PRE-STRESSED STEEL BRIDGE UNIT

-RESEARCH, DESIGN AND CONSTRUCTION-

by

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Presented at

The 1983 Northwest Bridge Engineers Seminar
Olympia, Washington
October 6, 1983

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Introduction

A study is being undertaken 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 pre-cast, prestressed steel beam composite bridge unit. Two, three or four of the units can be used for small stream crossings. The test unit is 55 ft. 0 in. long and 6 ft. 9½ in. wide consisting of two W21x50 steel beams and a 7½ in. thick reinforced concrete slab and weighs 39,000 lbs. 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 three and one-half years beginning in April 1982 with scheduled completion in December 1985. The project consists of six phases: one year of observation under sustained load; three months of repeated loading (500,000 cycles); operating rating loading; two and one-half years of observation under sustained dead load; static flexural test to failure; and transverse slab strength tests. As of this date (October, 1983), the first three phases of the research project have been completed. Results are summarized here.

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 steel form hung from the beams as shown in Figure 1. Once the concrete was poured and finished, an additional steel weight was placed on the beams. After the concrete had cured the weight and the form 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 upon delivery. The unit was then placed on an existing concrete slab. The location is outside and subject to weather conditions. Elastomeric bearing pads were placed between the support beams and the unit beams. On April 22, 1982 concrete blocks weighing 33±0.1 pounds each were placed on the unit to simulate dead load from an asphalt overlay.

Specimen Details

The full scale unit is 55'-0" long and 6'-9" wide and consists of two steel beams (W21x50) and a 7½ in. thick reinforced concrete slab. The concrete slab is reinforced with #4 bars both longitudinally and transversely. The longitudinal bars are spaced at 9 in. at the top and 18 in. at the bottom of the slab. The closer spacing at the top is intended primarily to minimize creep strains in the concrete slab. Both top and bottom transverse reinforcement was placed at variable intervals symmetrically about the mid-span. Starting from the mid-span and going toward each end, the spacings were 5½ in., 11 in. and 16½ in. for each one-sixth of the span length. The intention of this variable spacing is to study the influence of transverse reinforcement on the transverse strength of the concrete slab at a later stage.

An air-entrained concrete with a design strength of 5000 psi 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 about 1 inch. The design slump was between 1 and 3 inches. The measured average 28 day strength was 5300 psi.

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.

Shear connectors were welded to the W21x50 beams before the beams were placed in the forms. Shear connectors consisted of standard steel studs and short sections of hot-rolled channel. Two studs were welded to the top flange of each beam with variable spacing. The spacing varies from 12 in. at each end to 30 in. at the mid-span as shown in Figure 2. C7x9.8 by 6 in. long channels were welded to the top flange of each beam at 9'-0" intervals. These channels were used as spacers 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 3x3x¼ angles. Center-to-center spacing of the beams was 3 ft. 9½ in. Details are shown in Figure 3.

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 of single length except for those that were strain gaged. The longitudinal bars with strain gages 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 well oiled steel forms in the upside-down position. The form work, 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 weights (approximately 7000 pounds) were placed on the beams at midspan. Shims were used at midspan to limit vertical deflection to 3½ in.

The concrete slab, together with control cylinders, were moist-cured in the formwork for 7 days. The exposed surface of the concrete was covered with web burlap and polythene sheets during the seven-day curing period. On the eighth day after casting, 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.

Instrumentation

With the exception of the shear connectors, all components of the bridge unit, reinforcing steel, steel beams and the concrete are fully instrumented. Ten longitudinal reinforcing bars, five top and five bottom were strain gaged longitudinally at the midspan with an electrical resistance strain gage having a gage length of ¼ in. Similar gages were mounted on six transverse reinforcing bars, three top and three bottom. Three gages were placed on the top bars and two on the bottom bars, located as shown in Figure 4. The instrumented transverse bars were located in different spacing regions as discussed previously.

Prior to placing the steel beams in the concrete form, eight ½ in. gage length strain gages were installed on the beam flanges near midspan as shown in Figure 4. The gaged cross-section was 8 in. from midspan to avoid effects induced by a spacer channel located at midspan.

After removal from the forms but prior to turning, two 10 in. gage electrical extensometers were mounted on the top and the bottom surfaces of the concrete and along the longitudinal axis of the unit at midspan.

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

Once the bridge unit was set in place at the Laboratory, two DCDT 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 was attached to the interior portion of each bottom flange. A thermometer was also embedded in the concrete slab.

Two switch and balance units and two strain indicator units permanently wired to the strain gages were used to make strain measurements. A d.c. voltage supply and standard voltmeter was used to monitor the deflection transducers. The entire set of measuring instruments was stored in a refrigerator (set at 70^{0}F) in a temporary building adjacent to the test unit.

