Field Study of a Pedestrian Bridge of Reinforced Plastic

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A discussion of the behavior of the superstructure of a pedestrian bridge fabricated with glass-reinforced plastic under a field load test is presented. Experimental measurements of elastic vertical deflections were 1.8 times greater than those predicted by means of a finite element solution. A live load of 3.5 times the dead load of the superstructure and polymer concrete deck was used for the elastic load test. Elastic strains were uniform among the different elements of the superstructure and computed stresses did not exceed 10,000 lbf/in.^{2} at full live load. A residual deflection in the superstructure of 0.10 in. on removal of the live load was concentrated in the supports. Creep deflection and strain measurements recorded over 61 days indicated that negligible creep occurred under a load of 2.6 times the dead load. Air temperature variations produced pronounced changes in deflection and strain readings, but were reversible. The overall structural behavior of the bridge and resistance to handling abuse exceeded expectations.

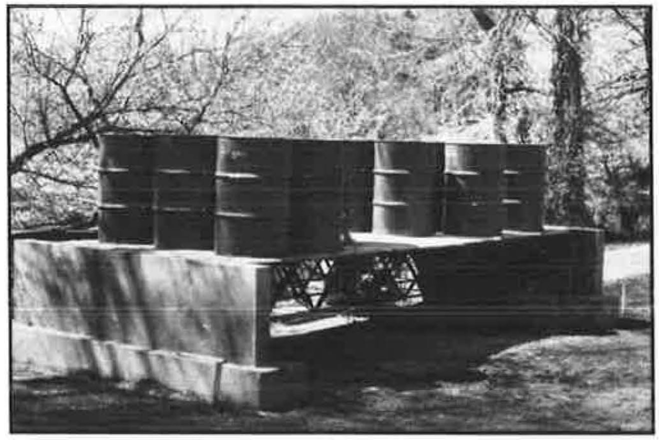

FIGURE 1 Pedestrian bridge in place over the discharge channel of an irrigation pond at Pen Park.

The design, development, fabrication, and laboratory testing of the components for the pedestrian bridge described in this paper have been discussed in previous publications *(1–5)*. The final configuration of the erected bridge was 16 ft long by 7 ft wide and 18 in. deep. The superstructure consisted of three identical trussed girders placed side by side and attached transversely by pultruded glass-reinforced plastic (GRP) plates bonded to the top flange of each girder. A detailed description of the configuration was presented by McCormick *(1)*. The foundation structure of reinforced concrete consisted of footings, backwalls, and precast seats. The seats were formed to match the triangular shape of the bearing surfaces of each of the bridge girders.

A multiple-layer polymer concrete (PC) overlay was applied to the deck of the bridge to provide a wearing course with a slight crown for drainage. The average depth of the wearing course was approximately ½ in. and weighed approximately 1,000 lb. The total weight of the superstructure was 2,300 lb.

The site chosen for the field study was in Pen Park, one of the municipal recreational areas of Charlottesville, Virginia. The bridge was located across the overflow channel of the primary irrigation pond for a golf course in the park as shown in Figure 1. The intended use of the bridge was for pedestrians and golfers using electric carts weighing approximately 1,000 lb fully loaded. Two months after erecting the bridge, a heavy rainfall in the park caused severe erosion of the region adjacent to the bridge foundation and required removal of the structure. Subsequently, the bridge was moved to the structural testing laboratory of the University of Virginia where an extensive cyclic load fatigue investigation was initiated. The results of the fatigue study will be reported at a later date.

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OBJECTIVES

The objectives of the field test were as follows:

1. To measure the elastic behavior of the bridge due to short-term loads.
2. To observe the viscoelastic (creep) behavior of the structure under a constant load applied over several weeks.
3. To assess the effects of weathering and service loads on the structure over a period of years.

Data from both the elastic and viscoelastic tests were obtained and are reported here. Early removal of the bridge from the test site precluded the weathering study.

ERECTION PROCEDURES

The precast concrete bridge seats were positioned on footings and anchored by casting the backwalls against them. Elastomeric (75 durometer neoprene) pads were placed on the bearing surfaces of the seats prior to installing the superstructure. These pads (2 layers, ¼ in. thick) assisted in distributing the bearing pressure uniformly along the contact surface of the pultruded end stiffeners in the girders and also served as shims to adjust the final elevation of the deck surface.

Because of the light weight (1,300 lb) of the bridge before the PC overlay was placed, it was moved manually from a staging area in the park and positioned on the seats without the assistance of mechanical equipment. Approximately 1 hr was required to assemble a crew of 12 workmen, remove the wooden shipping braces, and install the bridge on the seats. Figures 2 and 3 show the sequence of installation.

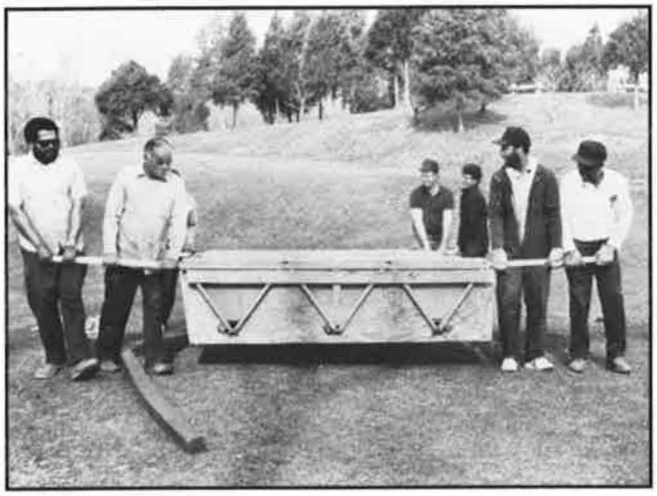

FIGURE 2 Movement of superstructure from staging area to abutments.

FIGURE 3 Lifting superstructure onto the precast concrete seats.

Successive PC layers were applied at intervals of approximately 2 hr to allow sufficient time for the resin binder to cure. Figure 4 shows the hand application of sand at a rate of 2.5 lb/yd^2 to the liquid resin to provide an individual layer thickness of approximately $^1/_8$ in. Excessive amounts of sand were applied to ensure maximum aggregate content in the layer of concrete. The small deck area to be covered did not warrant the use of mechanical equipment for placing the materials. Excess sand was removed after each layer was cured in preparation for the next resin coating. To facilitate water drainage to the sides of the otherwise flat surface, eight layers of sand and resin were applied over various areas of the deck to provide a crown along the center of the deck. Application of the PC in successive

FIGURE 4 Sand aggregate applied to polyester resin to form polymer concrete wearing course.

layers also permitted the buildup of a greater thickness at midspan to compensate for the deflection of the bridge due to its own weight and the weight of the wearing course. The polyester binder used in the concrete was Polylite 92–339 (Reichhold Chemical) and the aggregate was a silica sand from Morie Sand & Gravel. A Morie No. 1 grading was used on layers 1–4 and a Morie No. 2 on layers 5–8. A complete description of the properties of the polymer concrete is given in a report by Sprinkel wherein the resin is designated as Polylite 90–570 *(6)*.

