Field Testing of Two Prestressed Concrete Girders

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● THIS PAPER presents the procedures used and the results obtained in a cooperative research project designed to obtain field data to determine the actual behavior during construction and in service of one of the first prestressed concrete structures designed by the Missouri State Highway Department. During the initial planning stage of a pedestrian bridge across the Mark Twain Expressway in metropolitan St. Louis, it was decided to use prestressed, post-tensioned concrete girders. In discussing the design and specifications for the girders, it was suggested that worthwhile technical information could be obtained with a relatively low-cost field observation program.

The program was initiated with the design and manufacture of an instrumentation system which would meet the rugged demands of field service condition and yet provide reasonable accuracy. University of Missouri research personnel supervised the installation of all instrumentation appurtenances in the various elements of the structure, and collected all field data. Measurements were made before, during, and after the post-tensioning operations, before and after all major construction operations, and at appropriate time intervals following completion of the structure. To provide the additional information needed for interpretation of the field measurements, a companion test program was undertaken to measure the actual physical properties of representative samples of the materials used in the construction of this structure.

OBJECTIVES

The objectives of this investigation were to: (a) compare the actual prestress camber of the girder with the camber predicted by the design computations; (b) investigate the distribution of prestressing strains at several sections in the girder; (c) observe the changes in strains and deflections due to creep and shrinkage; (d) evaluate the composite behavior by comparing the observed dead load deflections due to the weight of the slab with the computed deflections; and (e) investigate the effect of differential shrinkage between the girders and the cast-in-situ slab.

INSTRUMENTATION REQUIREMENTS

This research project was primarily conceived as a field study wherein measurements were to be made at regular intervals over a long period of time. To obtain consistent data under these conditions it was considered desirable that the instrumentation used be relatively simple as well as flexible. After a preliminary study it was decided that the following measurements would best fulfill the objectives of the project: (a) concrete strain measurements at several different locations on the girders and in the cast-in-situ walkway slab to provide strain profiles at critical sections, and (b) girder and bridge deflection measurements to provide a record of the actual deflection or camber for correlation with theoretical computed values.

Consideration was given to the use of dynamometers or other instrumentation for determining the loss of tension in the prestressing tendons due to relaxation of the steel and to inelastic strains in the concrete. Although this latter information was felt to be most desirable, the ensuing complexity of the required instrumentation made measurements of this type infeasible. Furthermore, because the tendons were to be grouted soon after post-tensioning, such instrumentation would not have given accurate information on time-dependent losses but would only have reflected the dimensional changes in the concrete at a particular section. It was therefore decided to estimate the relaxation losses in the tendons by means of a series of companion tests on representative samples under laboratory conditions.

INSTRUMENTATION SELECTION AND LOCATION

Figure 1 shows the completed bridge and its substructure. The bridge is oriented on a north-south axis above the expressway which runs east-west. In all references to the girders, the slab, or the completed bridge, directions will be designated in accordance with this orientation; that is, the east girder, the south end, etc.

On the basis of a preliminary investigation of available strain measuring devices, the basic instrument selected for the measurement of concrete strains was a Whittemore strain meter with a 10-in. gage length reading directly to unit strains of 10 microinches per in. A series of brass inserts were cast into the girders to furnish suitable gage points. The strain-gage points were selected to provide typical strain profiles at the critical sections shown in Figure 2. In general, the stations were placed as close to the end points, the quarter points, and the centerline as was feasible. The occurrence of transverse diaphragms at the quarter points necessitated a slight displacement in the gage point locations. Inserts were installed at each station on the inside faces and on the bottom of each girder at the locations shown in Figure 2. In addition to these inserts, during the concreting of the cast-in-situ walkway slab, strain-gage inserts were installed along the centerline of the walkway in the bottom and in the top surfaces of the slab at each station.

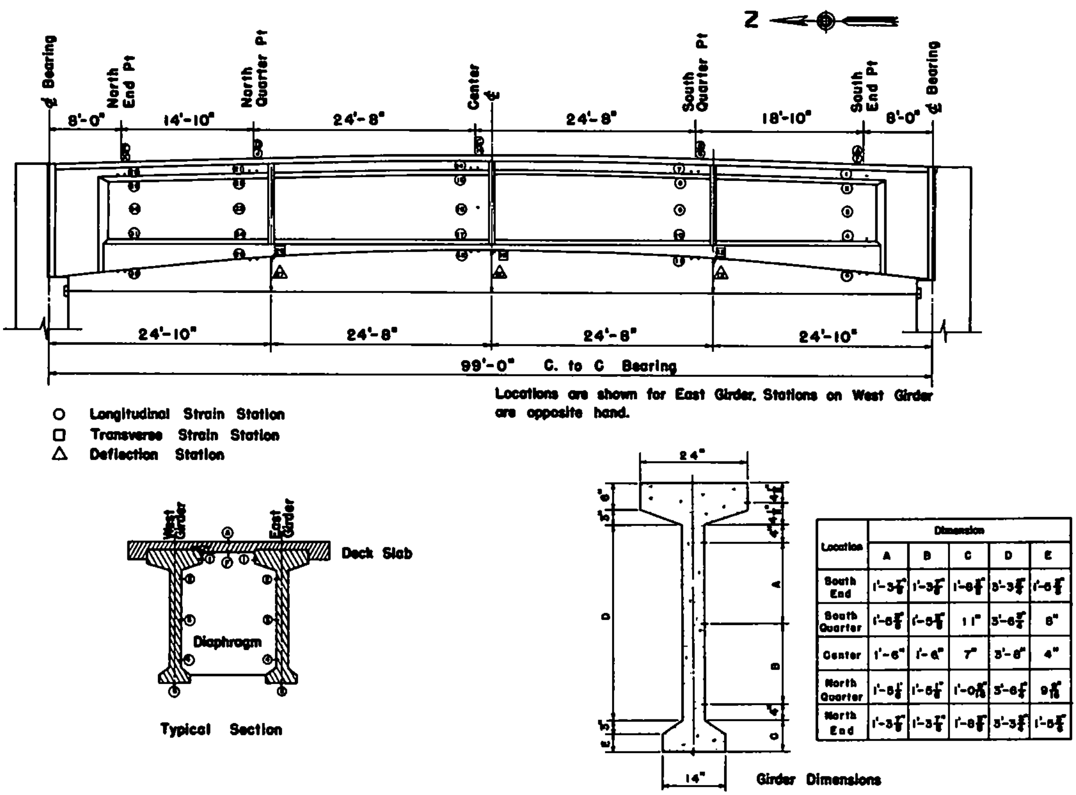
Figure 1. Completed structure.

In selecting a method for measuring the actual deflection or camber at various points along the girders, several schemes were investigated. Because of the small range of deflections anticipated, a system which would give an accuracy of a few thousandths of an inch was desired. Optical instruments available were not adopted because of their lack of sensitivity. After further preliminary study, it was decided that the most feasible method would be to install permanent reference points in the abutments to which a tensioned wire could be attached to serve as a baseline for these measurements. Then brass inserts, similar to those used in the strain measuring program, could be cast into the girder to serve as reference points. The actual deflection measurements would be made by using some mechanical means of determining the change in the distance between the baseline and the reference points on the girders. The system had to be further modified so that it could be used while the girders were still on the ground, before being placed on the abutments.

A baseline system was developed which consisted of a tensioned wire stretched between two fixed reference points. To provide reference points for the readings after the girders were in place on the abutments, special anchorage boxes were cast into the faces of the end piers about 10 in. directly below the center of each girder bearing plate. Four boxes were provided and installed, two in each pier. The position of the reference wire in each anchorage was fixed by passing the wire over a grooved bearing. The two boxes on the south pier were provided with attachments to anchor the wire, and the boxes on the north pier were provided with removable brackets. In setting up the baselines, wires were passed over an additional pulley on the removable brackets and calibrated 50-lb weights were attached to provide constant tension. The No. 23 gage steel music wire used was stressed to about 100,000 psi by the 50-lb weight. The wires were removed between tests and the boxes closed with cover plates.

To provide a reference line for deflection measurements before the girders were placed on the abutments, a temporary baseline system was developed. Two portable, demountable anchorage brackets were designed and manufactured so that they could be mounted on special plug inserts cast into the top surfaces of the girders above the bearing plates. At one end the bracket was provided with a fixed anchorage for the wire, whereas at the other end the bracket was provided with an arm and two additional grooved pulleys from which a 50-lb weight could be suspended. The initial deflection measurements were made using the temporary baseline and inserts spaced along the top surface of the girders. After the girders were installed on the piers, the deflection measurements were transferred to equivalent measurements by using the insert points on the bottom surface of the girder with the permanent baseline system.

Brass inserts were cast into the girders on both the top and bottom surface at the quarter- and center-points as indicated in Figure 2. All deflection measurements were made with the deflection meter shown in Figure 3. The meter incorporated a standard 12-in. Starrett height gage provided with a micrometer adjustment and with



Location	Dimension				
	A	B	C	D	E
South End	1'-3[illegible]"	1'-3[illegible]"	1'-8[illegible]"	3'-3[illegible]"	1'-5[illegible]"
South Quarter	1'-5[illegible]"	1'-5[illegible]"	11"	3'-6[illegible]"	8"
Center	1'-6"	1'-6"	7"	3'-8"	4"
North Quarter	1'-5[illegible]"	1'-5[illegible]"	1'-0[illegible]"	3'-6[illegible]"	9[illegible]"
North End	1'-3[illegible]"	1'-3[illegible]"	1'-8[illegible]"	3'-3[illegible]"	1'-5[illegible]"

Figure 2. Location of gaging stations.

a vernier which permits measurements to be made to an accuracy of 0.001 in. Two modifications were made to make field measurements feasible. A leveling base plate was provided to which the height gage was firmly attached. The base plate had two fixed support points which rested directly in drilled holes in the reference strip, and an adjustable support which permitted leveling of the meter. A level bubble on the base plate indicated when the instrument had been correctly leveled. The second modification was the addition of an extension arm with a vibrating reed. When the movable head of the reed was brought into contact with the baseline wire, the vibration of the reed was damped, giving a sensitive indication of position.