Initial Sustained Loading Test Results

The results of the creep data taken for the first year are presented graphically in Figures 5 to 8. Stress data was obtained from the strain data using Hooke's Law (assuming perfectly elastic material) and an assumed modulus of elasticity of 29,000,000 psi for steel. Variations in the concrete strain at midspan along the longitudinal axis of the bridge unit together with temperature changes are shown in Figure 5. Figure 6 depicts the variations in stresses at the top and bottom flanges of the west girder. The average midspan deflection of the unit is shown in Figure 7. Stress variations in both the top and bottom longitudinal reinforcement are illustrated in Figure 8.

An examination of the results shown in Figure 5 to 8 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 change while 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.

A study of Figure 5 shows that the change in strain at the concrete surface closely follows 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 strain steadily increased during the warm months (May to August 1982) and fluctuated during the late fall and winter months (December 1982 to February 1983) when the temperature also fluctuated significantly. As the plot indicates, strain values have not yet asymptotically approached a maximum and changes are still occurring. It is noted that the strain is plotted for the top surface at the centerline of the bridge unit.

Figure 6 shows the stresses of the bottom and top flanges of the west beam. Stresses were calculated from strain data assuming a modulus of elasticity of 29,000 ksi. A comparison of Figure 5 to Figure 6 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). Since that time, the stresses in both the top and bottom flanges have somewhat decreased. Initially, the stresses in the top flange increased at a slightly faster rate than those of the bottom flange. This indicated that the location of the neutral axis was moving away from the top flange toward the bottom flange. The location of the neutral axis was determined using the beam flange strain readings and assuming a linear strain distribution within the section.

Figure 7 shows the variation in vertical downward deflection at mid-span. It is evident that the variations in temperature cause variation in the vertical downward deflection. The figure shows that camber (positive deflection) existed in the bridge unit until the sustained dead load was placed on the bridge (approximately the 20th day). The vertical downward deflection then increased substantially as the temperature increased. The maximum change in vertical downward deflection, 0.87 inches, was reached near the 150th day (first week of September, 1982). Thereafter, the vertical downward deflection decreased (increased upward) as the temperature decreased.

Figure 8 shows the behavior of the strain on the reinforcing bars. A comparison of Figures 5 and 8 shows that the change in strain of the concrete surface is similar to that of the change in strain of the reinforcing steel. The strain (hence the stress) increased and decreased as the temperature increased and decreased. As with the concrete surface strains, the greatest increases in reinforcing steel strains also 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 as the temperature increased.

Repeated Loading Test

At the end of one year of creep 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 the two W10x49 steel beams and 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 being 14 feet apart and equidistant from the mid-span. Spreader beams were used to distribute the loads transversely to the unit at four load points which were directly above the steel beams.

To determine possible deterioration of composite action during the repeated loading, additional instrumentation was installed. Three displacement transducers were mounted on the suffit of the concrete slab next to the top flanges of one steel beam. The plunger of each transducer was placed 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 suffit of the concrete slab on the exterior side of the same beam at 48 in. intervals. These piano wires crossed the longitudinal seam between the top flange of the beam and the suffit of the concrete slab. Lines were then marked across the suffit of the concrete slab and the top flange to coincide with the piano wire. Through this means, any residual slip that occurs during the dynamic loading was detected and measured with a hand-held measuring microscope.

The repeated loading was applied with a 55,000 lbs closed-loop hydraulic testing system at a frequency of 0.5 Hz for 500,000 cycles. The repeated loading was varied between 7,000 and 62,000 lbs (including the weight of the loading and the spreader beams) corresponding to a minimum mid-span moment of 70,400 ft-lbs and a maximum mid-span moment of 636,000 ft-lbs, respectively.

The first cycle of loading was applied statically at 5,000 lbs intervals. After each load increment, strain readings on the longitudinal reinforcing steel, steel beams, and on the surface of the concrete were measured. In addition, readings of displacement transducers were taken to determine the vertical downward displacement as well as slip between the steel beam and the concrete. The repeated loading was interrupted after every 50,000th cycle thereafter and a static-cycle test conducted 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 the 50,000 cycle intervals.

Selected results of the repeated load tests are shown in Figures 9 and 10. Figure 9 shows the change in midspan deflection as measured at each 50,000th static cycle versus number of cycles. Figure 10 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 small amount of deterioration is detected between the 450,000th and 500,000th cycles.

Operating Rating Loading Test

Upon completion of the repeated loading test the unit was subjected to loading equivalent to AASHTO HS 30 rating. This loading is 50% 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 as used for the repeated loading test was used. Figure 11 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 permanent set was non-existent.

Conclusions

Based on the sustained loading test, strain values in the concrete slab, the reinforcing bars as well as the downward deflection of the unit are significantly influenced by changes in temperature. The strains on the beam flanges did not change with temperature change. The strain values in the concrete slab had not asymptotically approached a maximum at the conclusion of the test. On the other hand, strains (hence stress) in the steel beam and the reinforcing bars appear to have leveled off to constant values. As expected, the major portion of the creep deformation occurred during the first 90 days of testing. At the end of one year of sustained loading it appears that although creep deformation continues to occur, the rate of increase of creep is minimal.