Consideration had been given to placing the PC wearing course on the deck prior to moving the bridge to Pen Park. However, this alternative was rejected because it would increase the weight considerably and because it was questionable if the bond between the concrete and the deck plate would resist the various stresses and deformations caused by handling the bridge. As will be shown later, concern for the integrity of the interfacial bond appeared to be unfounded.

LIVE-LOAD TESTS

Loads were applied by filling 55-gal steel drums with water (≅500 lb total per drum) in the sequence shown in Figure 5. Note that the two center panels of the bridge were not loaded. The progressive manner of loading simulated a load moving from one end of the bridge to the other, which reversed the direction of the shear force in panels 6 and 7 as the load was added to the bridge. The original design with the heavy concrete deck slab precluded a shear reversal in the panels with the application of the design live load, so the diagonal elements in the panels were expected to resist only tensile forces resulting from transverse shears. Consequently, the deck elements ($^1/_2$-in.-thick flange, $^1/_4$-in.-thick coverplates, and $^1/_2$-in.-thick PC) were required to transmit the total live shear force from the loaded portion to the centerline of the bridge. Minor buckling of the plates was observed in several of the panels and considerable buckling occurred in the diagonal elements in panels 5 and 6 as the live load was applied successively to the end panels. The diagonals in panel 5 of one of the outside girders remained slightly buckled throughout the load test period. It is probable that the nonuniform application of the live load or

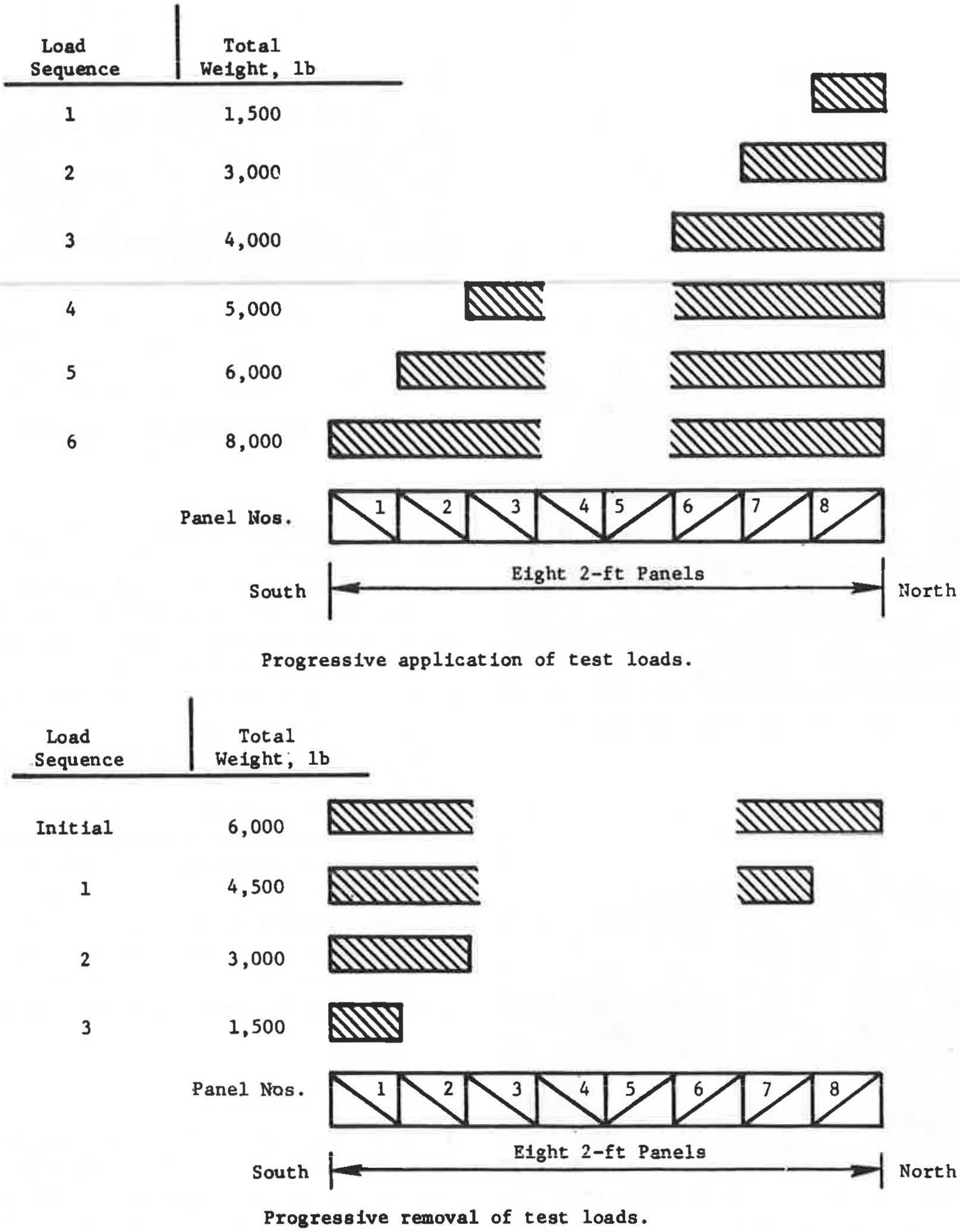

FIGURE 5 Sequence of load application and removal.

slight differences in the end supports induced sufficient torsional distortions into the superstructure to shift the shear forces from one girder to another. Differential distortions of this magnitude were not detectable by the deflection and strain measurements made during the application of the load.

A live load of 8,000 lb on the deck provided an average load of 71.4 lb/ft^2 based on a total surface area of 112 ft^2, or 83.3 lb/ft^2 based on the usable surface of 96 ft^2 between curbs. Because the structure was designed for a live load of 85 lb/ft^2 with the 4-in.-thick portland cement concrete deck in place and acting as the compression flange for the girders, a load of 83.3 lb/ft^2 was considered an overload without the regular concrete deck. The actual contribution of the polymer concrete to the structural behavior of the bridge was unknown, but it was not expected to generate much resistance to compressive flexural forces. An independent determination of a compressive modulus of 1.7×10^6 lbf/in.^{2} for the PC confirmed the expectation that the structural contribution of the wearing surface would be slight, particularly during the long-term creep test of the bridge.

