

independent examination and verification of its unrestricted availability for such use, and a determination of suitability for the application by professionally qualified personnel. No license under any United States Steel Corporation patents or other proprietary interest is implied by the publication

of this paper. Those making use of or relying on the material assume all risks and liability arising from such use or reliance.

Publication of this paper sponsored by Committee on Steel Bridges.

Prestressed Steel Beam-Concrete Slab Composite Bridge Units for County Road Use

THOMAS HENDRICK, CLIFFORD CLOTTEY, and THOMAS M. MURRAY

ABSTRACT

The results of the first three phases of a research project to study the behavior of a prestressed steel beam-concrete slab bridge unit specifically designed for county road use are presented. The units are constructed by steel fabricators in an upside-down position, trucked to the site, and turned over and set in one operation. During fabrication, heavy steel forms are hung from the beams and their weight plus the weight of the concrete provide the prestressing. The advantages of this type of construction are that it requires less structural steel, less reinforcing steel, and less concrete for the deck. In addition, because the units are prefabricated, the labor force and the construction time at the job site are reduced. Data are presented on concrete, reinforcing bar and steel beam strains, and vertical displacements for a 1-year period of observation with the unit under sustained loading. Behavior of the unit when subjected to 500,000 cycles of repeated loading is described. Data from a static 50 percent overload test are also presented. The test unit performed satisfactorily, and county road bridges are currently being constructed in Oklahoma using three or four of the units. An additional 2-year period of sustained load observation followed by static tests to failure is planned.

A research program is being conducted at the Fears Structural Engineering Laboratory, University of Oklahoma, under the sponsorship of the Oklahoma Department of Transportation, to study the strength and stiffness characteristics of a full-scale precast, prestressed steel beam composite bridge unit. Two, three, or four of the units can be used to construct county road bridges for small stream crossings. The test unit is 55 ft long and 6 ft 9.5 in. wide and consists of two W21x50 steel beams and a

7.5-in.-thick reinforced concrete slab and weighs 39,000 lb. The test unit was designed for AASHTO HS 20 loading with a 100,000 cycle fatigue rating.

The objective of the study is to experimentally investigate the behavior of the unit under various types of loading: sustained load, repeated load, and static loading to failure. The research project will take 3.5 years; it began in April 1982 with completion scheduled for December 1985. The project consists of six phases: 1 year of observation under sustained load; 3 months of repeated loading (500,000 cycles); operating rating loading; 2.5 years of observation under sustained dead load; static flexural test to failure; and transverse slab strength tests. As of this date (December 1983), the first three phases of the research project have been completed. Results are summarized herein.

BRIDGE UNIT SPECIMEN

The unit was fabricated by Robberson Steel Company, Oklahoma City, Oklahoma, using design drawings provided by Grossman and Keith Engineering Company, Norman, Oklahoma. The concrete was cast with the beams in an upside-down position using a heavy steel form hung from the beams as shown in Figure 1. When the concrete had been poured and finished, an additional steel weight was placed on the beams. After the concrete had cured, the weight and the forms were removed. The concrete was poured on April 1, 1982, and the forms were removed on April 8, 1982. The unit was moved to the Fears Structural Engineering Laboratory on April 8, 1982, in an upside-down position using a "pole" truck. The unit was turned over on delivery and was placed on small support beams on an existing concrete slab outside the laboratory. Elastomeric bearing pads were placed between the support beams and the unit beams. The bridge unit and the control cylinders were subjected to prevailing weather conditions. Three weeks after the unit was poured, 4 x 8 x 16-in. concrete blocks weighing 33 ± 0.1 lb each were placed on the unit to simulate dead load of 36 psf from an asphalt overlay.