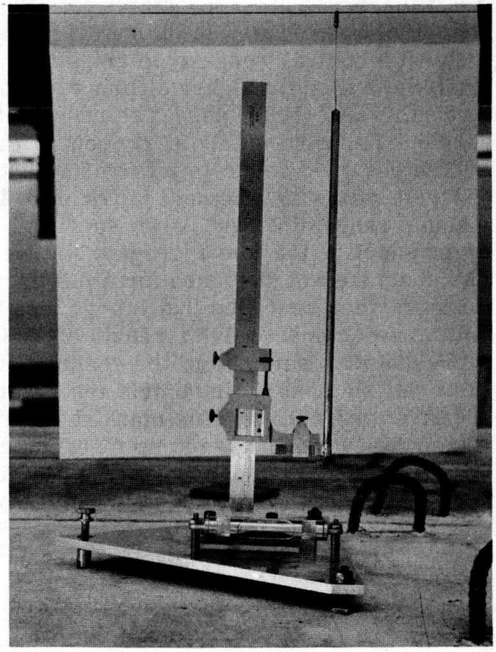


Figure 3. Deflection meter.

Preliminary experimentation and field experience showed that with this measuring system, deflection readings could be reproduced to within 0.002 in., except on unusually windy days when the wire would tend to oscillate. On such days extra care and patience were required, but on only one such day was the wind so severe as to actually interfere with the collection of deflection information. It was necessary to repeat the deflection readings for that date.

A complete description of the instrumentation system and drawings of all the special appurtenances have been given by Pauw and Breen (1).

COMPANION TEST PROGRAMS

A series of companion tests was carried out on representative samples of the materials used in the construction of the actual structure. The purpose of these companion tests was to provide additional information regarding the physical properties of the various materials used.

The post-tensioning system elected by the contractor was the Prescon system. Prescon tendons are manufactured units composed of a number of ¼-in. diameter wires laid in parellel. Positive end anchorage is insured by cold-formed ⅜-in. diameter button or rivet heads on both ends of each wire. All the specimens of the tendon steel furnished by the contractor had button heads formed on the ends, and they were equipped with stressing washers and spreader plates. The specimens were representative samples of the steel used in the main tendons. Information furnished by the contractor indicated that the tendons were manufactured from ¼-in. diameter cold-drawn, high-tensile wire, stress relieved in a lead bath at 600-800 F., and having a guaranteed minimum ultimate tensile strength of 240,000 psi.

The tension test specimens furnished consisted of three single-wire tendons, each of which was 36 in. long when measured between the rivet ends. In the initial tension tests, the specimens were gripped in a 60,000-lb universal testing machine in a manner simulating the way the wires are stressed in the girders; that is, the entire load was transferred to the tendon through bearing on the button heads. All three specimens displayed identical modes of failure, the failure occurring at the throat of the button head formed on the lower end of the wire as placed in the testing machine. The average ultimate strength was 238,300 psi.

Because all the specimens failed by separation of the cold-formed button head, it was decided to retest the specimens using conventional mechanical gripping devices to see if there was any appreciable loss in ultimate tensile strength associated with this method of anchorage. In the retest procedure, the specimens were gripped mechanically with conical wedge-type grips near the center of the strands. All specimens exhibited a slightly higher ultimate strength in the retest and the average ultimate strength was 247,800 psi. The results indicate that the button-head anchorage device developed approximately 96 percent of the tendon ultimate strength.

The designed concrete mix was selected to meet the specifications for a class B-1 concrete mix with a cement factor of 6.4 sack/cu yd. An air content of 4 percent and a slump range of 2-3 in. were specified. The aggregates selected were Missouri River sand for the fine aggregate and Meramec River gravel for the coarse aggregate. The concrete was designed for a minimum 28-day compressive strength of 5,000 psi. It was further specified that the girders be post-tensioned after the concrete attained a minimum compressive strength of 4,000 psi. During the concreting operations, representative samples of the concrete were obtained for each truck load, and conventional slump and air content tests were made. The results obtained in these tests are given in Table 1. The marked variation in the concrete properties should be noted. Although several of the slump tests indicated greater slumps than permitted by the specifications, the contractor was permitted to complete the casting of this girder at his own risk with acceptance depending on the results of the compressive strength tests.

TABLE 1

CONCRETE PROPERTIES

East Girder Cast October 4, 1957					West Girder Cast October 28, 1957				
Truck	Slump (in.)	Air Content (%)	f'_c (psi)	E_c (psi)	Truck	Slump (in.)	Air Content (%)	f'_c (psi)	E_c (psi)
1	2.0	2.8	4,750	5.33	1	3.5	5.0	5,620	5.23
2	Rejected				2	1.0	3.8	5,690	5.72
3	2.5	5.0	5,580	5.14	3	6.0	7.5	4,950	5.04
4	7.0	7.5	4,600	5.42	4	4.0	7.5	4,630	4.69
5	2.25	4.1	5,720	5.59					
Average			5,160	5.37	Average			5,190	5.27

Compressive strength tests were performed on standard 6- x 12-in. concrete cylinders as well as on special 3- x 6-in. cylinders. All cylinders were cast on the job site from representative samples of each truck load of ready-mix concrete. Cylinders from each batch were tested at various ages to provide information on the development of compressive strength. These tests indicated that although many of the batches did not reach the specified 5,000-psi compressive strength at 28 days, all attained the 4,000-psi strength specified for post-tensioning. The contractor was permitted to prestress the girders on December 5, 1957. As of that date the east girder was 61 days old and the west girder was 37 days old. The cylinder strength, f'_c, and the secant modulus of elasticity, E_c, at $f'_c/3$ were determined from tests of companion cylinders. The average values of these properties at the time of prestressing are given in Table 1. Because of the wide scatter and of the overlap in the values from the samples for both girders, it was felt that a single average value of E_c of 5.3×10^6 psi for the modulus was reasonable and this value was used to analyze the behavior of the structure.

DESIGN FEATURES

The design of the pedestrian overpass was governed by applicable requirements of the Missouri State Highway Commission "Standard Specifications," the "Bridge Specifications" of the American Association of State Highway Officials, and the U.S. Bureau of Public Roads "Criteria for Prestressed-Concrete Bridges." The final design dimensions of the girders and of the composite bridge section are shown in Figure 4.

The design specified an initial prestress force for each girder of 424 kips based on a required effective post-tensioning force of 360 kips with the losses assumed equal to 15 percent of the initial force. A temporary increase of the allowable initial unit stress from 160,000 psi to 192,000 psi was permitted to allow temporary overstressing to reduce losses owing to friction and tendon relaxation.

Following award of the contract, the contractor submitted the tendon layout shown in Figure 4, consisting of five cables so arranged as to meet the specifications as to tendon area and the required location of the center of gravity. The upper and lower tendons (Mark A and E) each consisted of twelve ¼-in. diameter wires, each wire having a minimum specified area of 0.0491 sq in. The three interior tendons (Marks B, C, and D) were each made up of ten wires of the same diameter and area. The contractor proposed to use an initial transfer stress of 160,000 psi, corresponding to an initial prestress force of 424 kips; and a working stress after losses of 136,000 psi, corresponding to the specified 360 kip effective post-tensioning force. The order of post-tensioning of the tendons was specified as Mark C, D, B, E, and A.

CONSTRUCTION OPERATIONS

The girders were cast and stressed at a location adjacent to the bridge site. The soffit forms were supported by timber sleepers set in the ground and were adjusted to grade by the use of blocks and wedges. The girders were stressed on December 5, 1957, after the cylinder tests indicated that the girders had reached the necessary

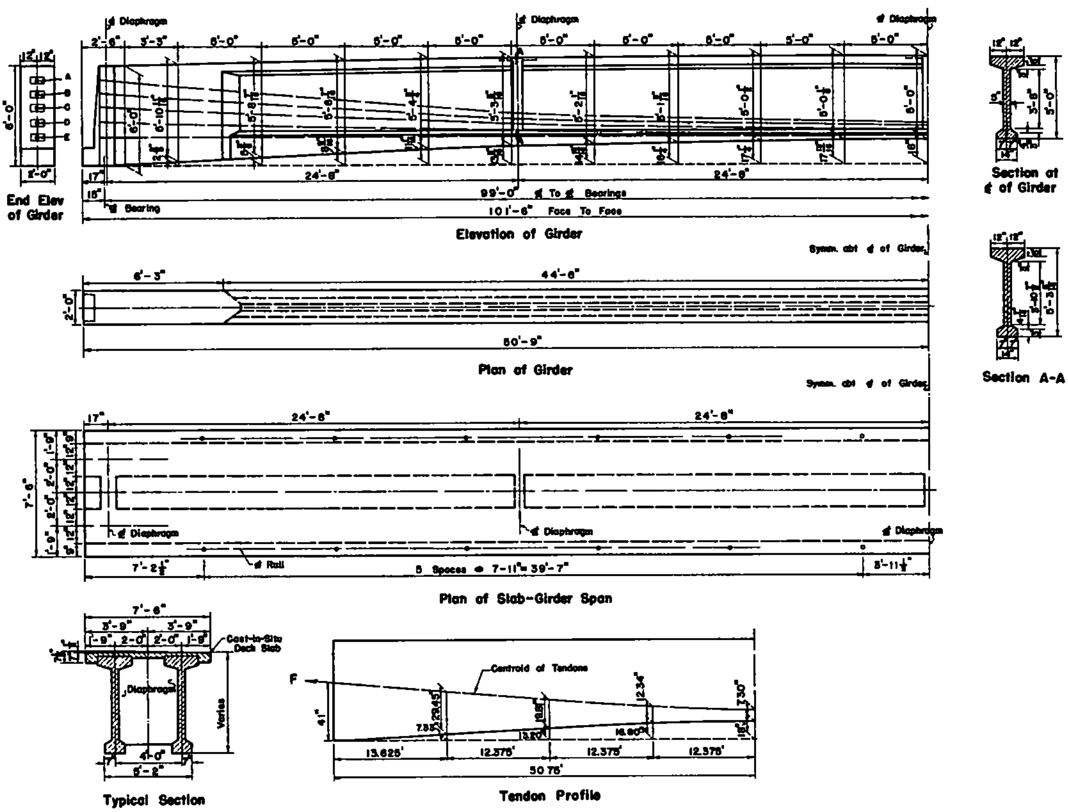


Figure 4. Design dimensions.