Early evidence of excessive compressive stresses in the flanges was manifested in a slight buckling of the flanges. The maximum amplitude, estimated at 0.10 in., occurred in the end panels in both outside girders. No assessment of buckling was attempted in the interior girder, but it is quite likely that the behavior was similar to that of the exterior girders. There was concern that the displacements of the flanges would grow and possibly result in a catastrophic failure of the bridge as the ambient temperature increased during the summer months and thereby reduced the effective modulus of the flange and deck material. Consequently, two drums of water were removed from panels 3 and 6 (Figure 5) to reduce the live load to 6,000 lb. This load remained undisturbed on the structure throughout the remaining creep test period of 55 days.

After 61 days under live load, all drums were removed, and rebound deflections of the bridge were measured over a period of 46.5 hr. The unloading sequence for live load removal was depicted in Figure 5. In general, the same progressive load removal sequence was used as was followed for the application of the load to observe the effect of a simulated load passing off of the structure. Some buckling of the diagonals in panels 3 and 4 was observed as the load was removed, but not as much buckling was noticed as occurred in the opposite end of the girders when the load was applied.

Elastic Deflections

Lower chord vertical deflections were measured by dial-gauge indicators with least readings of 0.001 in. located at three positions under the center girder. Because of adverse climatic and other conditions at the site, the dial gauges remained in place for only a short time following the application and removal of the live load. The locations of the gauges beneath the girder and the measured deflections of the lower chord are shown graphically in Figure 6.

All deflection measurements included movement of the end supports in the seats due to compression of the elastomeric

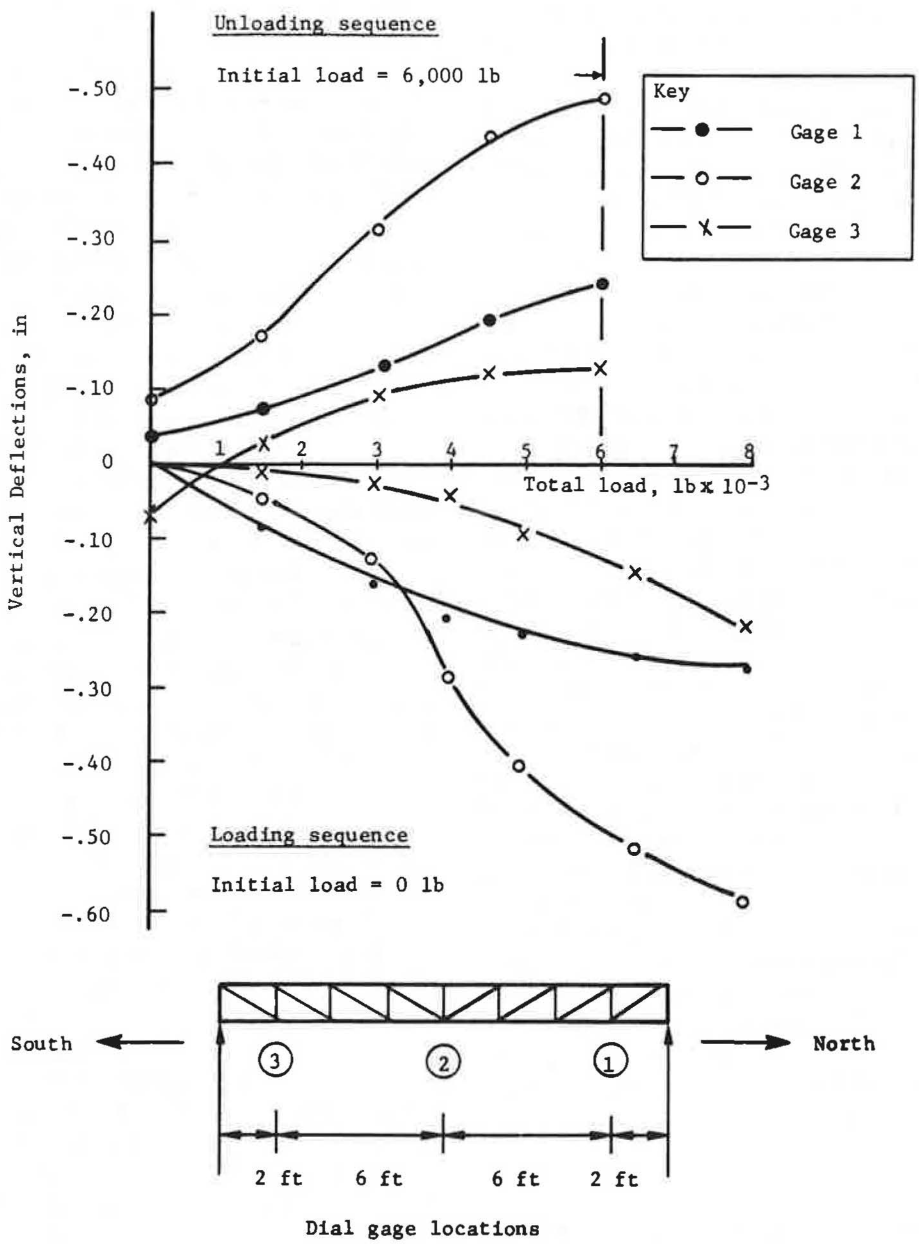

FIGURE 6 Deflections of the lower chord of the center girder as a function of live load.

pads and distortion of the stiffeners in contact with the bearing surfaces. It was not possible to evaluate these components of the measured deflections separately, but they are believed to be a significant part of the values measured by gauges 1 and 3. This supposition is supported by the deck deflection data discussed later. The larger deflection, shown by gauge 1 as compared with that by gauge 3, resulted from the application and removal of the load progressively from the northern end of the bridge to the southern end.

It was expected that the deflection at gauge 3 would reach that of gauge 1 when the full load was applied, because an effort was made to distribute the filled drums symmetrically about midspan. However, a check of the positions of the drums on the deck after they were filled indicated that the loads on the southern half of the bridge were approximately 4 in. closer to the center of the bridge than were loads on the northern half. Thus, it is believed that the off-center loading, plus probable differences in the settlement characteristics of the supports, completely explains the difference of 0.06 in. in the measured deflections.