Concrete

An air-entrained concrete with a design strength of

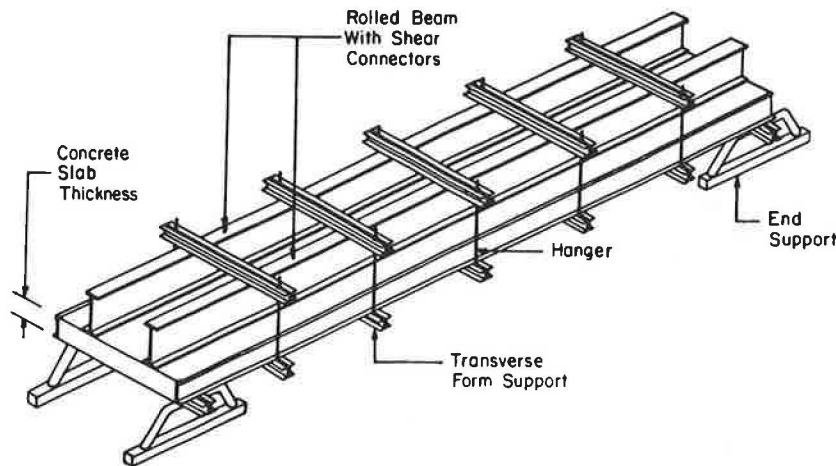


FIGURE 1 Method of fabrication of bridge unit.

5,000 psi and a design slump of 1 to 3 in. was used for the unit. The concrete was obtained from a local ready-mix plant and was manufactured with Type III portland cement and normal weight aggregate having a maximum size of 1 in.

During the pouring of the concrete, a total of 50 control cylinders, 6 in. diameter by 12 in. high, were cast from two mixing trucks. Slump tests were performed before the cylinders were cast. Water was added at that time and additional mixing was done until the appropriate slump was achieved. Then cylinders were cast for approximately each half of each truck designated as Series 1A and 1B for the first truck and Series 2A and 2B for the second truck. Because of high slump obtained for the second half of the second truck, an additional ten cylinders were cast for that half (Series 2C). The amount of entrapped and entrained air was determined using stereo-optical microscope scanning. The slump, air content, age, and strengths of concrete are given in Table 1.

Steel

The two steel beams were standard W21x50 sections of

A588 steel. Deformed steel bars of Grade 60 steel were used as reinforcement for the concrete slab deck. Stress-strain curves for the reinforcing bars were obtained from tension tests of coupons supplied by the fabricator.

Fabrication

Shear connectors were welded to the W21x50 beams before the beams were placed in the forms. The shear connectors were standard steel studs. Pairs of studs were welded to the top flange of each beam with variable spacing. The spacing varied from 12 in. at each end to 30 in. at the midspan. In addition, seven C7x9.8 by 6-in. long channels were welded to the top flange of each beam at 9-ft intervals. These channels were used as spacers between the top flange of each beam and the steel form to ensure uniform thickness of the concrete slab deck.

The beam sections were connected together at the ends and third points with a welded diaphragm consisting of 3 x 3 x 0.25 angles. Center-to-center spacing of the beams was 3 ft 9.5 in.

TABLE 1 Concrete Properties—Bridge Unit

Properties	Age (Days)	First Truck		Second Truck		
		First Half (1A)	Second Half (1B)	First Half (2A)	Second Half (2B)	Second Half (2C)
Slump (in)	-	1½	2¼	1 3/4	4¼	4¼
Air Content %	-	5.4	5.4	5.8	5.8	5.8
Entrained Air	-	4.5	4.5	5.0	5.0	5.0
Entrapped Air	-	0.9	0.9	0.8	0.8	0.8
Compressive Strength (Psi)	1	2970	2370	3060	2430	2150
	4	4320	4240	4240	3890	3860
	8	4750	4540	4890	4100	4100
	21	5450	5215	5720	4840	5040
	29	5730	4950	5700	5290	4830

The concrete slab was reinforced with No. 4 bars both longitudinally and transversely. The longitudinal bars were spaced at 9 in. at the top and at 18 in. at the bottom of the slab. The closer spacing at the top was intended primarily to minimize creep strains in the concrete slab. Both top and bottom transverse reinforcement was placed at variable intervals symmetrically about the midspan. Starting from the midspan and going toward each end, the spacings were 5.5 in., 11 in., and 16.5 in. for each one-sixth of the span length. The intention of this variable spacing was to study at a later date the influence of transverse reinforcement on the transverse strength of the concrete slab.