4,000-psi minimum compressive strength specified for post-tensioning. Strain and deflection measurements were made after tensioning each cable. Approximately two weeks after tensioning, the tendons were grouted. The girders were raised into place on the piers on January 27, 1958. An auxiliary bracing system was used to increase the lateral stiffness of the girders to permit handling of the long, slender sections with a minimum danger of failure. This bracing system consisted of a set of auxiliary posts and a pair of tensioned cables acting as a double king post truss. It proved quite effective in increasing the lateral resistance of the girders. However it introduced unbalanced strains due to lateral bending. Unfortunately the strain instrumentation was provided on only one side of the girders, and it was therefore not possible to measure the transverse strains which this system might have induced in the girders, thus adding an additional unknown factor.

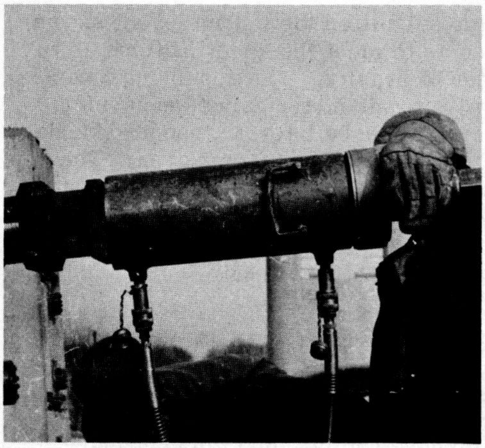


Figure 5. Stressing operation.

After the girders had been set in place on their bearing surfaces, deflection measurements were made to transfer the deflection baseline from the temporary reference line on top of the girders to the permanent baseline installed in the piers. Deflection and strain measurements were continued at regular intervals following erection of the girders.

On April 30, 1958, the deck slab and connecting diaphragms were placed. Prior to the concrete placement temporary shores were installed under each girder at the quarter-points and at the center of the span using sections of tubular scaffolding. The purpose of these shores was to minimize the deflection due to the dead load of the deck slab by developing composite action of the slab with the girders before the total weight of the slab was transferred to the structure. The results of this procedure are discussed in a subsequent section. After completion of the top slab and of the diaphragms which joined the girders together, a hand railing was added and by June 17, 1958, the bridge was completed.

FIELD MEASUREMENTS

At the various stages of construction, complete sets of field measurements were made except when measurements were rendered impossible by the construction procedures. The original readings of the strain and deflection instruments at the respective stations immediately prior to the commencement of the post-tensioning operation has been selected as the arbitrary reference base. Camber is designated by a positive change in elevation and deflection by a negative change. Tensile strain is designated by a positive change and compressive strain by a negative change.

At the time of stressing, the girders rested on the soffit forms and it was not possible to make measurements on the bottom strain gaging stations; hence, strains in the lower fibers could not be referenced to initial measurements before stressing. These measurements were therefore interpreted on the basis of an assumed linear strain distribution in the girder section 53 days after stressing, at which time the bottom strain gaging stations were first accessible. The linear strain distribution was determined from the measured strains at the other longitudinal strain gaging stations at that section on that date. A correction factor was then computed to make the bottom strain reading correspond with this assumed strain distribution. All subsequent strain measurements for the bottom stations were corrected by this same factor. Discussion of the significance of the data obtained is presented in subsequent sections as they pertain to the various subjects under study.

POST-TENSIONING PROCEDURE

The basic objective of post-tensioning is to secure the specified level of prestress in the girder. To accomplish this objective, a means of measuring the actual prestressing force applied as well as a reasonable method of predicting changes in this force and of estimating the prestress level at a given section and at a given time are required.

In the Prescon system, tension is applied to the tendon with a jack which bears on a stressing cage placed between the jack and a special end plate cast into the end of the girder. Then tendon is gripped by an adaptor rod connected to a special stressing washer which bears against the rivet heads formed on the individual wires. When the jack is activated, the stressing washer is forced away from the end bearing block until a desired elongation in the tendon is obtained. Shims are then inserted between the bearing block and the stressing washer. When the jack is released, the force is transmitted to the shims. Figure 5 shows the ram attached to tendon B andthe shim plates inserted between the bearing plate and the stressing washer. In this figure the jack has not yet been released to transfer the force to the shim plates. Tendon A, located directly above the ram, is still in its original position flush with the end bearing plate, because it has not yet been stressed. Tendon C, located below the ram, is in its final stressed position, and the shims, which prevent the stressing washer from returning to its original position, can be seen plainly.

In the post-tensioning procedure used for these girders, the tendons were jacked simultaneously at both ends to reduce frictional losses, and shims of proper length were installed prior to release. Ample clearance was available for inserting the shims because the tendons were temporarily overstressed to reduce friction and creep losses. Because only one of the two jacks had been adequately calibrated in the equipment provided for the post-tensioning operation, the tendon stress was controlled by measuring the tendon elongation, and the jack pressure gage readings were observed only as a check for correlation purposes.

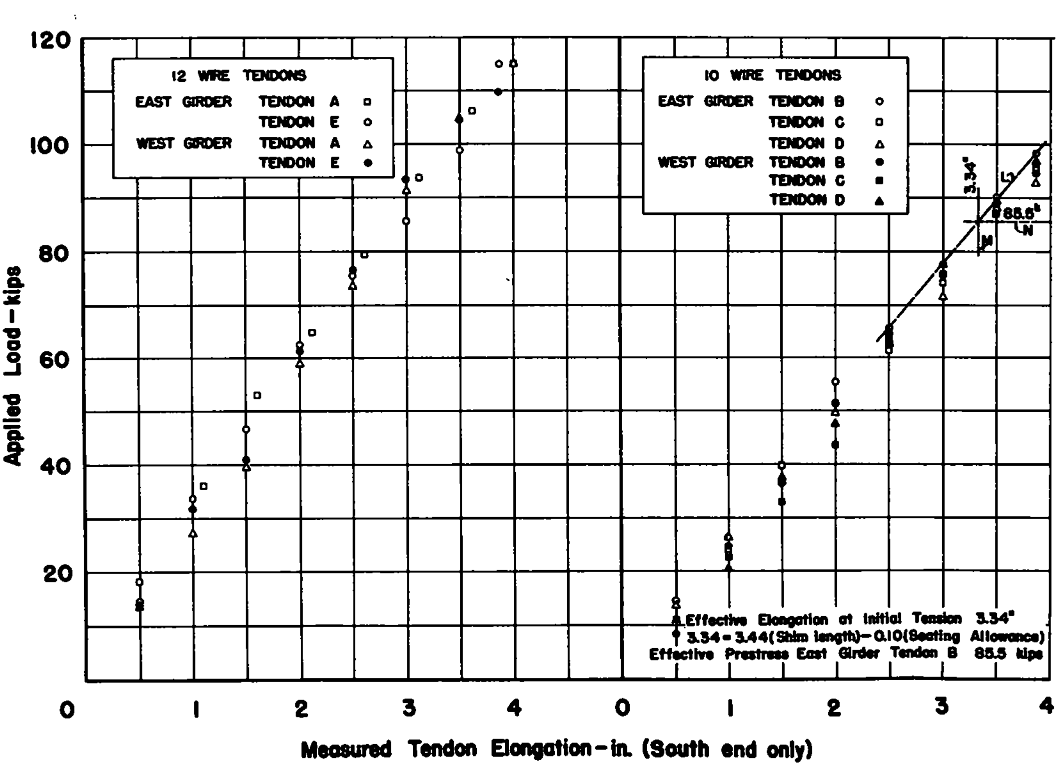


Figure 6. Tendon elongation.

The required length of the shims was calculated on the basis of elastic elongation of the tendons at the desired transfer stress of 160,000 psi. In addition to this the manufacturers recommended that the computed elongation be increased by 0.19 in. to allow for seating of the rivet heads. No allowance on the apparent elongation was made for the effect of elastic compression of the concrete. Based on an average concrete stress of 800 psi, this factor would amount to an additional elongation of 0.18 in. Because shims are used at both ends of the tendon, the net correction is divided by two to give a shim length of 3.44 in. This recommended shim length was modified in the field to correct for initial slack in certain of the tendons.