The difference between the average measured deflections of the end gauges (1 and 3) and gauge 2 was 0.324 in. for the full load of 8,000 lb. If it is assumed that approximately one-half (0.12 in.) of the average deflection (0.24 in.) of the end panels was caused by the settlement of the bearing pads and support stiffeners, the centerline deflection of the girder due to flexural action would be 0.44 in. The estimated center deflection of 0.44 in. results in an L/S value of 435 for a span of 16 ft. This is approximately twice the AASHTO limit for pedestrian bridges. The deflection of 0.44 in. compares with a range of values from 0.25 to 0.30 in. computed from a theoretical analysis of the bridge. McCormick and Alper *(3)* describe the finite element model and solution for the three-girder bridge configuration.

Deflections measured during the unloading cycle of the test mirrored the pattern observed during the loading cycle. Both gauges 1 and 2 indicated a residual net deflection, while gauge 3 showed a greater elastic recovery than that measured during the loading cycle. The residual deflection values recorded when the load was removed were somewhat arbitrary, because the starting values indicated as 6,000 lb in Figure 6 were selected as equal to those measured at 6,000 lb during the loading cycle. The actual deflected positions of the gauge reference points were due to the creep of the bearing pads, temperature-induced distortions, and creep of the trussed girders during the period of loading. The differential deflections between gauge 2 and the average values of gauges 1 and 3 would be affected less by these variables than were the direct readings from the individual gauges. A calculation at zero load for the differential residual deflection indicates a value of 0.11 in. While the exact value of the residual deflection is uncertain, the computed value of 0.11 in. should be an indication of the magnitude of the nonelastic deflection that occurred over the test period. Some of the nonelastic deflection is recoverable, however, as discussed in the following section.

Creep Deflections

Elevations of reference points on the deck were measured periodically to determine the creep deflection of the bridge. The reference points were established by installing brass 1/4-in.-diameter machine bolts through the deck with the heads protruding slightly above the top of the wearing surface. Elevations were measured with a surveyor's precision level and an engineer's scale that was read directly to the nearest 0.05 in. Benchmarks were selected at one point on each abutment, and the deck elevations were computed relative to the benchmarks. A difference of 0.24 in. in the elevation of the benchmarks remained constant throughout the 61 days of readings.

Figure 7 presents the variation in air temperature, the average displacement of the supports, and the average displacement of the midspan of the bridge. Readings of the other four reference points were prevented by the location of the drums.

Initial creep readings taken within 30 min after the final load increment was applied correspond to the zero deflection value at time zero in Figures 7*b* and 7*c*. The deflections shown in Figure 7*b* may be attributed principally to the distortion of the elastomeric bearing pads beneath the supports. The data of Figure 7*c* are plotted as movements of the center span with reference to the benchmarks (solid lines) and also with reference to the supports (dashed lines). Dual data points shown on three different days reflect the reduction of load from 8,000 to 6,000 lb on Day 6 and readings taken in early morning and late evening for temperature fluctuation effects on Days 26 and 54.

In general, the deflection data of the supports and the center span clearly follow the ambient temperature fluctuations—when the temperature increased, the deck rose; when the temperature decreased, the deck fell. As might have been expected, the movement of the midspan was more pronounced than the movement of the supports. The supports were shaded from direct sunlight by the bridge superstructure and the abutments but the deck surface was exposed to heating from the sun during the day and rapid cooling during the night. Because of the continual movement of the deck, it was difficult to determine from the available data whether any viscoelastic creep occurred in the superstructure, but if so, it was not detectable during the 61-day test period.

Figure 8 shows the creep recovery, sometimes referred to as an "elastic aftereffect," of the bridge following removal of the live load. Also indicated is the considerable influence of ambient temperature variations on the deflected position of the lower chord of the bridge. The plotted points indicate gauge readings at times of 0, 24.0, 37.0, 45.5, 46.0, and 46.5 hr. Lines connecting the points are not intended to represent the variation in readings, except for the period from 45.5 to 46.5 hr, when the gauges were monitored continuously. Two phenomena were at work to influence the deflection measurements: creep recovery and temperature variation. Regrettably, a careful record of the ambient temperature was not maintained during the test period, principally because temperature changes were not considered to be such an influential factor as they (in hindsight) apparently were. The effect of temperature was clearly demonstrated by the upward movement of the lower chord over a period of 1 hr (from 7 to 8 a.m.), during which time the ambient temperature increased from 60° to 70°F. Values of 0.02, 0.03, and 0.01 in. occurred at gauges 1, 2, and 3, respectively, during that period of time. The movement due to temperature in 1 hr represents approximately 15 percent of the maximum creep recovery measured in 37 hr. Although the change in geometry of the