The top and bottom slab reinforcement mats were preassembled and tied using standard practices in a specially constructed layout template. All of the longitudinal reinforcing bars were single length except for those that were strain gauged. The longitudinal bars with strain gauges were lap-spliced at the third points with a splice length of 2 ft 6 in. The reinforcing mats and the beam assembly were then hoisted into a well-oiled steel form in the upside-down position. The formwork, which was simply supported at the ends, was then hung from the steel beams at 9 ft 0 in. intervals with hangers as shown in Figure 1. The concrete was then poured and additional weight (approximately 7,000 lb) was placed on the beams at midspan. Shims were used at midspan to limit vertical deflection to 3.5 in.

The concrete slab and the control cylinders were moist-cured in the formwork for 7 days. The exposed surface of the concrete was covered with wet burlap and polyethylene sheets during the 7-day curing period. On the eighth day, the forms were stripped and the specimen was transferred in the upside-down position from the fabrication shop to the Fears Structural Engineering Laboratory where it was turned over and set on supports as previously described.

Instrumentation

With the exception of the shear connectors, all components of the bridge unit, reinforcing steel, steel beams, and concrete slab were fully instrumented. Ten longitudinal reinforcing bars, five top and five bottom, were strain gauged longitudinally at the midspan with electrical resistance strain gauges having gauge lengths of 0.25 in. Similar gauges were mounted on six transverse reinforcing bars, three top and three bottom. The instrumented transverse bars were located in the different spacing regions discussed previously.

Before placing the steel beams in the concrete form, eight 0.5-in. gauge length strain gauges were installed on the flanges of each beam near midspan. The gauged cross section was 8 in. from midspan to avoid effects induced by a spacer channel located at midspan.

After the bridge unit was removed from the forms but before the unit was turned over, two 10-in. gauge electrical extensometers were mounted on the top and the bottom surface of the concrete slab at midspan and along the longitudinal axis of the unit.

After the bridge unit was turned over, six concrete strain gauges were mounted longitudinally at midspan and on the top surface of the concrete slab. Initially, four strain gauges with 2-in. gauge length were mounted. These were later replaced with two strain gauges with 4-in. gauge length.

When the bridge unit had been set in place at the laboratory, two direct-current differential transformer deflection transducers were positioned to measure the midspan deflections of the slab near the

top flange of each beam. To determine the influence of temperature, a thermometer bulb was embedded in the concrete slab.

To accumulate creep data, two switch and balance units and two strain indicator units were permanently wired to the strain gauges to make strain measurements. A DC voltage supply and standard voltmeter were used to monitor the deflection transducers. The entire set of measuring instruments was stored in a refrigerator (set at 70°F) in a temporary building adjacent to the test unit.

SUSTAINED LOADING TEST

Creep Measurements

Before the unit was turned over, initial readings were taken from the two extensometers, the eight strain gauges on the beams, and the ten strain gauges on the longitudinal reinforcing bars. After the bridge unit was turned over, readings from the extensometers and the strain gauges were again taken to determine the strains due to the turning operation. Two weeks after the specimen was turned over, final instrumentation of the specimen was completed. Additional dead load was applied when the specimen was 21 days old.

Daily readings were taken for the first 60 days of the sustained loading tests. Thereafter, readings were taken weekly. Strain data from the ten longitudinal reinforcing bar strain gauges, eight beam strain gauges, two concrete strain gauges, and two extensometers were recorded. In addition, output from the deflection transducers and temperatures were recorded at the same time. Readings were taken between noon and 4 p.m. and weather conditions were recorded each time data were taken.

Test Results

The creep data taken for the first year are presented graphically in Figures 2-6. Stress data were obtained from the strain data using Hooke's law (assuming perfectly elastic material) and an assumed modulus of elasticity of 29 million psi for steel.