From the repeated loading test, there is not significant difference between the slope of the static load-displacement curve obtained at the first cycle of loading and that obtained at the end of the 500,000th cycle. After 500,000 load repetitions, it appears that the strength and stiffness characteristics of the bridge unit are not affected by repeated 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 will now be placed outside for two 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 Messrs. Tim Borg, Jim Schmidt and Dwight Hixon of the ODOT Research and Development Division and Mr. Veldo Goins of the ODOT Bridge Division for their helpful suggestions and assistance. The assistance of Ms. Lori Creech of the Fears Structural Engineering Laboratory in the research reports is acknowledged.

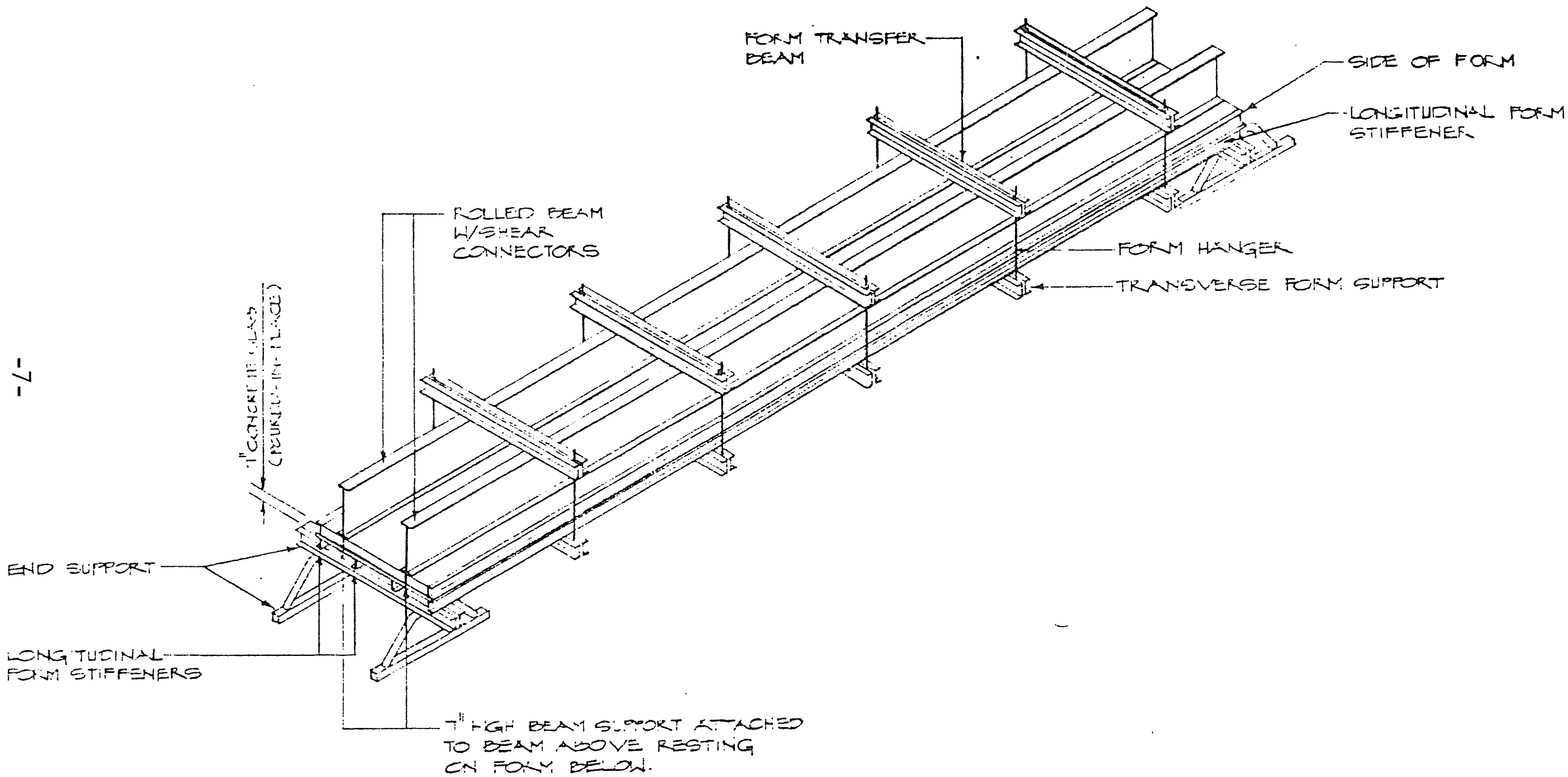

Figure 1. Method of Fabrication