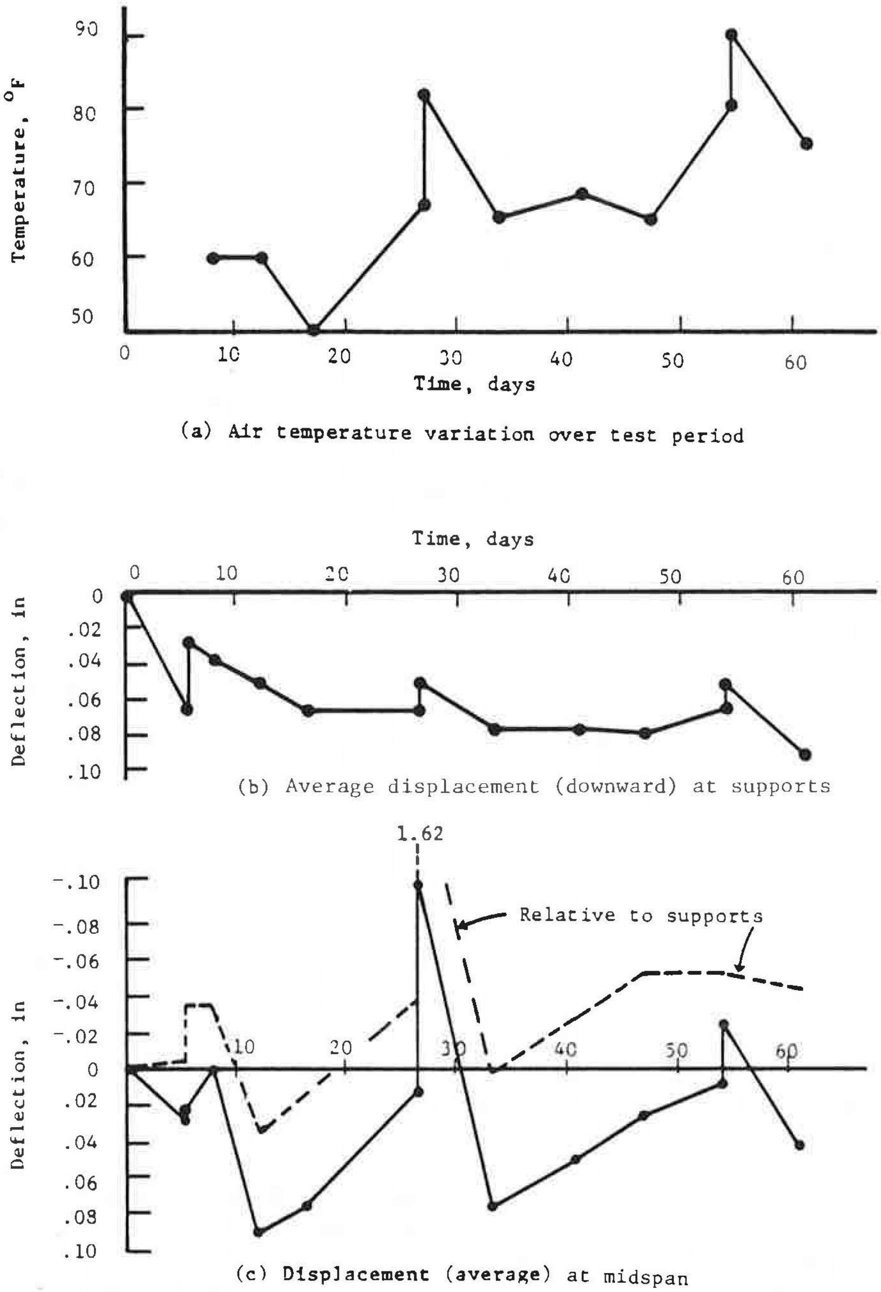

FIGURE 7 ***(a)*** **Air temperature fluctuation,** ***(b)*** **Deflections of deck at supports, and** ***(c)*** **Deflections at midspan.**

structure due to temperature variation is not completely understood, it is believed that the heat absorbed by the deck material when exposed to the sun is the predominating factor for change.

Analyses of the measurements for deflection recovery indicated that essentially all of the movement of the structure occurred at the supports. That is, after allowing for temperature fluctuations, the movement of gauge 2 relative to gauges 1 and 3 was nearly equal over the observation period of 46.5 hr. Therefore, it appears that little, if any, viscoelastic creep occurred in the bridge itself and that the measured rebound of the structure was due to the recovery of the elastomeric bearing pads. A similar observation was made in the creep study of a single girder over a period of 3 months *(1)*.

Elastic Strains

When the structure was built, 20 electrical resistance strain gauges (EA–06–250–BF–350 by Micromeasurements Co.) were bonded to various elements of the deck, web, and lower chord as shown in Figure 9. After 8 years in storage, 18 gauges remained functional and were attached to two portable switching units and one indicator (Bud Co., Model P350).

Strain measurements were recorded during the period of the application of live load and at intervals during the period of the creep test. The strain data obtained during the load test were considered reasonably accurate, but the creep strain data were not considered to be quantitatively correct. Several days after completion of the load test, the switching units were exposed to

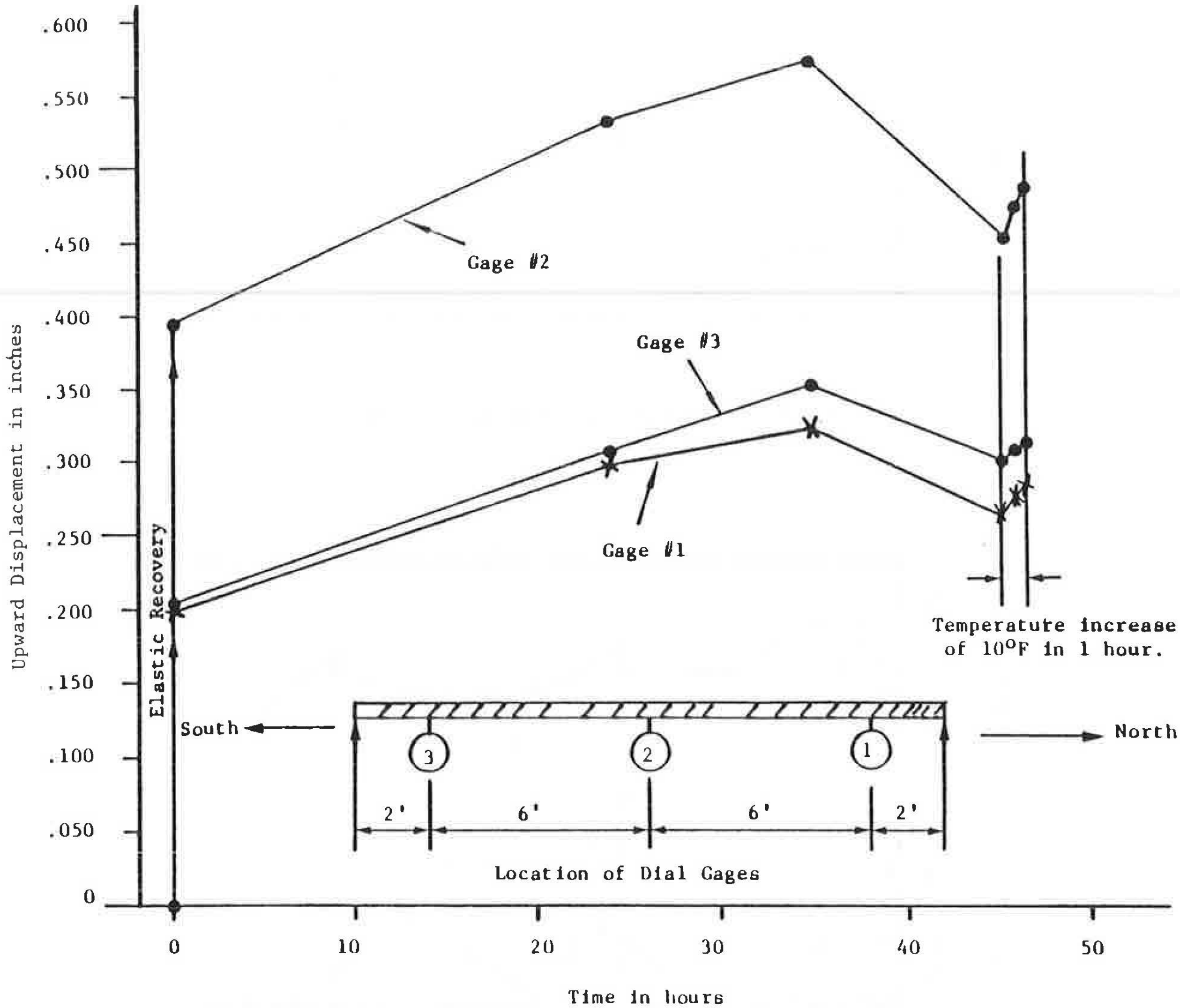

FIGURE 8 Deflection recovery of bridge following load removal.

moisture and an uncontrollable drift in the strain values was noted on subsequent readings.