The initial stressing force, F_i, applied to the tendon was calculated from the field observations of tendon elongations and the corresponding pressure gage readings, and from the jack-calibration data furnished by the contractor. The relationship between the applied force, F, as computed from the pressure gage observations, and the observed tendon elongations, Δ_s, is shown in Figure 6. From these curves the maximum total force applied to the east girder is estimated to have been 516.3 kips corresponding to a stress of 194.7 ksi and the maximum force applied to the west girder, 512.7 kips, corresponding to a stress of 193.4 ksi. These temporary stresses while over-tensioning were slightly more than the 192 ksi allowed by the specifications and considerably higher than the 180 ksi which the contractor had proposed to use.

After holding the over-tensioning force in each tendon for a period of 5 min, shims were inserted and the jacks were then gradually released, transferring the load to the permanent anchorage devices. The initial effective prestress at the anchorage after release of the over-tensioning is designated as F_o and was not measured directly. To estimate its probable value, a semi-graphical method was used based on the tendon characteristics displayed in the initial tensioning and the final tendon elongation determined from the wedges. For example, for tendon B of the east girder, the relationship between the applied prestressing force, F, and the tendon elongation, Δ_s, was obtained for the final stress level in the girder. This relationship is depicted by the dashed line, L, in Figure 6. It was then assumed that on release of the over-tensioning force the tendon force would follow this relationship during the unloading stage. Line M defines the assumed final elongation, Δ_{sf}, of the tendon. This elongation was computed from the known value of the shim length and the manufacturer's suggested value for the tendon seating allowance. In the example, the tendon shim had a length of 3.44 in. and after deducting the 0.10 in. seating correction, Δ_{sf}, is 3.34 in. The effective initial prestress force, F_o, was found to be 85.5 kips as determined by the horizontal line N passing through the intersection of lines L and M. A similar procedure was used for each of the ten tendons. It is recognized that these procedures are somewhat approximate, but in view of the inaccuracies in the initial stress measurement inherent in the system used, a more refined method for estimating the transfer stresses did not appear to be warranted. Computed on this basis, the total effective transfer stress at the jack end, F_o, was 452.4 kips for the east girder, and 450.3 kips for the west girder. Both of these values are approximately 6 percent higher than the design value of 424 kips.

EFFECT OF CABLE FRICTION AND WOBBLE ON THE PRESTRESS FORCE

The tendon forces at the jack were determined from the applied forces and corresponding elongations at the point of jacking. It can be shown, however, that the observed stress at the jack is not applicable throughout the whole length of the tendon. For example, tendon B on the east girder is composed of ten wires, each wire having an area of 0.0491 sq in. In the companion test program the elastic modulus of the wire, E_s, was found to be 28.8×10^3 ksi. Inasmuch as the tendon was stressed simultaneously from both ends, the recorded tendon elongation at one end Δ_s, of 3.875 in. corresponds to the elongation for an effective length, L, of 602 in.; that is, the distance from the point of application of the jack to the centerline of the beam. Solving the elastic deformation equation, it can be shown that an average force of 91.0 kips is required to produce the measured elongation. However, the actual measured initial stress for tendon B, F_i, corresponding to this value was 98.23 kips. Thus it can be seen that in this particular instance the stress at the jack was about 8 percent greater than the average stress required for the desired elongation.

The difference between the average tendon stress and the stress applied by the jack is the result of tendon friction and the so-called "wobble" effect. Because the tendon is curved, it rubs against the tendon sheath as it is elongated, developing a friction force which is a function of the curvature and the coefficient of friction. Similarly, additional frictional forces are introduced due to small local dislocation of the sheath, which causes the tendon to rub against the sheath when the tendon is tensioned. The force loss due to these dislocations is called the "wobble effect." This problem has been the subject of extensive research (2, 3), and it has been found that the magnitude of the frictional and wobble effects can be approximated by an exponential expression governed by the physical characteristics of the post-tensioning system, and the length and curvature of the tendons.

In the case of a girder where all tendons have the same approximate length so that the length, L, can be assumed constant, the relationship between the applied prestress force, F_i, and the effective force, F_x, at any point a distance x from the point of application of the prestress, can be written as a function of a combined friction-curvature and wobble factor β, thus

$$F_x = F_i\, e^{-\beta x} \tag{1}$$

It has been shown (2, 3) that for small frictional losses, that is, when β is small, the tendon elongation Δ_s corresponds closely to the elongation computed on the basis of assuming the average tendon force equal to $\frac{F_i + F_L}{2}$ where F_i is the applied force at the jacking end of the tendon and F_L is the reaction at the anchorage end. In the case of a beam simultaneously stressed from both ends, F_i is the force at the jacks and F_L is the force at the assumed point of zero slip; that is, the center of the beam. For this assumption

$$\Delta_s = \frac{F_i + F_L}{2}\,\frac{L}{E_s\, A_s} \tag{2}$$

Substituting the observed values of the jacking force, F_i, and the measured elongation, Δ_s, and using the measured values of A_s and E_s, Eq. 2 was solved for F_L. Then using this value for F_x and letting x be equal to one-half the span length, the frictional loss coefficients were computed by solving Eq. 1 for β.

If the girders had not been initially over-tensioned and then released, the tendon force at any section could have been computed directly from Eq. 1, and the distribution of the prestress force along the girder would have been as shown in Figure 7(a). However, the partial release of the over-tensioning force, reduced the initial force, F_i, to the transfer force, F_o. The release of the initial tendon force reversed the direction of the frictional forces along a portion of the tendon adjacent to the jacking end. After release, the tendon force in this region is then defined by

$$F_x = F_o\, e^{\beta x} \tag{3}$$

As shown in Figure 7(b) for values of x greater than z the force profile developed during over-tensioning remains undisturbed. The value of z can be computed by solving Eq. 1 and Eq. 3 simultaneously for x equal to z. As can be seen from Figure 7(b) and (c), the effective prestressing force at any section a distance x from the point of jacking can be computed as follows:

$$\text{for } x \leq Z \qquad F_x = F_o\, e^{\beta x} \tag{4}$$

$$\text{for } x \geq z \qquad F_x = F_i\, e^{-\beta x} \tag{5}$$

Values of the tendon force, F_x, were computed on this basis at the eighth-points

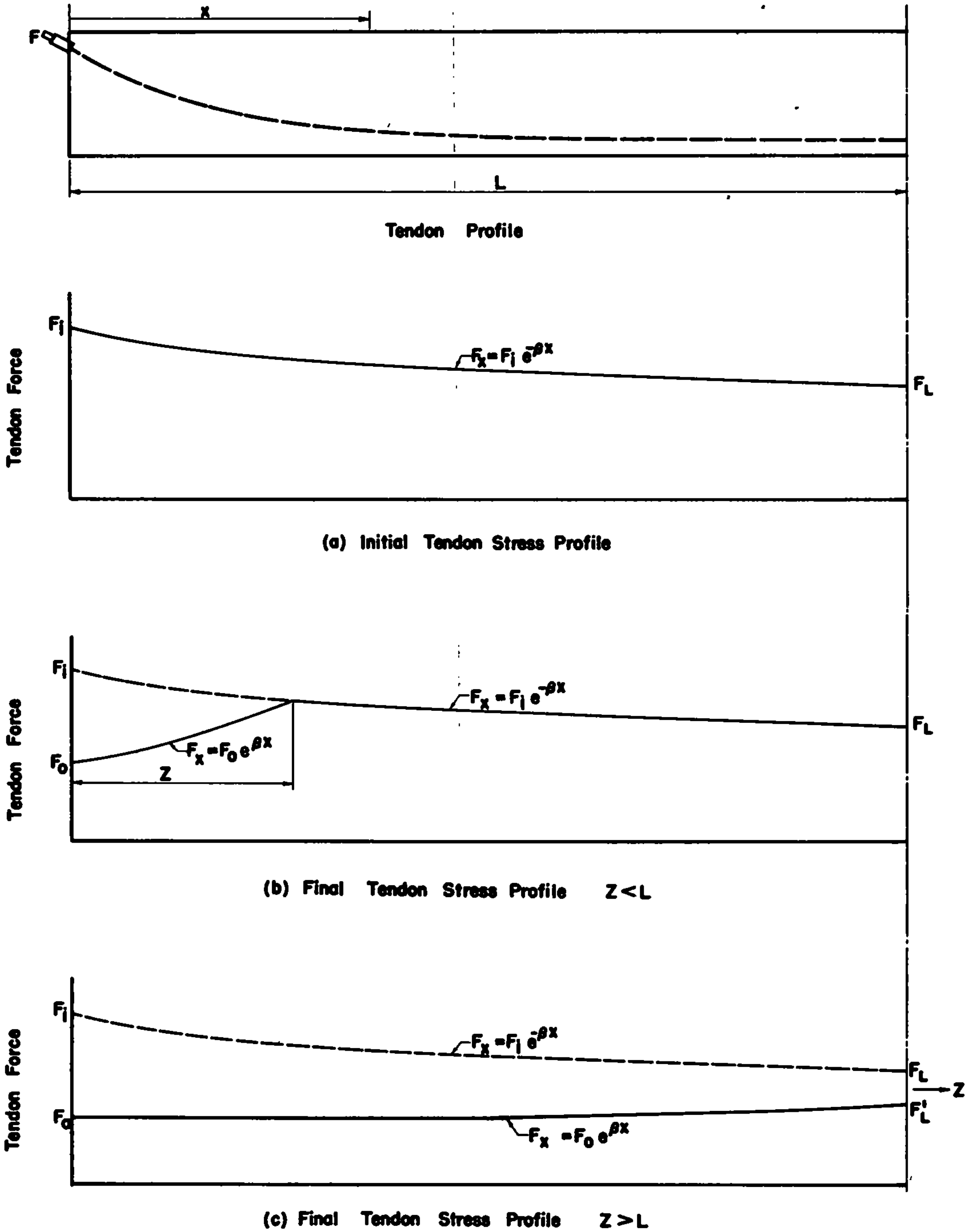


Figure 7. Typical tendon stress profiles.