The average midspan deflection of the bridge unit and temperature changes are shown in Figure 2. Variations in the concrete strain at midspan along the longitudinal axis of the bridge unit are shown in Figure 3. Stress variations in both the top and bottom reinforcements along the longitudinal axis of the bridge unit are shown in Figure 4. Figure 5

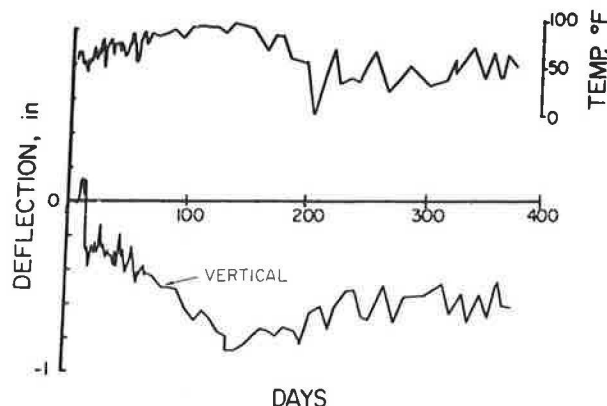


FIGURE 2 Change in deflection versus time.

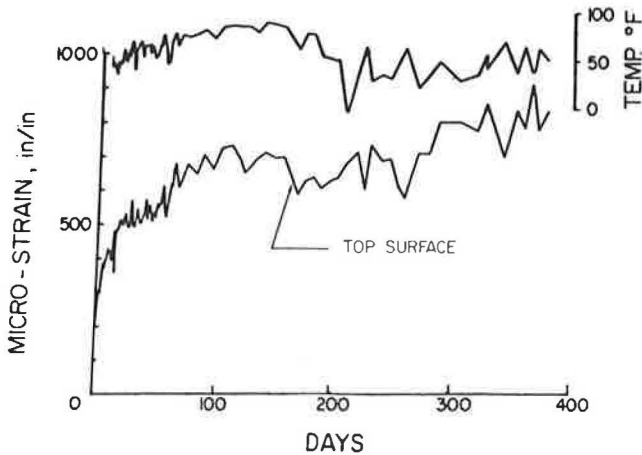


FIGURE 3 Change in strain of concrete surface versus time.

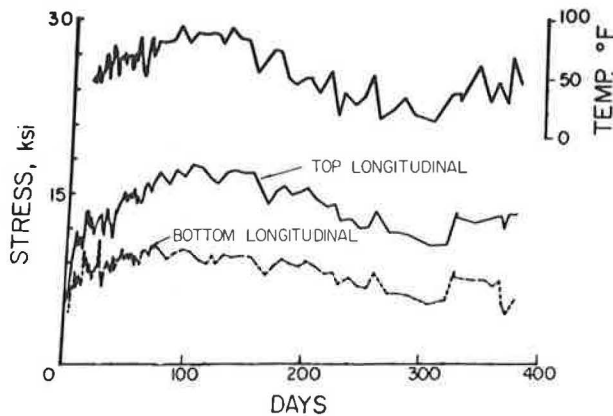


FIGURE 4 Change in stress of reinforcing bars versus time.

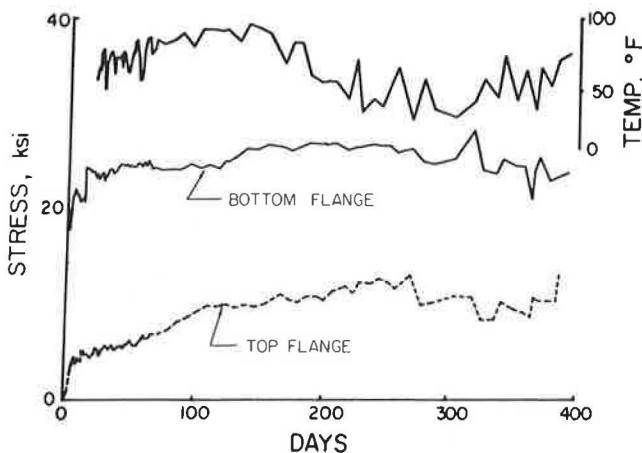


FIGURE 5 Change in beam flange stress versus time.

shows the variations in stresses on the top and bottom flanges of one beam. Changes in the neutral axis location in the steel beam versus time are shown in Figure 6.