Figure 10 presents data from six gauges mounted on the inclined web elements. Gages 2, 3, and 4 in the southern end panel tracked each other closely during the period of loading and ranged from 11,000 to 14,000 µin./in. when the total load was applied. These three gauges indicated no reversal of strain increase in the end panel. The difference in strains between gauges 2 and 3 is indicative of a possible load-carrying discrepancy between one of the outside girders and the inside girder, whereas the data from gauges 3 and 4 indicate similar load distributions between the center girder and the other outside girder. Gages 5, 6, and 7 in the second and third panels from the southern end indicated approximately the same increased rate of strain with the application of the load as did gauges 2, 3, and 4 until loads were applied directly to their respective panels. At that time, the strains reversed direction. This change in direction reflected the change in magnitude of the negative shearing force in the interior panels as the load was increased and more uniformly distributed along the length of the bridge.

Figure 11 presents strain data from five gauges mounted on the lower chords. All gauges indicated increasing strain values as the load was increased on the bridge. As expected, a sharp reduction in the strain rate in all five gauges was noted with the application of the last increment of load. As noted previously, the last load increment was applied in the southern end panel and, therefore, should not have affected the flexural stresses in the girders as much as the prior load increments had. Figure 11 shows that a relatively narrow range of strains was measured throughout the four panels and two girders monitored by the gauges, particularly through the application of the first 5,000 lb of load. The narrow range of strain values indicates that the lower chord elements were stressed as uniformly as might be expected with the nonuniform arrangement of the test load. The relative uniformity of stress indicates that the design procedure used to dimension the chord elements produced an efficient structural configuration. Also, the lateral transfer of the load and the interaction between girders during the load test appeared to be satisfactory as indicated by the random variation in strain values in the chords of the inside and outside girders.

Using a tensile elastic modulus value of 7×10^6 lbf/in.^{2} for the lower chord and web elements *(4)*, the strains shown in Figures 10 and 11 may be converted to axial tensile stress in the elements. The inclined web elements, therefore, developed

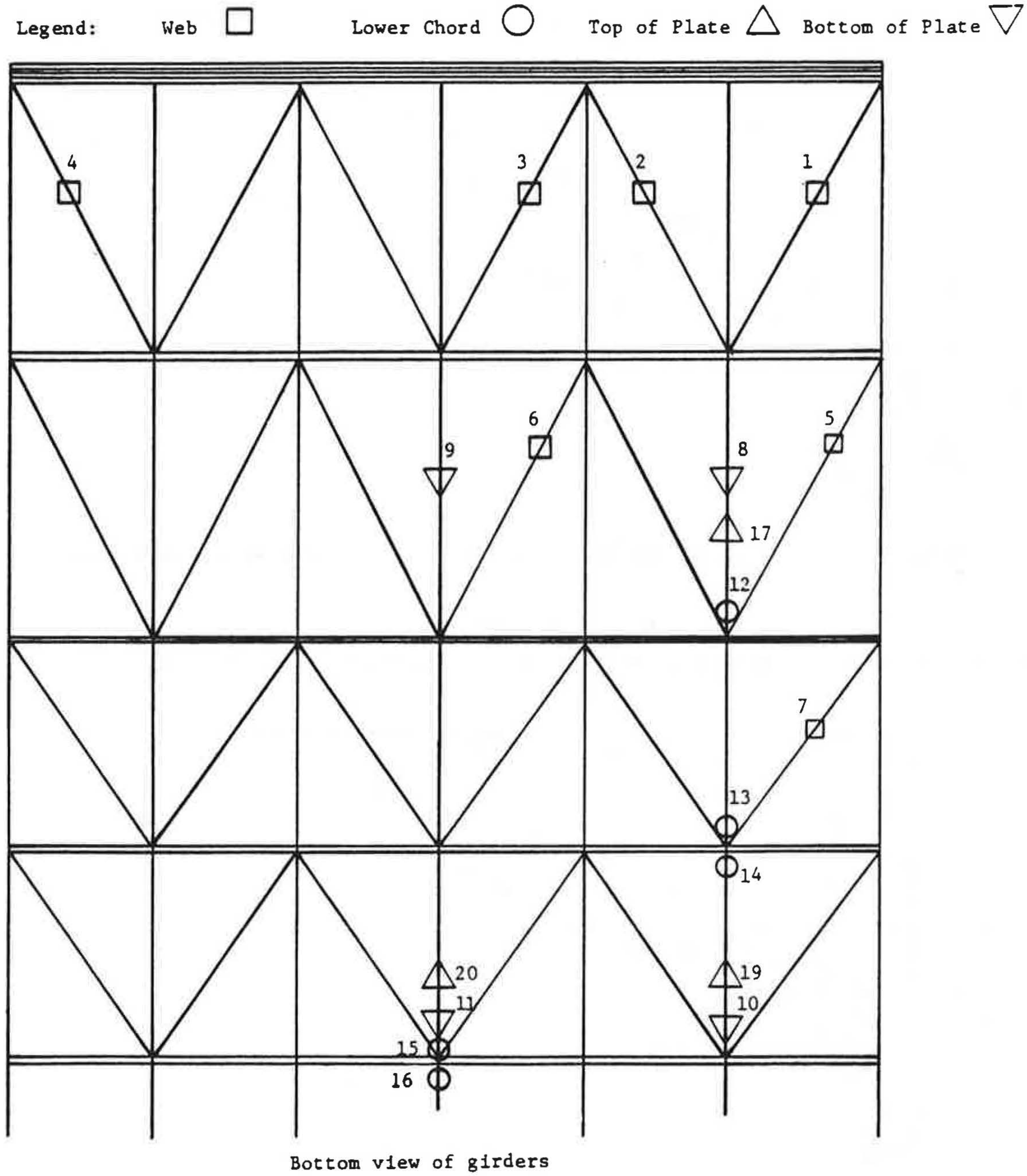

FIGURE 9 Location of electrical strain gauges bonded to elements of the bridge superstructure.

stresses ranging from 4,200 to 9,800 lbf/in.^{2}. Similarly, the lower chord stresses ranged from 3,150 to 6,650 lbf/in.^{2}. With a conservative ultimate strength value for the tensile strands exceeding 50,000 lbf/in.^{2}, the safety factors against tensile failure of the material exceed 5. Obviously, the design limitation of the GRP material system is the deflection of the structure due to the low modulus of the composite or the shear strength of the connections.