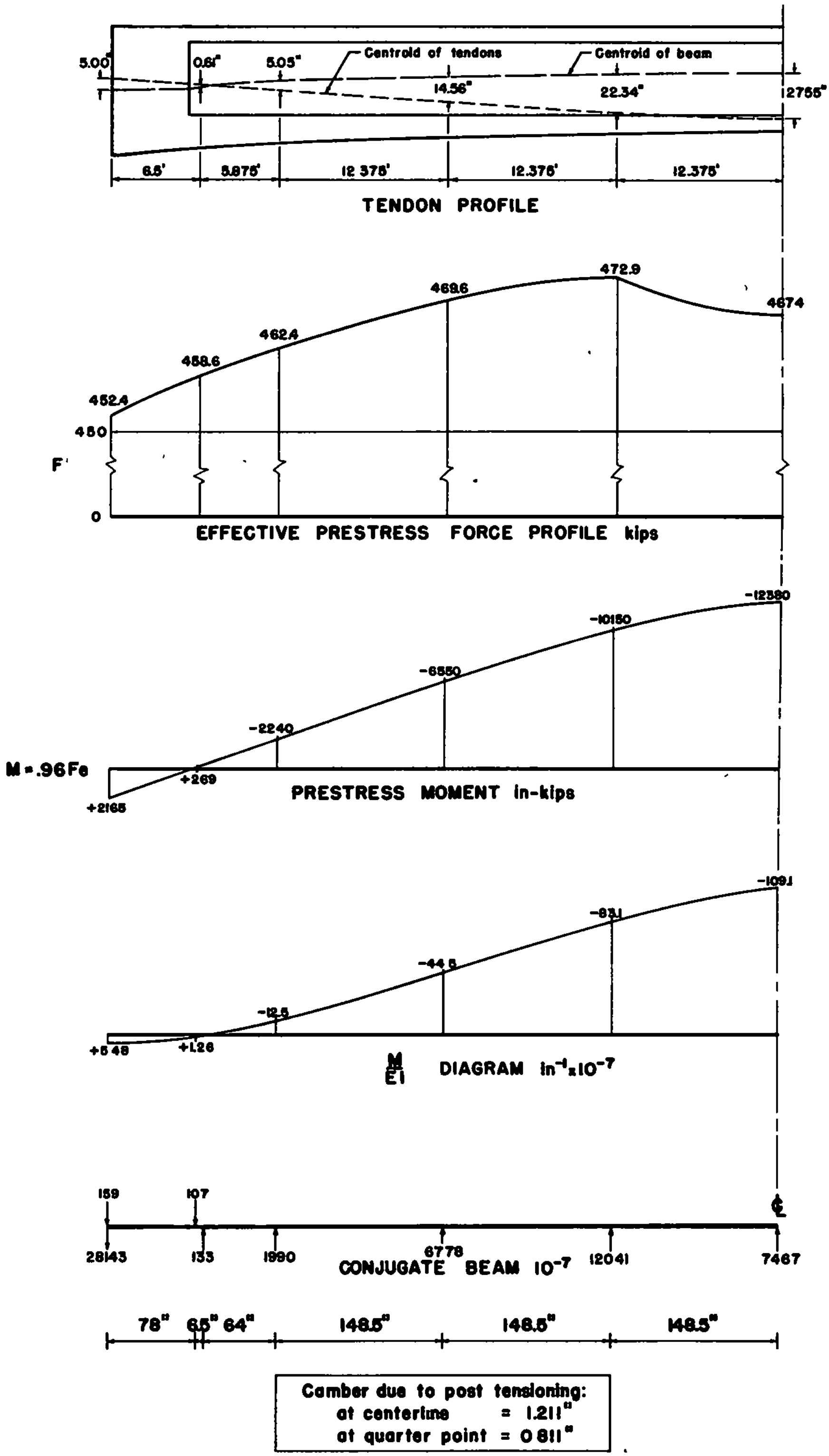


Figure 8. Prestress camber calculations.

and because of the effect of reversed friction, the majority of the tendon force profiles were similar to that shown in Figure 7(b). The combined profile for the east girder may be seen in Figure 8. Computed values of the total tendon force at initial transfer range from 450.3 kips to 472.9 kips as compared to the specified value of 424 kips. This discrepancy points out the need for careful consideration of frictional effects during the post-tensioning operations, because for structures in which the prestress camber is closely balanced by the dead load deflection an error of only 10 percent in the applied prestress force can result in relatively large deflections.

DEFLECTIONS

The girder dead load deflections were computed by the conjugate beam method using the gross section for determination of the moment of inertia, assuming an average modulus of elasticity for the concrete, E_c, of 5.3×10^6 psi and an average unit weight of 150 pcf. The computed centerline deflection was 0.960 in. and the quarter-point deflection, 0.667 in., for the dead load of the girder alone.

The deflection due to the magnitude and eccentricity of the prestressing force was also calculated by the conjugate-beam method. The calculations for the east girder are shown in Figure 8. The calculations are based on an applied prestress moment which was calculated from

$$M_x = 0.96 \; F_x e \tag{6}$$

in which

- e = tendon eccentricity as shown in Figure 8:
- F_x = effective prestress force as calculated by Eqs. 4 and 5; and
- 0.96 = a factor introduced to correct for the loss in prestress due to elastic shortening of the girder when subsequent tendons were stressed. The value of the factor was determined by superposition of strains at the tendon centerlines.

The computations shown in Figure 8 show that the effect of the prestress would result in a 1.211-in. camber at the centerline of the east girder and a 1.196-in. camber in the west girder. The computed camber at the quarter-points was 0.811 in. and 0.800 in., respectively.

The actual computed deflection or camber is the algebraic sum of the computed dead load deflection and the computed prestress camber. The computed net camber is therefore 0.251 in. and 0.236 in. at the centerline of the east and of the west girder, respectively, and 0.144 in. and 0.133 in. at the respective quarter-points. The correlation of these results with the actual measured values is shown in Figure 9. Considering that the total range of the camber and the deflection calculated for the centerline of the east girder is the sum of 1.211 in. and 0.960 in., or a total of 2.171 in., the deviation of the measured value + 0.292 in. from the computed value 0.251 in. represents an error of less than 2 percent. Similar checks showed that the greatest deviation at any point was only 4.8 percent.

In view of this close agreement between the computed and the observed deflections, the procedure outlined is believed to present a satisfactory method for computing elastic deflection or camber in a statically-determinate prestressed concrete beam.

Because time-dependent inelastic phenomena such as creep and shrinkage are very important factors in prestressed concrete, they must be considered in the prediction of the resultant net deflection after a specified time interval. Lin (2) proposes a formula of the form

$$\Delta_t = C_t \left(\Delta F_t + \Delta_{DL} \right) \tag{7}$$

in which

- Δ_t = deflection at any time, t, after stressing;
- C_t = creep coefficient expressing the relationship between the deformation at a particular time, t, and the initial elastic deformation;

Δ_{Ft} = deflection due to the effect of effective tendon force at time, t; and
Δ_{DL} = deflection due to the effect of dead load.

This method assumes that the deflection at a particular time may be expressed as a ratio of the net effect of the tendon force acting at that time and the dead load of the member. From the general nature of creep deformation in concrete it is known that a substantial portion is not generally recoverable (4). In cases in which relatively large losses in the effective tendon force take place gradually, substantial creep effects are developed at a relatively early stage. The proposed formula ignores these effects in computing long-time deformations.

If no cognizance were taken of the effect of the tendon stress losses, the deflections could be predicted by the formula

$$\Delta_t = C_t\,(\Delta_{F_0} + \Delta_{DL}) \qquad (8)$$

in which Δ_{F_0} is the deflection due to the initial effective tendon force. Obviously this method is in error due to the known loss in camber resulting from tendon stress loss; these values can be regarded, however, as representing the extreme upper limit for residual camber. The previous example can be considered to give the extreme lower limit for deflection of the member. The actual deflection pattern should fall somewhere in between these limits.

If it were possible to express the creep coefficient and the tendon stress losses as time functions, then a theoretically exact solution could be obtained by the use of difference methods. The creep coefficients and the various tendon stress loss components can however, at best, only be determined in an approximate manner. The average of these two limits was therefore used as a reasonable method for predicting the resulting deflections. This approximation results in the expression

$$\Delta_t = C_t\left(\frac{\Delta_{F_0} + \Delta_{F_t}}{2} + \Delta_{DL}\right) \qquad (9)$$

when Eq. 9 is further modified to include the deflection due to loads applied for short-time periods or after most of the creep losses have taken place,

$$\Delta_t = C_t\left(\frac{\Delta_{F_0} + \Delta_{F_t}}{2} + \Delta_{DL}\right) + \Delta_{AL} \qquad (10)$$

in which

Δ_{AL} is the deflection due to short time loads.