Discussion of Results

An examination of Figures 2-5 shows that all the

strain values, and consequently all the stress values, along with vertical deflections are sensitive to changes in temperature. Changes in strain on the girder and changes in strain on the rebars appear to be least affected by temperature changes; vertical deflection and changes in strain on the concrete surface are the most sensitive. Thus, the variation in temperature must be considered in any interpretation of results.

Figure 2 shows the variation in vertical downward deflection at midspan. It is evident that variations in temperature caused variations in the vertical downward deflection. Figure 2 also shows that a growth in camber occurred in the bridge unit until the additional sustained dead load was placed on the bridge (approximately the 21st day). The vertical downward deflection then increased substantially as the temperature increased. The maximum change in vertical downward deflection of 0.87 in. was reached near the 150th day (first week of September 1982). Thereafter, the vertical downward deflection decreased (increased upward) as the temperature decreased.

A study of Figure 3 shows that the change in strain at the concrete surface closely followed the change in temperature. As the air temperature increased, the change in strain increased and as the air temperature decreased, the change in strain also decreased. Increases in strain were found to be most rapid during the first few days of observation. The change in concrete strain steadily increased during the warm months (May-August 1982) and fluctuated during the late fall and winter months (December 1982 to February 1983) when the temperature also fluctuated significantly. As Figure 3 indicates, concrete strain values had not yet asymptotically approached a maximum and changes were still occurring at the end of the observation period.

Figure 4 shows the behavior of the strain in the top and the bottom longitudinal reinforcing bars at the centerline of the bridge unit. A comparison of Figures 3 and 4 shows that the change in strain of the concrete surface is similar to the change in strain of the reinforcing steel. The strain (hence the stress) increased and decreased as the temperature increased and decreased. As was the case with the concrete surface strains, the greatest increases in reinforcing steel strains occurred during the first few days of observation. However, the strains on the reinforcing bars did not increase as rapidly as the strains at the concrete surface, nor did they change as rapidly as the temperature increased.

Figure 5 shows the stresses on the bottom and top flanges of one beam. Stresses were calculated from strain data assuming a modulus of elasticity of 29 million psi. A comparison of Figures 3 and 5 shows that temperature has less effect on the change in stress of the beams than it does on the concrete surface strains. A sharp increase in stress in the bottom flange occurred when the bridge unit was turned over and when the additional dead load was applied. Increases also occurred in the top flange but to a lesser degree. After the application of the dead load, the stresses in the top and bottom flanges gradually increased until about the 265th day (January 1, 1983). Subsequently, the stresses in both the top and bottom flanges decreased somewhat. Initially, the stresses in the top flange increased at a slightly faster rate than those of the bottom flange, indicating that the location of the neutral axis was moving away from the top flange toward the bottom flange.

Figure 6 shows the neutral axis location from the top of the beam versus time. The location of the neutral axis was determined using the beam flange strain readings and assuming a linear strain distri-