Elastic strains monitored in the top flanges and cover plates of the girders were erratic and, therefore, are not discussed further here. It has been noted previously that the thin deck assembly of pultruded plates and polymer concrete overlay deflected locally when the drums were applied to the bridge deck. In addition, slight buckling (both upward and downward) of the plates was observed as the live load was increased across the span. The combination of these two effects accounted for the erratic behavior of the strain gauges.

Creep Strains

Figure 12 presents representative strain data from five active strain gauges and a fixed reference circuit over the duration of the creep test. The reference circuit was fixed at 1,000 μin./in. as a check on the stability of the measuring indicator. As can be seen in Figure 12, the reference circuit remained essentially unchanged for the first 6 days of the creep test. Thereafter, wide fluctuations appeared in the data until, finally, the drift in the reference circuit exceeded the range of the indicator. Also, note

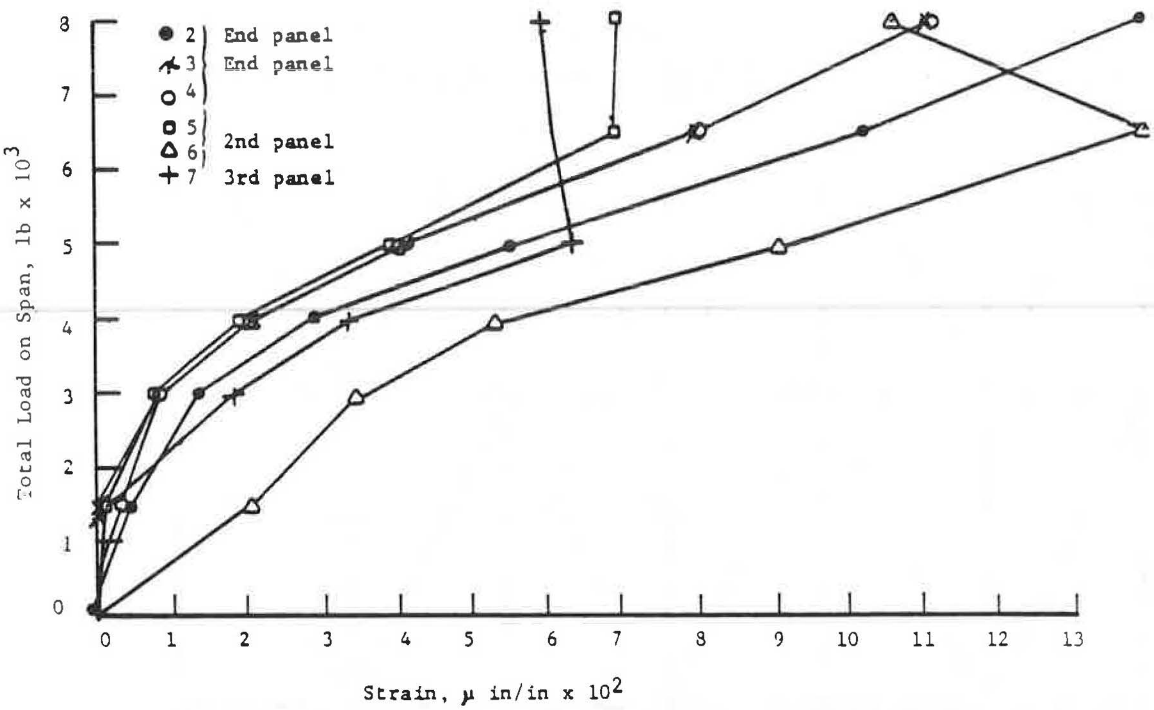

FIGURE 10 Elastic strain in inclined web elements in the girders due to application of loads.

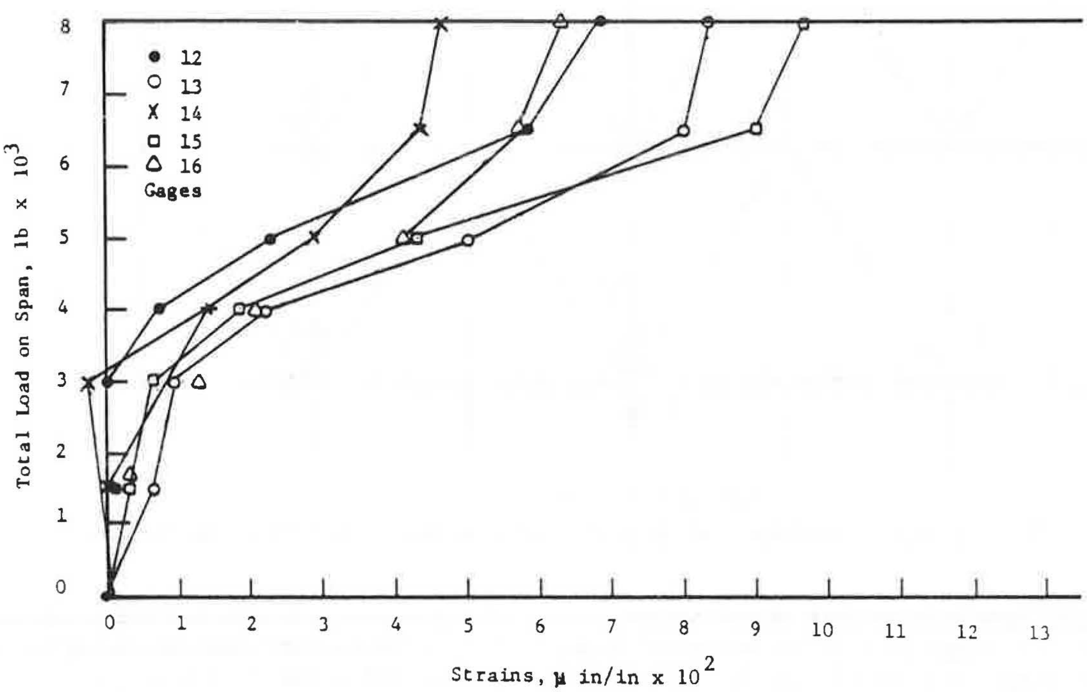

FIGURE 11 Elastic strain in lower chord elements in the girder due to application of loads.

that the magnitude of variations in the reference considerably exceeded those of the measuring circuits, even though the directions of the deviations were the same. Efforts were made to correct the obvious malfunction of the instrument as soon as the deviations were noted, but to no avail. Apparently, moisture penetrated the switching units and altered an internal resistance common to all of the gauge circuits to produce the results obtained. The measurements were discontinued after 54 days.