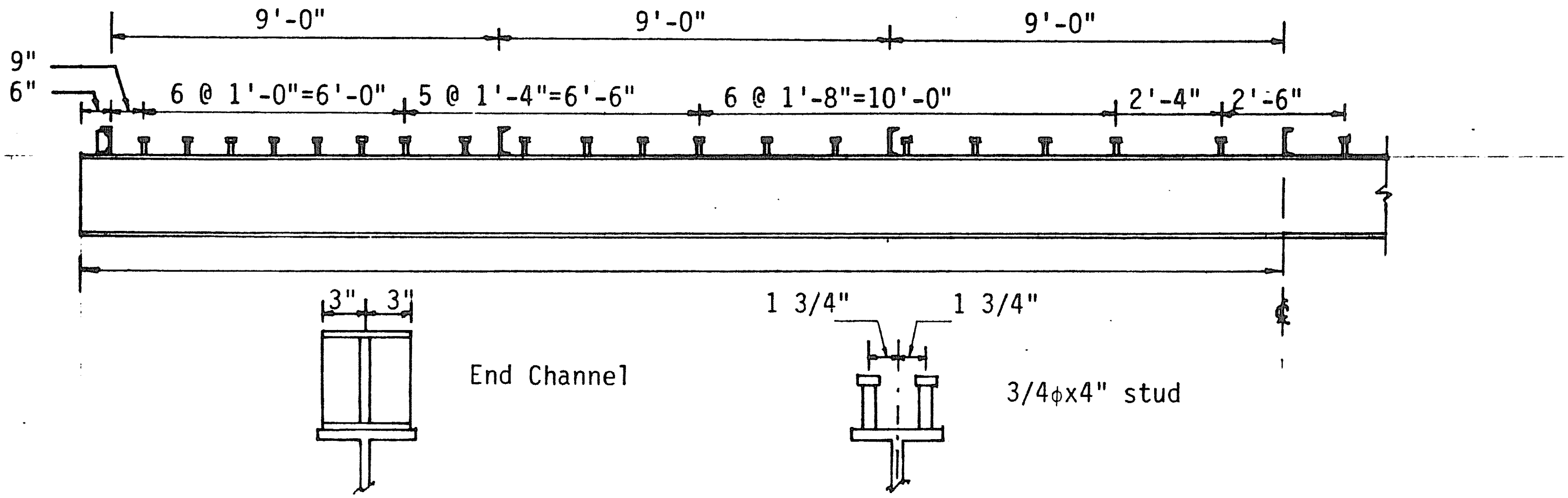

Figure 2. Details of Shear Connectors

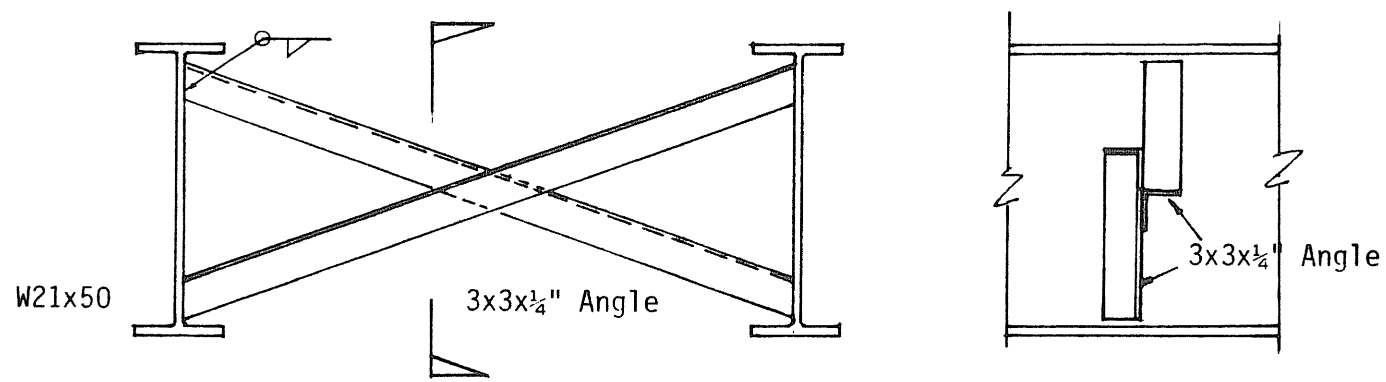

Figure 3. Detail of Diaphragm

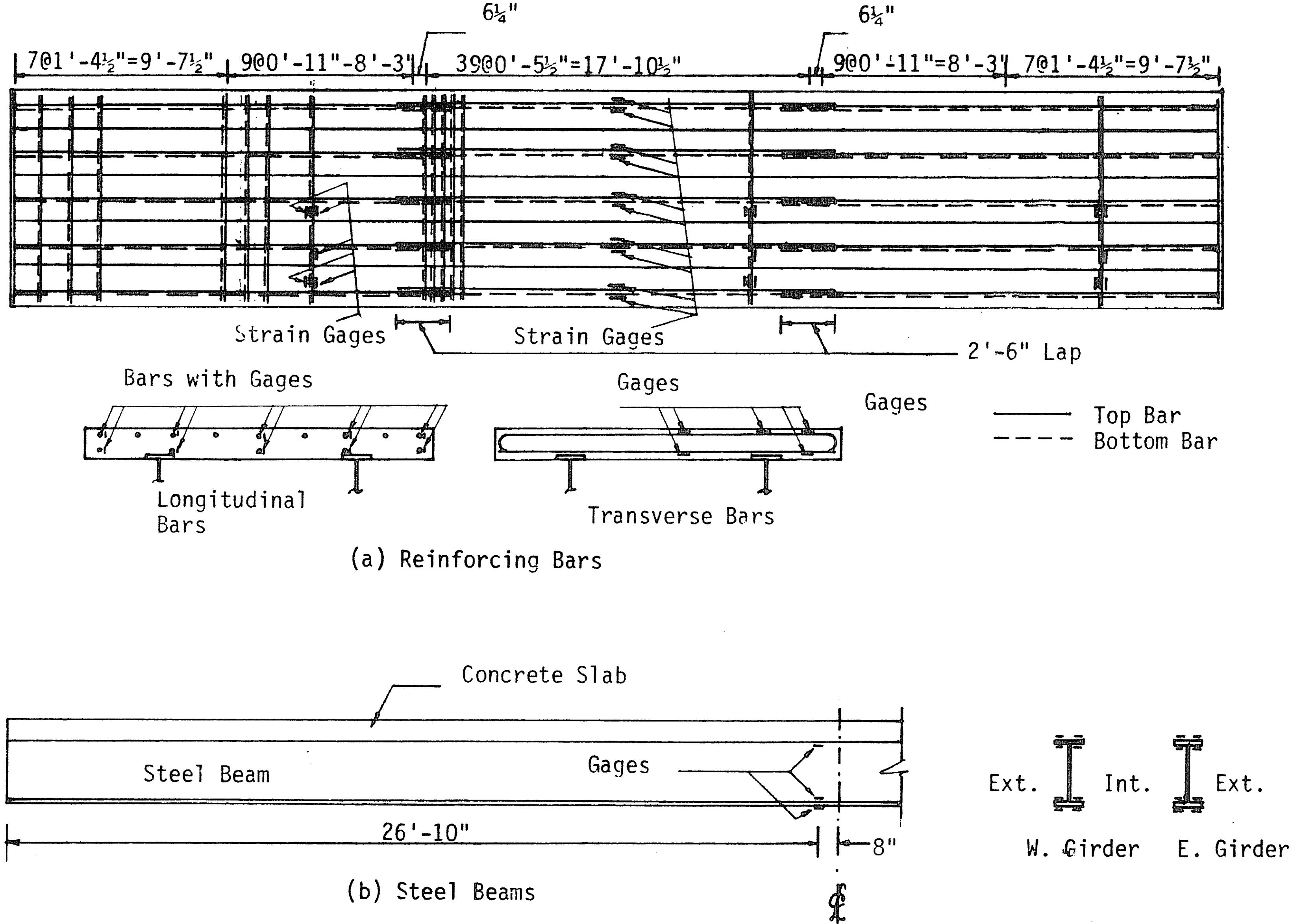

Figure 4. Location of Strain Gages

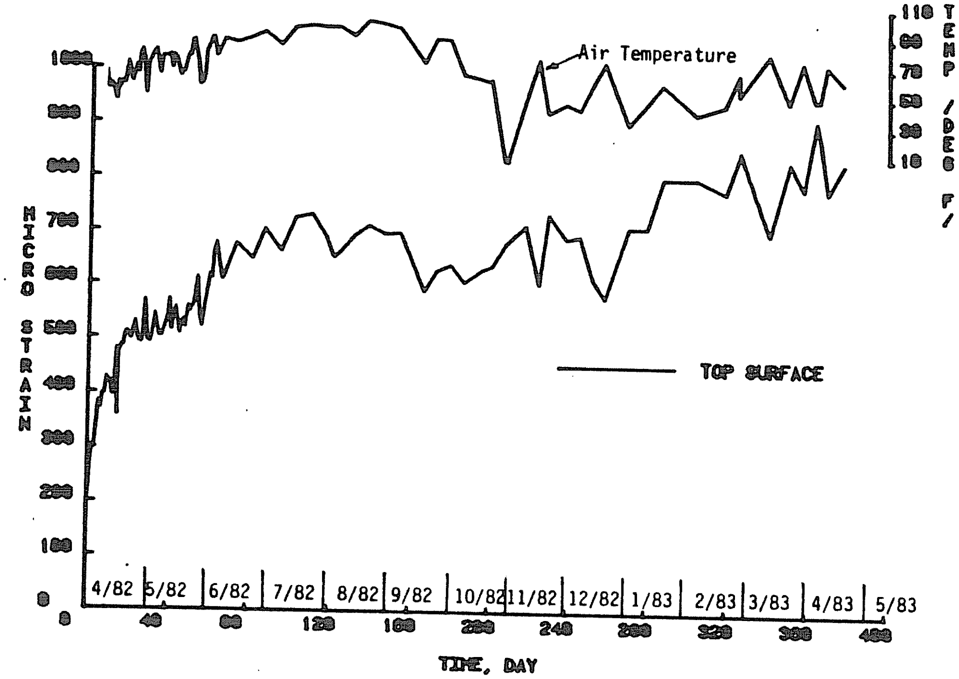