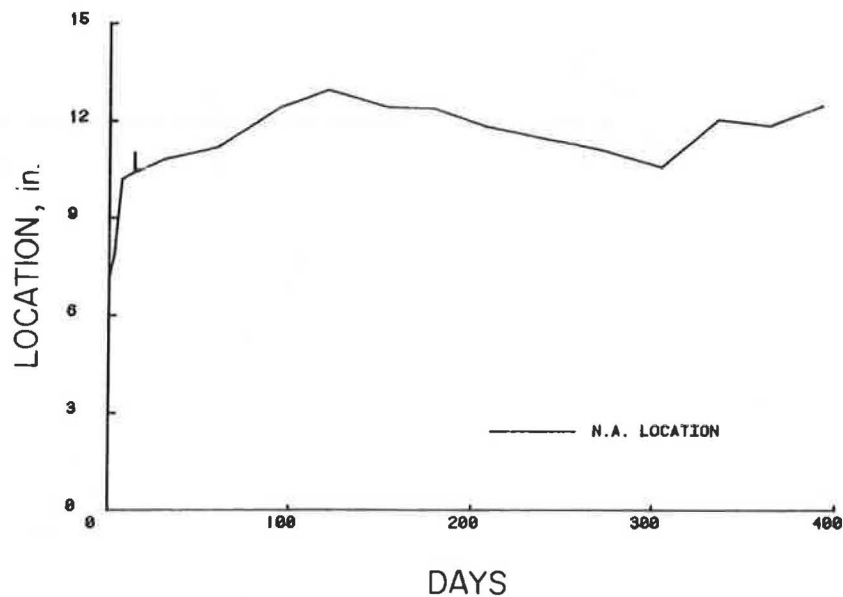


FIGURE 6 Neutral axis location from top of concrete versus time.

bution within the steel section. The location is plotted in Figure 6, with respect to the top concrete surface. Initially the neutral axis moved rapidly toward the bottom flange. The rate of increase subsequently decreased and at the end of the reporting period the neutral axis location appeared to be stationary. The location remained within the steel section throughout the observation period.

Figure 7 shows a comparison of the measured midspan deflection of the bridge unit with the deflections predicted using a method proposed by Branson (1). The predicted total deflection due to creep and shrinkage plus temperature is shown as well as the individual computed deflections due to differential shrinkage and creep and those due to temperature effects. Only the total measured deflection is

shown because no distinction could be made between deflections caused by creep and shrinkage and deflections caused by temperature effects.

It can be seen from Figure 7 that the measured and the predicted curves have basically the same shape and are in good agreement considering the complexity of predicting creep, shrinkage, and temperature effects. The measured effects are less than the computed effects even though prestressing effects were not considered in the calculations.

REPEATED LOADING TEST

Test Description

At the end of 1 year of sustained loading observation the bridge unit was moved into the laboratory to be subjected to 500,000 cycles of repeated loading to simulate traffic load. The unit was supported on two W10x49 steel beams. The same elastomeric bearing pads used earlier were placed between the support beams and the unit beams.

To simulate traffic load a two-point loading system was used with the load points 14 ft apart and equidistant from the midspan. Spreader beams were used to distribute the loads transversely to the unit at four load points that were directly above the steel beams.

Additional Instrumentation

To determine possible deterioration of composite action during the repeated loading, additional instrumentation was installed. Three displacement transducers were mounted on the soffit of the concrete slab next to the top flanges of one steel beam. The plunger of each transducer rested against an angle spot welded to the top flange of the steel beam. These transducers were placed at sixth points starting from one end and were used to measure differential displacement (slip) between the concrete slab and the steel beam.

In addition to these displacement transducers, piano wires were anchored to the soffit of the concrete slab on the exterior side of the same beam at

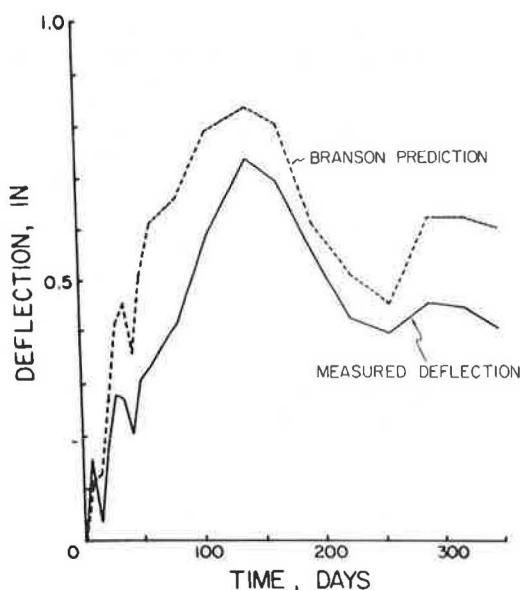


FIGURE 7 Comparison of measured midspan deflection due to shrinkage, creep, and temperature effects with that predicted by Branson method.

48-in. intervals. These piano wires crossed the longitudinal seam between the top flange of the beam and the soffit of the concrete slab. Lines were then marked across the soffit of the concrete slab and the top flange to coincide with the piano wire. Through this means, any residual slip that occurred during the dynamic loading could be detected and measured with a hand-held measuring microscope.

