull. D91, July 1965, p. 38.
2. V. Marshall and W.L. Gamble. Time-Dependent De-

formations in Segmental Prestressed Concrete Bridges. Civil Engineering Studies, Univ. of Illinois, Urbana, Structural Research Series 495, Oct. 1981, 242 pp.
3. International Recommendations for the Design and Construction of Concrete Structures: Principles and Recommendations. Comite Europeen du Benton (European Committee on Concrete), Paris, June 1970.
4. H.D. Hernandez and W.L. Gamble. Time-Dependent Prestress Losses in Pretensioned Concrete Construction. Civil Engineering Studies, Univ. of Illinois, Urbana, Structural Research Series 417, May 1975.

Publication of this paper sponsored by Committee on Concrete Bridges.

Abridgment

Load Capacity of Concrete Bridge Decks

DAVID B. BEAL

The behavior of two reduced-scale concrete bridge decks subjected to simulated wheel loads was evaluated in a series of tests. One slab was reinforced in accordance with American Association of State Highway and Transportation Officials requirements and the other had three areas with varying amounts of isotropic reinforcement. Results show that with either reinforcement pattern, service load bending moments are from 40 to 65 percent of those predicted by flexural theory. Failures were by punching shear rather than flexure and occurred at loads at least six times larger than design.

The need to determine the influence of heavily loaded, closely spaced wheels and axles on reinforced concrete bridge decks prompted the New York State Department of Transportation (NYSDOT) to initiate an analytical study of bridge deck behavior in 1977. The products of this study were charts that permitted the determination of the induced bending moment in decks due to any pattern of wheel loads (1). During the course of the research, more evidence became available [which culminated in the publication of the Ontario Highway Bridge Design Code (2)] that the failure mode of reinforced concrete bridge decks was punching shear and not flexure as assumed in design. Because of this evidence, the study reported here was started to investigate the ultimate capacity of bridge decks.

The Ontario bridge deck design resulted from extensive physical and analytical research (3). This work demonstrated that not only is the failure mode of reinforced bridge decks different from that historically assumed, but that the load capacity is substantially greater than necessary for safety. The enhanced behavior of bridge decks is explained by hypothesizing large in-plant compressive forces that result from the restraint of deck expansion under load. These compressive forces form an internal couple that enhances the flexural capacity of the deck to a level such that punching shear failure controls. Subject to certain restrictions on span length, slab thickness, and detailing of diaphragms and shear connectors, Ontario permits an empirical slab design that has a minimum of 0.3 percent isotropic reinforcement in each face. For a 9-ft slab span, this represents a reduction in reinforcement of 43 percent from that now required by New York State standards (4).

In addition to the savings that result from the reduction of steel, benefits may accrue from this empirical design by reducing fabrication costs and deck deterioration due to reinforcement corrosion. The reinforcement can be standardized over the normal range of girder spacings and has the potential benefit of modular prefabrication. Reinforcement corrosion is alleviated because the cover on the top steel can be increased without an increase in slab thickness. In addition, the reduction in bar size increases the important cover-diameter ratio (5).

The objective of the work described in this paper was to collect data on bridge slab capacity with different reinforcement schemes. This work was accomplished with several reduced-scale models that are described in detail in the complete report (6).

EXPERIMENTAL WORK

Two reinforced concrete bridge decks were constructed to study behavior under working loads and at failure. Model 1, which represents the current standard bridge deck design (4), was included to demonstrate the great reserve capacity of that design and to provide a standard of comparison with alternative designs. The 8.5-in-thick slab was reinforced with No. 5 bars in the longitudinal and transverse directions. Longitudinally, the top layer of bars was spaced at 18 in and had the bottom layer spaced at 7.75 in in the middle half of the slab span and at 18 in elsewhere. Both layers of transverse steel were spaced at 5.25 in.

Model 2 represented an 8-in-thick deck that has three different isotropic reinforcement patterns comprised of No. 4 bars. Two layers at 8-in spacing represented current Ontario practice (3). A single layer at 8 in was used because of the construction benefits to be gained if this pattern could be adopted. Two layers at 12-in spacing were used to represent the minimum reinforcement now permitted (7). In addition, an unreinforced section was included to demonstrate the inherent strength of confined concrete slabs. Both models were constructed to a linear scale factor of 5.9 and were based on a five-girder, 72-ft simple-span bridge, which is representative of composite highway structures now being built. Details of model materials and construction details are given in the full report (6). Electrical-resistance strain gages were mounted on the rebars and deflection at the center of the slab was measured.

TEST RESULTS

Each instrumented section of the model slabs was subjected to test loads for two distinct purposes: (a) determination of the distribution of bending