Figure 5. Change in Strain of Concrete Surface vs. Time

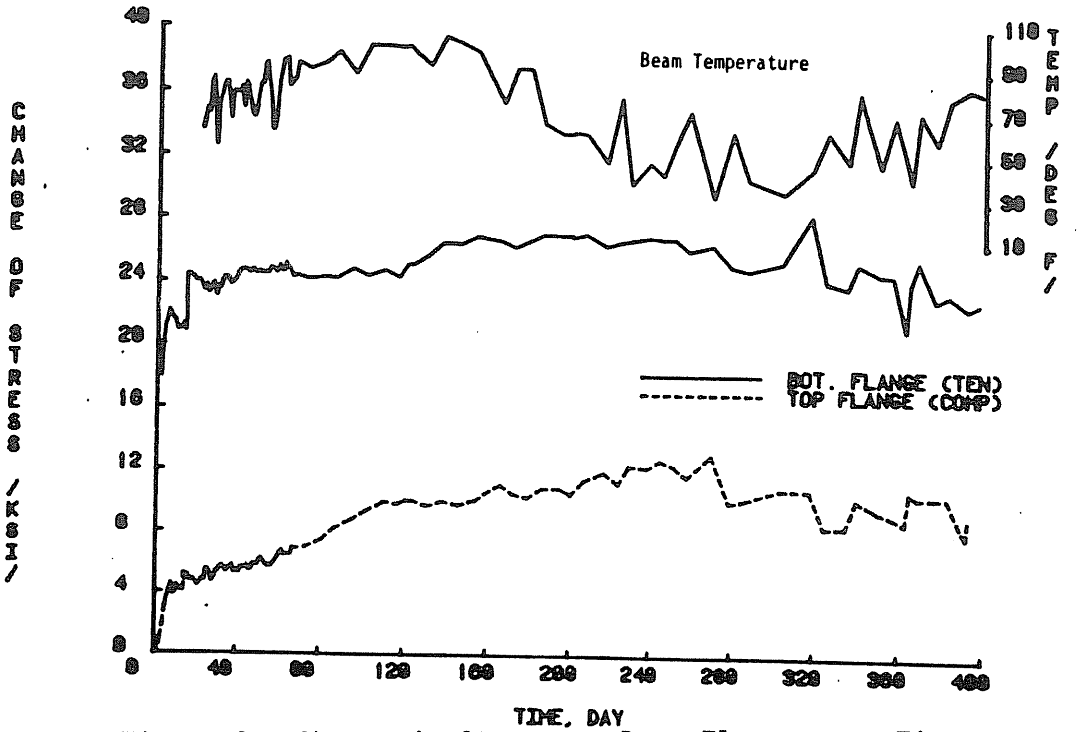

Figure 6. Change in Stress on Beam Flanges vs. Time

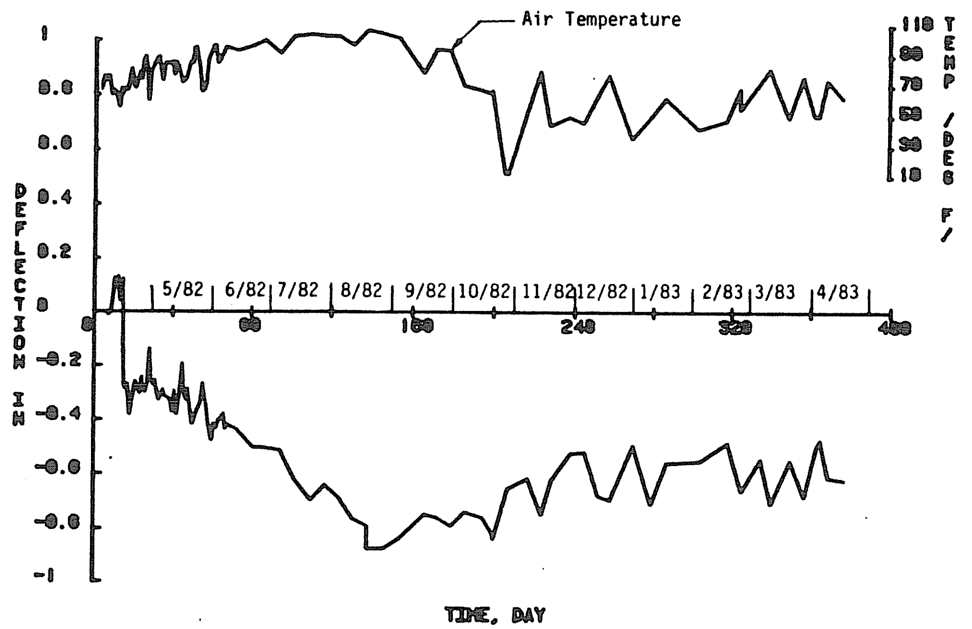

Figure 7. Time vs. Change in Deflection

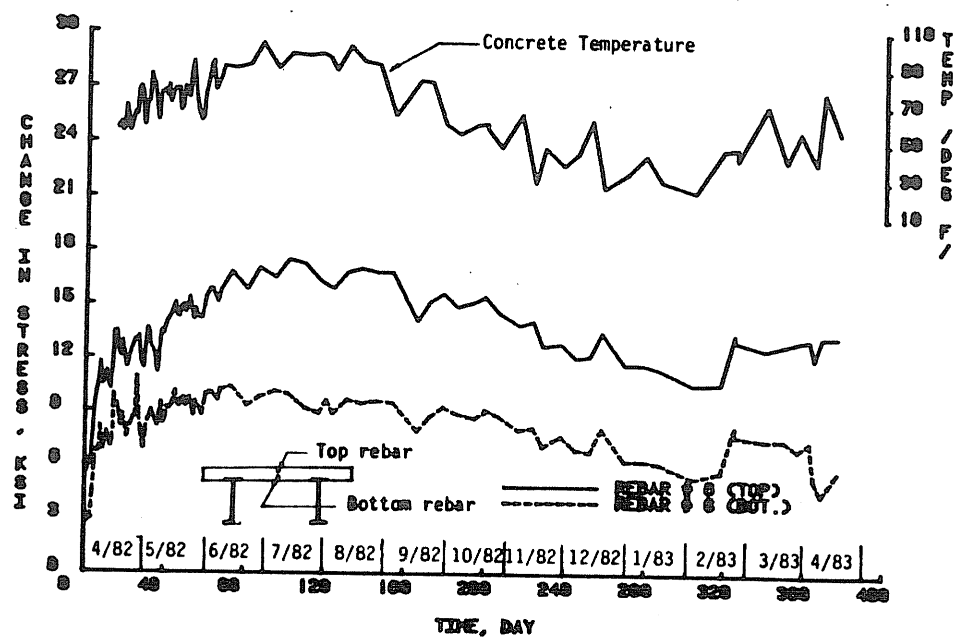

Figure 8. Change in Stress of Rebars vs. Time