Procedure

The repeated loading was applied with a 55,000-lb closed-loop hydraulic testing system at a frequency of 0.5 Hz for 500,000 cycles. The repeated loading varied between 7,000 and 62,000 lb (including the weight of the loading and the spreader beams) corresponding to a minimum midspan moment of 70,400 ft-lb and a maximum midspan moment of 622,300 ft-lb. The repeated loading was applied 8 hr per day.

The first cycle of loading was applied statically at 5,000-lb intervals. After each load increment, strain readings on the longitudinal reinforcing steel, the steel beams, and the surface of the concrete were taken. In addition, readings of displacement transducers were taken to determine the vertical downward displacement as well as the slip between the steel beam and the concrete. The repeated loading was interrupted after the 1st cycle and after every 50,000th cycle thereafter to determine any deterioration in stiffness of the bridge unit that may have occurred during the repeated loading. In addition, the piano wires and diaphragm welds were visually inspected for any sign of distress at 50,000-cycle intervals.

Results and Discussion

Selected results of the repeated load tests are shown in Figures 8 and 9. Figure 8 shows the change in midspan deflection as measured at each 50,000th static cycle versus number of cycles. Figure 9 shows a change in stress in the top reinforcing steel and bottom beam flange at maximum static load versus number of cycles. From these plots, a very

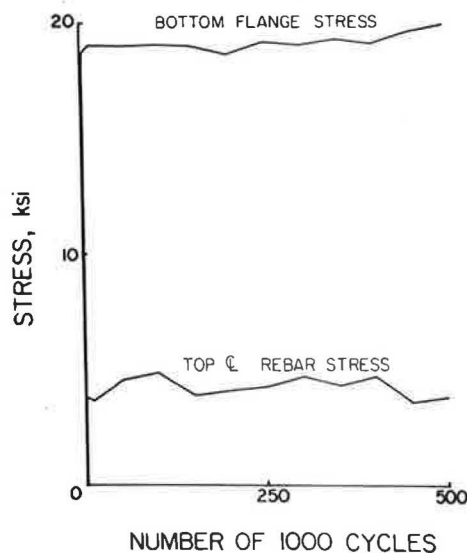


FIGURE 9 Flange and rebar stresses versus number of cycles.

small amount of deterioration is detected between the 450,000th and 500,000th cycles.

OPERATING RATING LOADING TEST

At completion of the repeated loading test the unit was subjected to loading equivalent to AASHTO HS 30 rating. This loading is 50 percent greater than the design loading (HS 20) and the magnitude was determined based on a tension flange stress of 0.75 of the yield stress of the material (e.g., 37.5 ksi). The same test setup used for the repeated loading test was used. Figure 10 shows the resulting measured vertical load versus centerline vertical deflection relationship. The theoretical deflection based on a modular ratio value, n , of 7.5 is also shown. It is noted that the experimental load-deflection curve is linear and there was no permanent set.

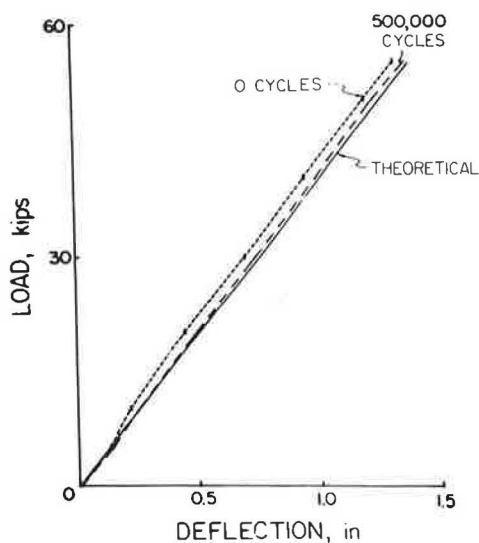


FIGURE 8 Load versus vertical deflection, repeated loading test.