In Figure 9 the measured girder deflections are compared to the theoretical values as computed from Eq. 10 using the data for the east girder. The creep coefficients, C_t, were obtained from an assumed creep curve based on the shape of an idealized curve recommended by Lin (2), but modified for an over-all creep coefficient of 3.0 in accordance with normal design assumptions. The mangitudes of the tendon relaxation losses were based on the manufactuer's recommendations, with a maximum value of 4 percent. Tendon stress losses, ΔF_e, due to elastic shortening were based on an assumed average uniform allowance of 4 percent as previously explained, and the stress loss, ΔF_c, due to the effect of concrete creep was calculated from the formula

$$\Delta F_c = (C_t - 1)\,\frac{f_{c_{ave}}\,E_s\,A_s}{E_c} \qquad (11)$$

in which $f_{c_{ave}}$ (the average concrete compressive strength over the entire girder) was determined as 820 psi by averaging the values of the calculated stress profiles. The theoretical deflection values computed on the basis of these assumed tendon stress losses are in good agreement with the measured deflections for the initial 150 days prior to construction of the deck slab and appear to confirm the assumptions in Eq. 8.

The construction operations involved in the casting of the deck slab, removal of the shores, and placement of the hand railing took place over a period of several weeks and for the purpose of simplicity the over-all deflection due to these various operations is shown as acting at 150 days. The actual schedule consisted of placing of the deck slab on day 146, removal of the shores on day 153, and completion on day 194.

The theoretical deflections due to the weight of the cast-in-situ slab on removel of the shores were computed on the basis of full continuity of the precast girders for the weight of the deck slab only at the time of placement of the deck slab, and for full composite action between the girders and the deck slab at the time of removal of the shores. The observed deflections were of a smaller magnitude than those indicated by the computations.

A further discrepancy in the readings was caused by the construction procedure required in preparing the girders to receive the deck slab. In erecting the form-work for the deck slab and girder-connecting diaphragms, the girders were aligned vertically with crossed guy wires to enable correct placement of the forms. This alignment was performed between the set of readings taken at 139 days and those taken at 146 days. A close inspection of Figure 9 shows that the east girder stations were depressed and the west girder stations were elevated approximately equal amounts during that period. This construction practice resulted in the shifting of a portion of the load from one girder to the other. The effect of such corrections is almost impossible to predict, and interpretation of the data is extremely difficult.

However, it is possible to obtain a qualitative appraisal of the effectiveness of shoring to take advantage of composite action for deflection control. Had the deck slab been carried by the non-composite girders (that is, no shoring) the deflection at the center point would have been 0.337 in. whereas the actual values measured with composite action were less than 0.2 in. However, it is important to note that these data are still inconclusive because of the fact that elevation changes of the deflection stations (which were also the location of the shoring points) occurred with the shores in place. This

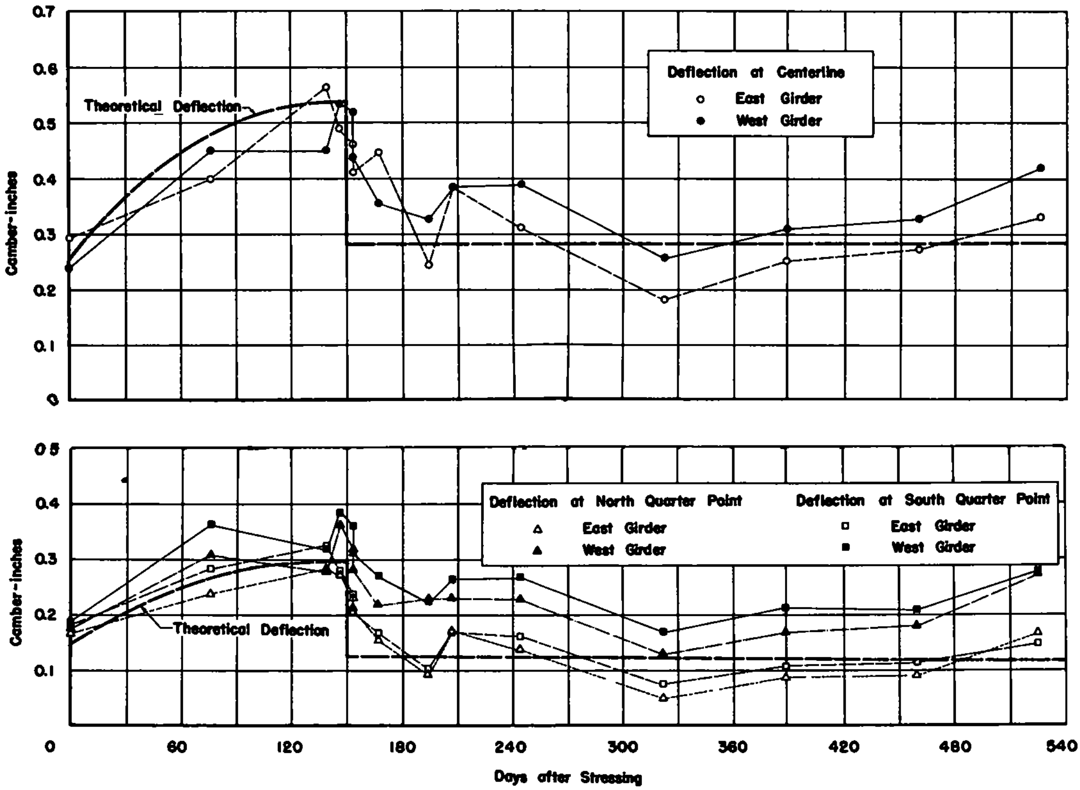


Figure 9. Girder deflections.

indicated that the shores yielded somewhat, thus preventing development of full continuity. Thus it is apparent that if this technique is to be used successfully, careful attention to minor construction details is imperative.

The final variable affecting deflections is the effect of differential shrinkage between the cast-in-situ slab and the original girders. The approximate magnitude of this effect was calculated by assuming that because the girders restrain shrinkage in the slab, tensile stresses will be produced in the slab along the slab-girder junction. Inasmuch as these stresses would be relieved by local cracking, their average value would be one-half of the tensile capacity of concrete. Assuming 250 psi for this value, an effective force equal to this average stress over the slab cross-section was computed and assumed to act at the slab centroid. The resulting moment about the centroid of the composite section was computed, and the shrinkage deflection at the centerline was calculated as 0.088 in. It appears likely that this factor accounts for at least a portion of the deflection noted in the period between 153 and 322 days after stressing.

CONCRETE STRAIN MEASUREMENT

Because it was possible to measure the concrete strains at several sections of the girders after the stressing of each tendon was completed, additional information as to the behavior of the members during the post-tensioning operation was obtained. The theoretical stress distributions were computed on the assumption that the girder behavior was in accordance with the generally accepted elastic theory. These computed stresses were then transformed into equivalent strains using $E_c = 5,300,000$ psi as found in the companion test program. Values were calculated for the stress and strains with:

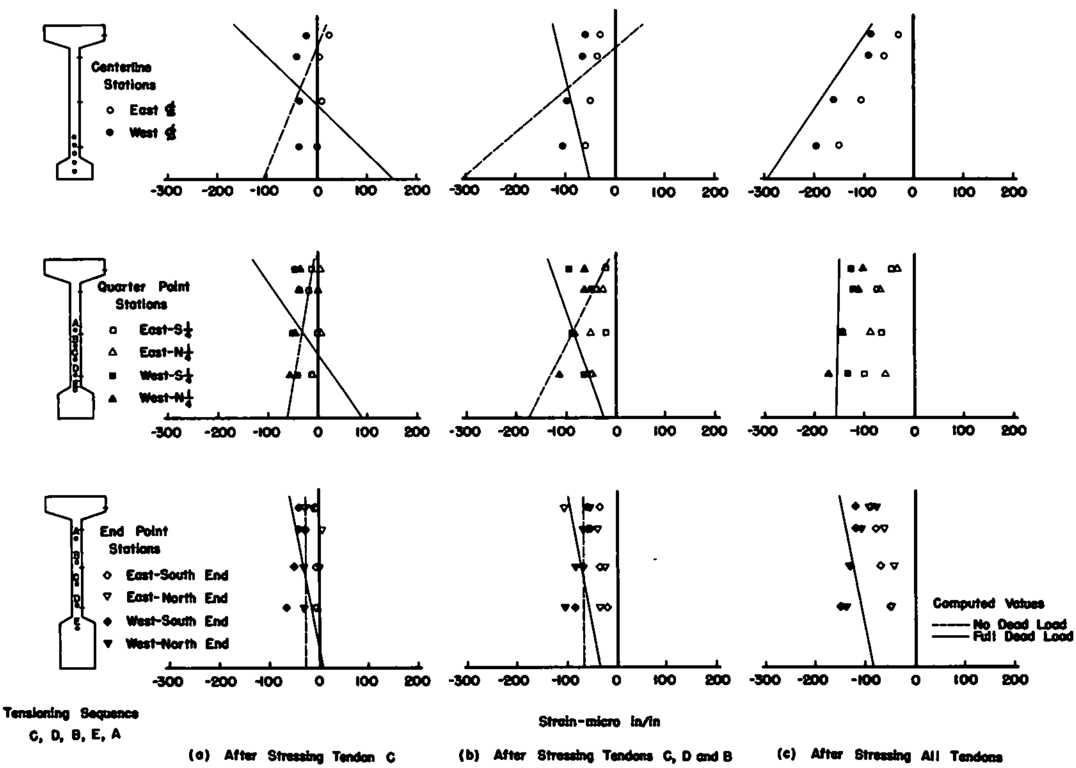


Figure 10. Prestress strain profiles.

1. Tendon C stressed — no dead load acting;
2. Tendon C stressed — full dead load acting;
3. Tendons B, C, and D stressed — no dead load acting;
4. Tendons B, C, and D stressed — full dead load acting; and
5. Tendons A, B, C, D, and E stressed — full dead load acting.

The computed strain profiles are shown in Figure 10 together with the corresponding measured strains.