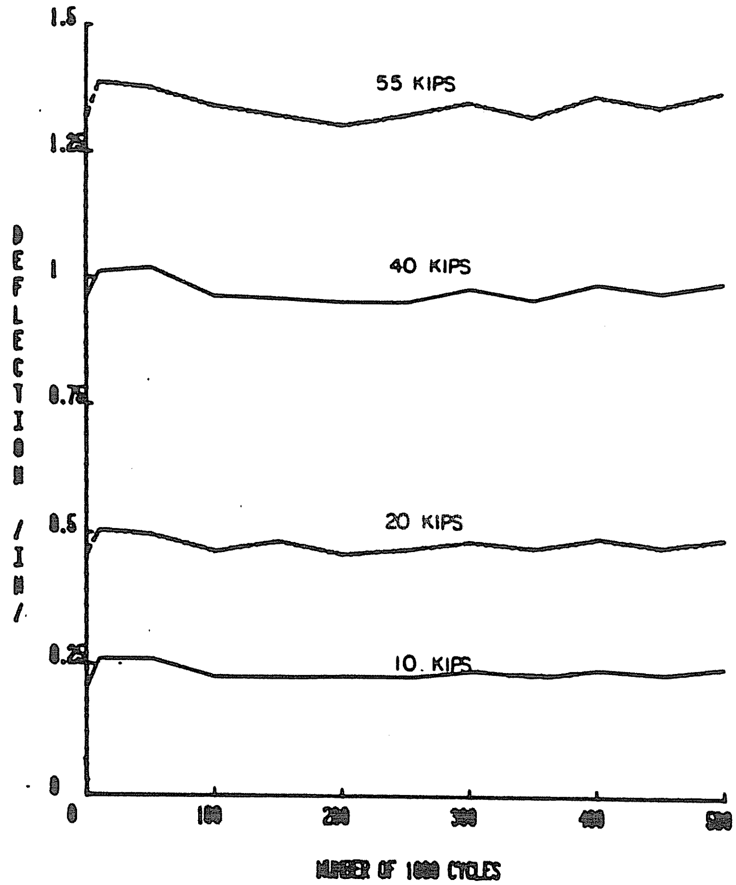

Figure 9. Vertical Deflection vs. Number of Cycles

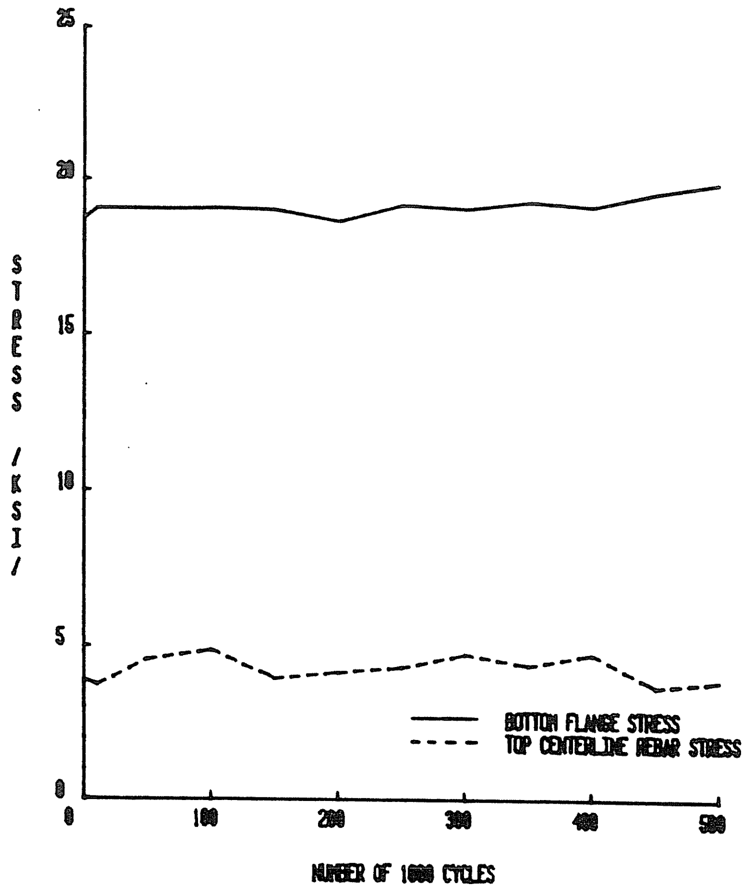

Figure 10. Stress vs. Number of Cycles

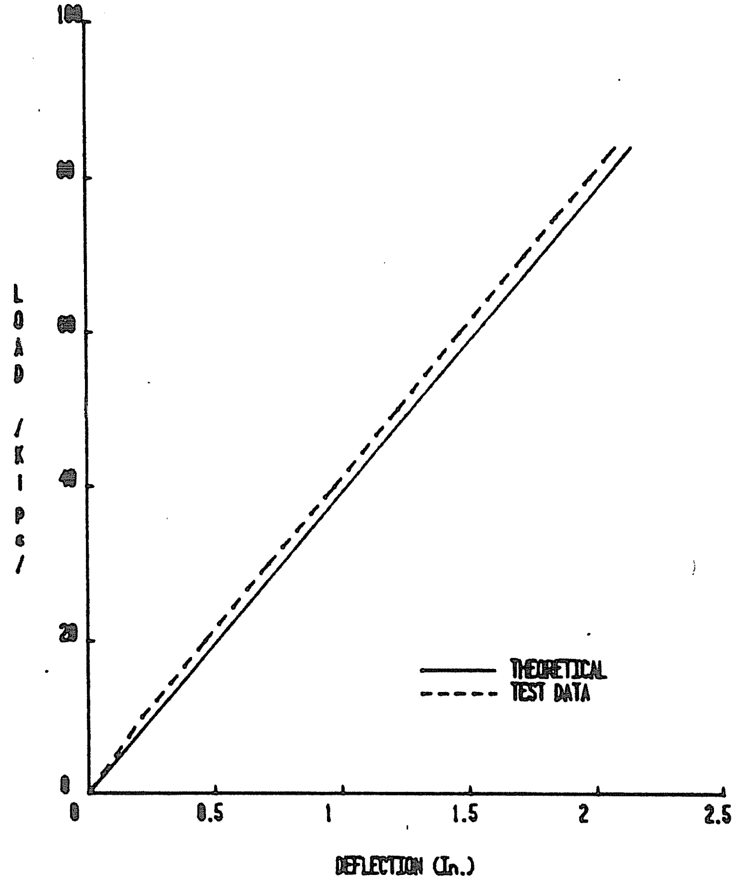

Figure 11. Load vs. Deflection from Operating Rating Loading Test