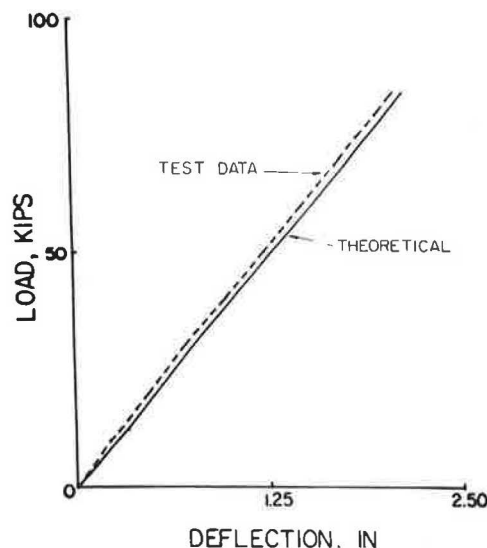


FIGURE 10 Load versus deflection, operating rating loading test.

CONCLUSIONS

Sustained Loading

The strain values in the concrete slab, the reinforcing bars, and the steel beam as well as the downward deflection of the unit were influenced by changes in temperature. The strain values in the concrete slab did not asymptotically approach a maximum. On the other hand, strains (hence stress) in the steel beam and the reinforcing bars appeared to have leveled off to constant values. The neutral axis location also appeared to be stationary and the vertical deflection appeared to have stabilized. As expected, the major portion of the creep deformation occurred during the first 90 days of testing. At the end of 1 year of sustained loading, it appears that although creep deformation continues, the rate of increase of creep is minimal.

Repeated Loading

The results show that the load-displacement curve changed little between the first and last cycles of testing. Insignificant slip was recorded between the concrete slab and steel beams during the static cycles. It is concluded that the strength and stiffness characteristics of the bridge unit are not detrimentally affected by repeated loading for which the unit was designed.

Operating Loading

Results of the operating loading test show that the test unit is able to sustain this loading without permanent set. The behavior of the unit was elastic during the test.

The unit has been placed outside for 2 additional years of sustained loading observation. At the end of that period, the unit will be loaded to flexural failure and the slab will be studied for transverse bending strength.

ACKNOWLEDGMENTS

This paper was developed from a research project sponsored by the Oklahoma Department of Transportation. The authors wish to thank Tim Borg, Jim Schmidt, and Dwight Hixon of the ODOT Research and Development Division and Veldo Goins of the ODOT Bridge Division for their helpful suggestions and assistance.

REFERENCE

1. D.E. Branson. Deformation of Concrete Structures. McGraw Hill, New York, 1977.

Publication of this paper sponsored by Committee on Steel Bridges.

Temperature Problem in a Prestressed Box-Girder Bridge

P. C. HOFFMAN, R. M. McCLURE, and H. H. WEST

ABSTRACT

The temperature effects on an experimental, prestressed concrete, segmental, box-girder bridge were evaluated. Field measurements, including vertical deflections and thermocouple readings for temperature, were collected over a 1-year period under various weather conditions. Critical ambient temperature conditions were gleaned from field observations. With the field observations (surface temperatures) as input, three-, two-, and one-dimensional heat-flow analyses were conducted. The three-dimensional heat-flow analysis was a statistical comparison of temperature readings at various longitudinal locations in the experimental bridge. The two-dimensional analysis involved the collection of hourly temperature readings from 24 thermocouples at the mid-span of the test bridge for 18 diurnal cycles within a 1-year period. In addition an alternating-direction implicit finite difference analysis was performed, and the

results compared well with the field observations. Finally, a one-dimensional approach was formulated with the application of an initial-value method. This was found to be in agreement with the alternating-direction implicit finite difference model and the collected field data.

In the past, the thermal environment was ignored in the design process except for accounting for the longitudinal movement that could occur and the influence such movement could have on joint integrity. However, relatively recent incidents of bridge distress, including the Newmarket Viaduct in New Zealand (1), the fourth Danube Bridge in Vienna, and the development of a crack 56 ft (17 m) long and 0.20-0.24 in. (5-6 mm) wide in the web of the Jagst Bridge in Untergreisheim (2) have prodded the profession to address the complete structural response to the thermal environment.

A field study was conducted to determine the complexity and magnitude of the temperature problem