It is apparent that the measured strains for the west girder are in much better general agreement with the theoretical strain profiles than the measured strains for the east girder. A slight lateral bowing of the east girder was noted during the post-tensioning operation, and it is possible that the resulting secondary strains modified the values of the principal longitudinal strains. Another possible explanation for the discrepancy is that in some way the effective prestress force applied to the west girder was greater than the force applied to the east girder. The stressing procedures were identical in nature; discrepancies noted were minor and the observed vertical movements of both girders were in reasonably good agreement. It should be observed that, in general, the measured strain profiles after partial stressing of the girders fell between the computed strain profile for the case of zero dead load and the profile based on the full dead load acting. Thus, it appears that an increasing portion of the dead load was effective for each prestress increment, even though the exact amount could not be ascertained. It should be noted that no significant tensile stresses due to dead load were developed during the stressing operation because the soffit forms provided partial support until the full dead load was transferred to the stressed tendons.

After completion of the prestressing, further strain measurements were taken before and after all major construction operations and on an intermittent time schedule after final completion of the bridge. Typical examples of the observed strain profiles are shown in Figures 11, 12, and 13. These measurements confirm Navier's hypothesis

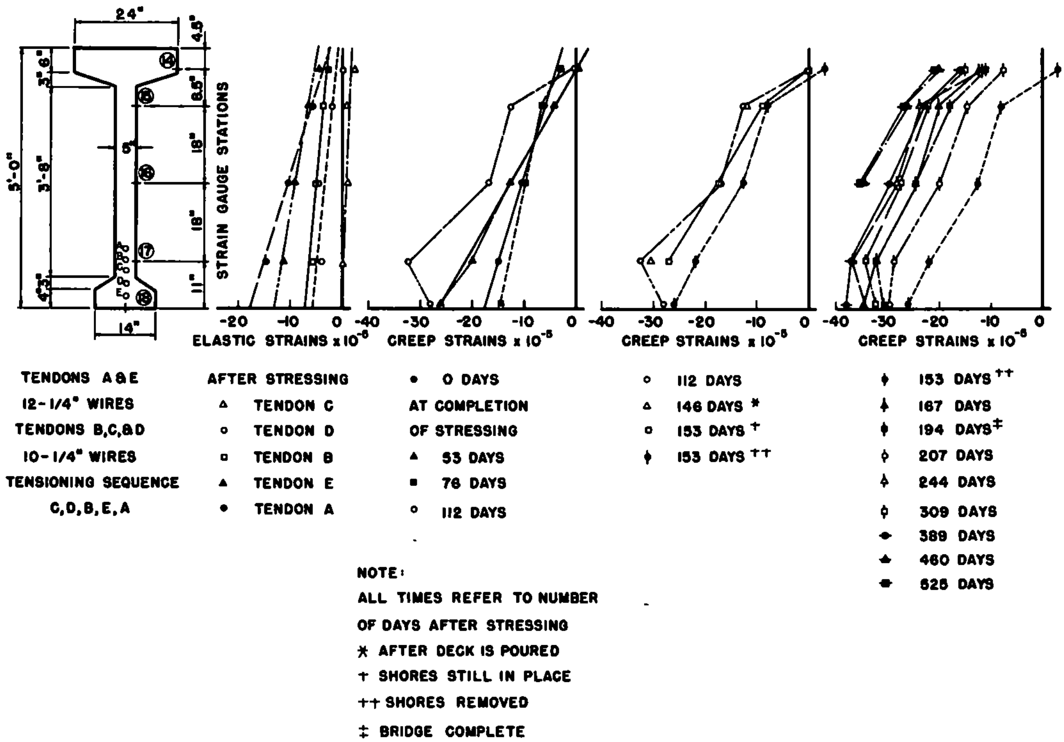


Figure 11. Strain profile at center of east girder.

because it can be seen that the observed strain distributions were reasonably linear and remained linear even after considerable inelastic deformation.

In general, the longitudinal strain measurements subsequent to post-tensioning reflected principally the effect of creep and shrinkage and, to a lesser extent, the effect of the composite action. Because the stress and, hence, the strains produced by placement of the deck slab and of the hand railing were of a comparatively small magnitude, it was difficult to correlate in anything more than a general way the observed readings with the computed data. Although the girder strains were relatively unaffected by the placement of the slab, and all stations showed the influence of a positive bending moment on removal of the shores, there apparently was an almost immediate strain recovery following this construction operation because the strain readings returned to the same general pattern that they had been following. It may be concluded that for materials subjected to appreciable inelastic deformations, the effects of additional small elastic deformations are quickly suppressed by the inelastic effects so that it becomes almost impossible to separate the effects of the elastic and inelastic properties.

The longitudinal strain measurements also tend to emphasize the reduction of the inelastic effects of creep and shrinkage with time. It may be observed that the major portion of the inelastic changes took place during the first year after stressing and that subsequent changes were a great deal smaller. This result is in accordance with accepted theories of creep and shrinkage.

STRAIN MEASUREMENTS IN THE DECK SLAB

The inclusion of strain gaging stations in the deck slab provided an opportunity to evaluate the effectiveness of composite action between the girders and the slab and also to observe the magnitude of differential shrinkage present. The variation of the longitudinal strains along the centerline of the bottom surface of the cast-in-situ slab

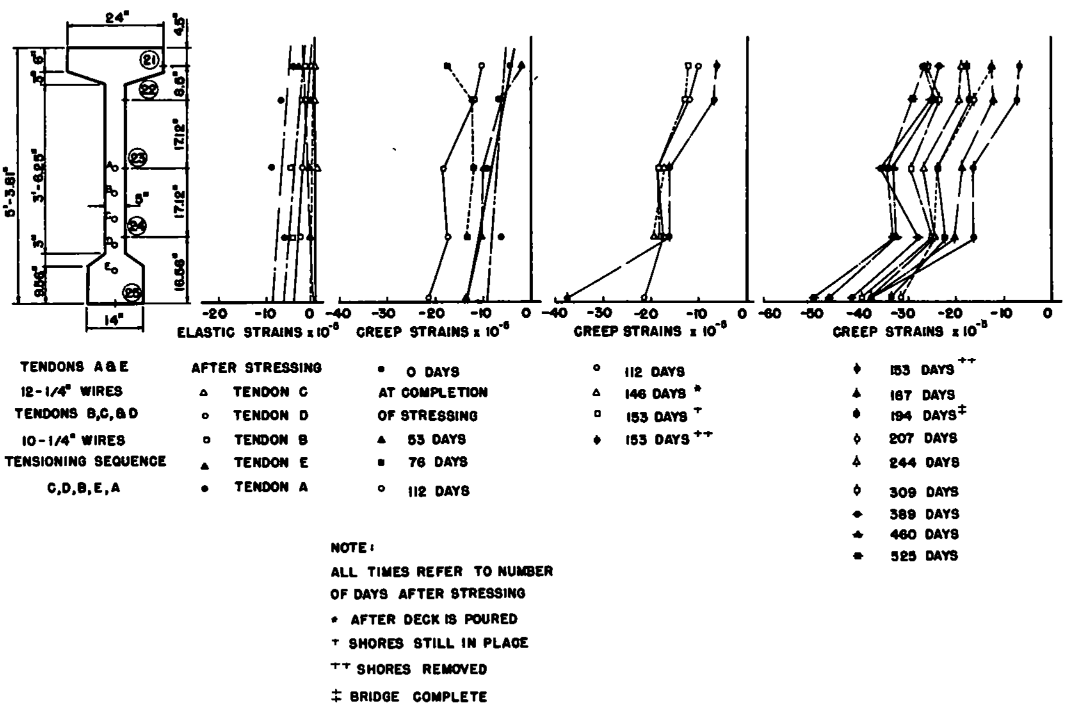


Figure 12. Strain profile at north quarter point of east girder.

are plotted in Figure 14. It should be noted that immediately on removal of the shores (153 days after stressing of the main girders and 7 days after pouring of the cast-in-situ slab) all of the stations along the centerline of the lower surface of the deck slab indicated tensile strain. At the corresponding points on the top surface, the slab was subjected to compression. According to conventional elastic theory, all stations should have registered compressive strain on removal of the temporary shores, because the centroid of the composite section lies below the bottom surface of the deck slab. This discrepancy may be due to localized bending in the longitudinal direction coupled with larger localized transverse bending of the slab due to unequal deflection of the two girders on removal of the shores. However, because no positive explanation was noted, the applicability of conventional composite action theory in this case is therefore questionable.

The effects of shrinkage of the cast-in-situ slab became immediately apparent, however, because subsequent readings at all stations indicated compressive strains which grew in magnitude with a diminishing time relationship. It is extremely interesting to note that the magnitude of the observed strains after 500 days is in very close agreement with the predicted value of 195 microinches per in. which was obtained from a series of shrinkage tests on prisms cast from the same concrete as the deck slab (1).

GENERAL SUMMARY

This report describes a program for a field study of the behavior of a prestressed concrete pedestrian overpass. The purpose of this study was to develop information which might be useful in the design and construction of future prestressed concrete structures. The field study was accompanied by a companion test program to determine the physical characteristics of the materials used in the actual construction. The results of the test on the samples of steel used in the tendons indicated substantial agreement with the elastic properties claimed by the manufacturer. However, field measurements and tests on samples of the concrete used in the girders indicated sizeable differences in the quality of the concrete used in various parts of the girder. These differences made the correlation of observed and theoretical values, influenced by such factors as the modulus of elasticity, extremely difficult.

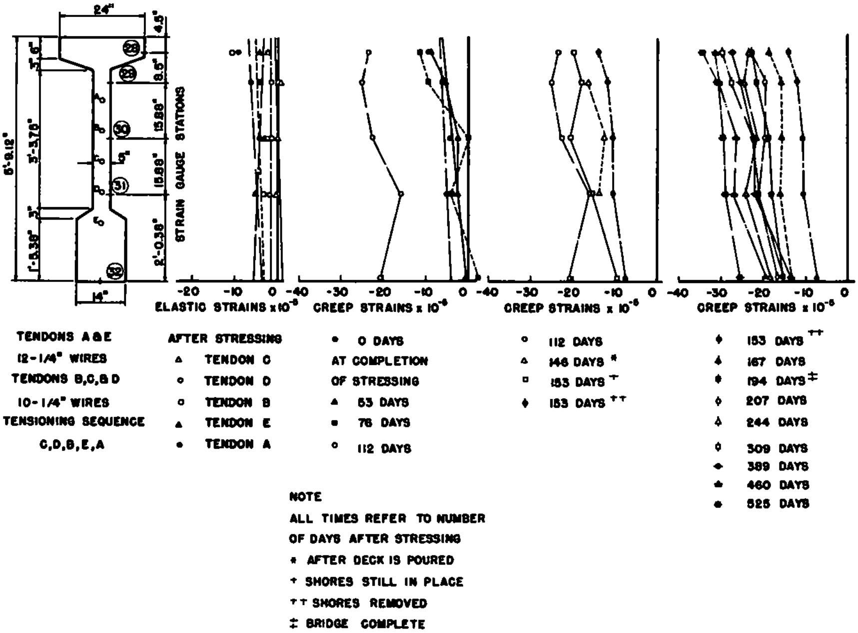


Figure 13. Strain profile at north end of east girder.

Jack pressure and tendon elongation measurements made during the post-tensioning operation indicate that the girders may have been slightly overstressed, due to the fact

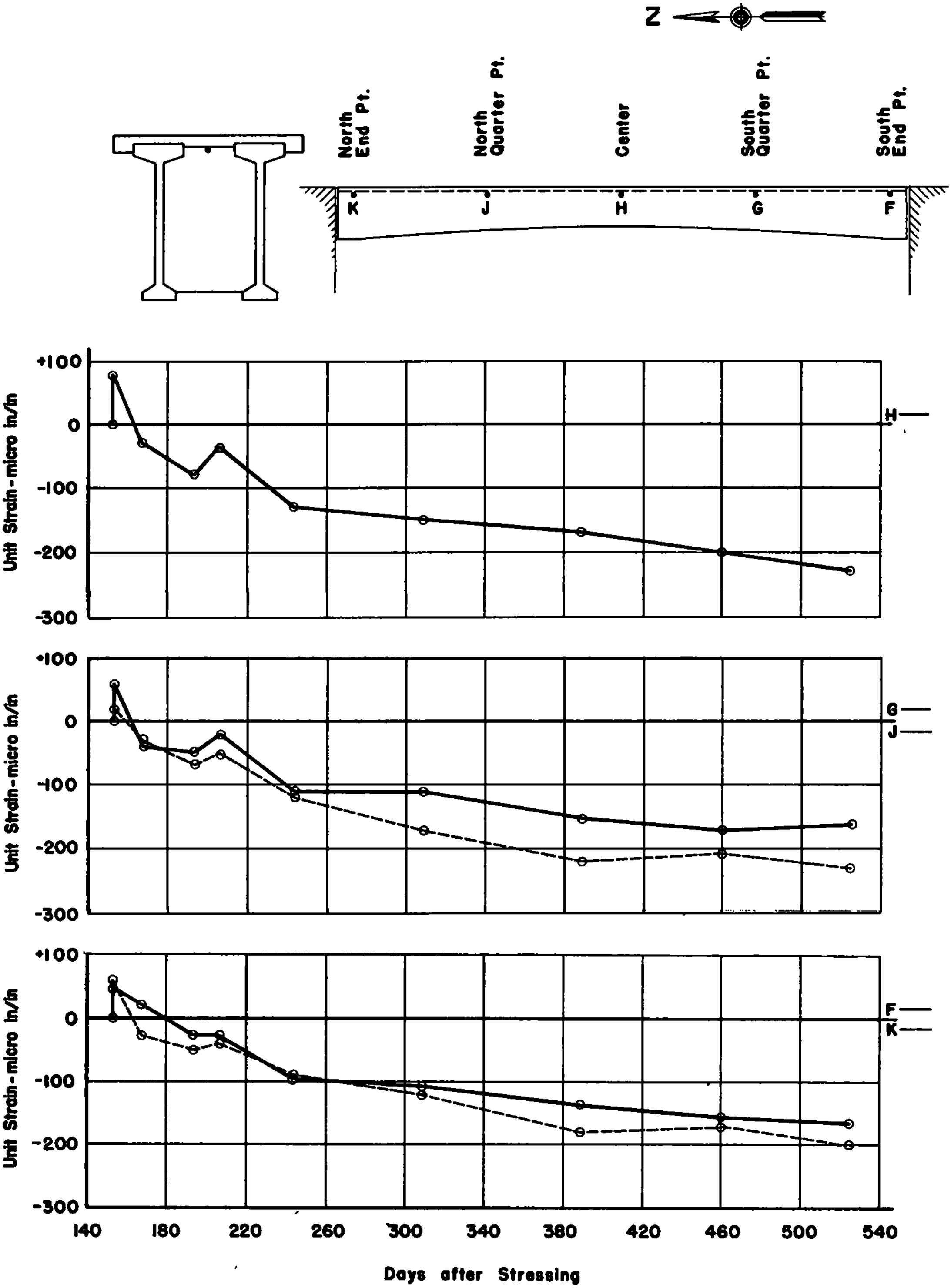


Figure 14. Strains in bottom of deck slab.

that the frictional characteristics of the tendons were not considered. The magnitude of the prestressing force was controlled by measurement of tendon elongation; the observed force, based on the hydraulic jack calibration, was used for reference purposes only. The analysis of the observed strains and deflections during the post-tensioning operation was found to be complicated by the uncertainty involved in the transfer of the girder dead load from the soffit form to the tendons. It is significant to note, however, that no appreciable tensile strains were measured at any section of the girder during the post-tensioning operation. Even though small tensile strains were noted after the stressing of the first tendon, these strains were reversed on the application of additional prestress.

The deflection measurements during the period in which the cast-in-situ slab was placed, revealed that the construction procedure used introduced strains and deflections which were not considered in the design assumptions. The assumption made was that continuity would be developed for the dead load of the slab only. However, in setting up the slab and diaphragm forms and prior to the placing of the deck slab, the girders were realigned by means of crossed wires to permit leveling of the forms. This construction practice resulted in the shifting of a portion of the load from one girder to the other girder. Furthermore, the deflection profiles indicated that the girder elevations decreased at the support points between the time of placing of the concrete and the removal of the shores. This decrease indicated that the shores yielded under the load of the cast-in-situ slab, and hence, a loss of continuity occurred.

Development of composite action between the cast-in-situ deck slab and the precast girders was confirmed by the observed deflection measurements. The measured deflections were in much closer agreement with the predicted values for composite action than they would have been had no composite action been assumed. The effectiveness in reducing the dead load deflections and in equalizing small discrepancies in initial camber of the prestressed girders by shoring the girders before placing of the deck slab was established. Finally, the time-dependent behavior of the structure was found to be predictable from a consideration of the material properties and the applied loads. With proper assessment of the tendon losses, due to such factors as elastic shortening, shrinkage, creep of the concrete, and relaxation of the steel, the deflection of a structure at any given time may be estimated by

$$\Delta_t = C_t \left(\frac{\Delta_{F_0} + \Delta_{F_t}}{2} + \Delta_{DL} \right) + \Delta_{AL} \qquad (10)$$

The deflection components used in Eq. 10 may be calculated by the conjugate beam method. Using the measured values of the prestress force and the elastic modulus, and with the moment of inertia based on the gross concrete section, the deflections predicted by Eq. 10 were found to be in good agreement with the observed values. The effect on the deflection of the differential shrinkage between the concrete in the girders and that in the cast-in-situ slab was found to be one of the most difficult variables to assess. An approximate method for evaluating this factor was developed, and although the measured deflections tended to confirm the theory, it was not conclusively proven.

CONCLUSIONS AND RECOMMENDATIONS

Although the test program was extremely limited in nature, because in effect it involved only the use of a single test specimen, the following conclusions are believed to be warranted:

1. The over-all behavior of the structure was found to be in reasonable agreement with the design assumptions used in general elastic theory.
2. The actual effective tendon force was approximately 10 percent higher than the design value due to the effect of tendon wobble and friction.
3. The assumption that the tendon losses may be approximated as 15 percent of the initial prestress force was confirmed by a comparison of the measured deflection with the calculated values based on the actual properties of the materials used in the structure.

4. The technique of providing temporary shores for the girders during the placing and curing of the cast-in-situ slab was found to be effective in developing composite action before transfer of the slab dead load and in equalizing minor differences in the camber of the precast girders. However, it was noted that careful attention to construction details is required to insure that the designer's assumptions will be fully justified.

ACKNOWLEDGMENTS

The work reported herein was supported by the Missouri State Highway Commission and by the Bureau of Public Roads under a cooperative research agreement. The assistance of the personnel of District 6 of the Missouri State Highway Department in facilitating the field operations was greatly appreciated. Also the assistance of James M. Belling, Malise J. Graham, and Phillip H. Huff, graduate research assistants, in the performance of the testing and in the reduction of the data is gratefully acknowledged.

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HRB:OR-464