It is not believed that the fluctuations of the measuring instrument were due to changes in the ambient temperature. The comparison of the strain deviations with the temperature fluctuations shown in Figure 9 indicates little to no correlation. Because of the gross deviations of the measurements, the data are worthless for quantitative use. However, all of the gauges underwent the same magnitude of creep strain, whatever it was. If the first 6 days of the creep strain readings might be

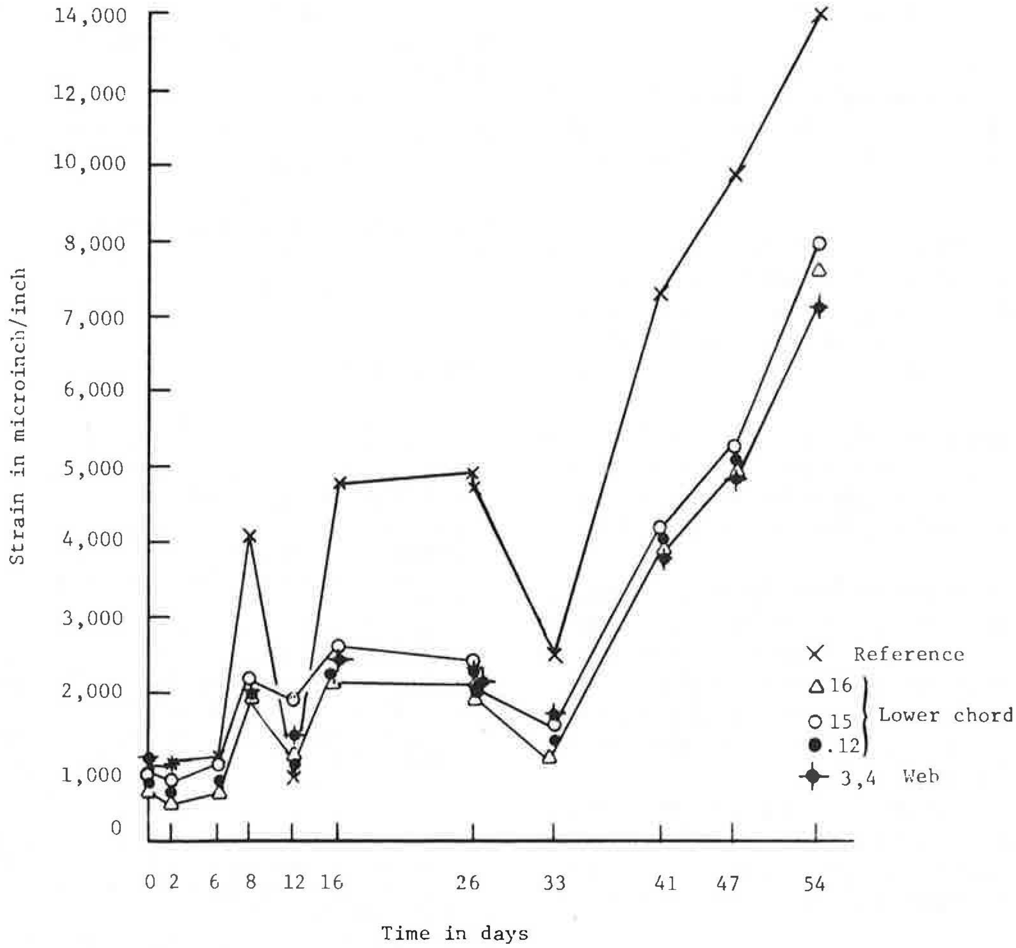

FIGURE 12 Strain in selected elements versus time with full live load.

considered reliable, it appears that no measurable creep occurred in the three lower chord and two web elements shown in Figure 12. The absence of detectable creep strain in the individual elements of the girders corresponds to the deflection measurements of the lower chord and the deck as discussed previously. The relatively low stresses developed in the elements due to the full live load accounts for the absence of creep in the composite material.

HANDLING AND TRANSPORTATION OF BRIDGE

Unexpected but valuable information was obtained when the superstructure was moved to the test site. The bridge was loaded onto a lowboy by means of a front-end loader and transported approximately 8 mi to the erection site. During loading and handling, the outside stiffeners at both ends of the center girder were broken where a lifting chain made contact. The extent of the damage to a stiffener may be seen in Figure 13. Aside from some abrasion on the edge of the cover plate on the deck, no other damage to any of the structural elements was observed. The displaced stiffeners were realigned manually with the lower chord connector and reinforced by cutting and bonding a pultruded GRP plate 1/2 in. thick to the mating stiffeners. The repaired members showed no distress during the load tests.

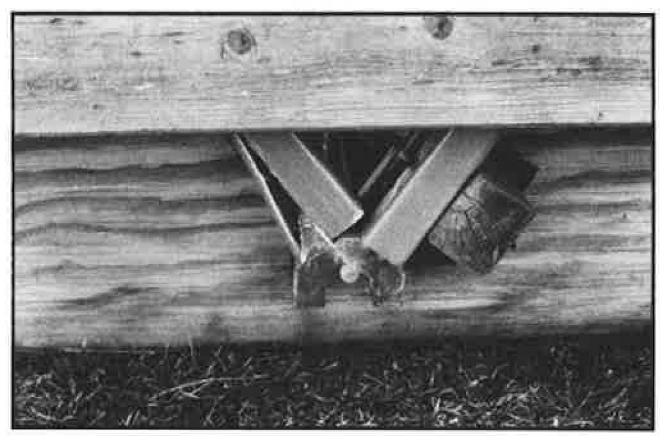

FIGURE 13 Damaged web stiffener at the end of the center girder.

While improper handling of the bridge may have been disastrous, the episode just described turned out to be quite valuable because it provided a test of the toughness of the structure that would not have been conducted intentionally for fear of irreparable damage to the joints and elements.

ASSESSMENT OF PERFORMANCE

The following assessment of the performance of the GRP pedestrian bridge is based on both the qualitative observations made by the project director during the handling and testing of the structure and the quantitative data obtained from the load tests.

1. Elements of the superstructure resisted abuse from lifting and handling better than was anticipated from work with previous laboratory test specimens. Although it was demonstrated that elements could be fractured by highly concentrated forces from lifting chains or bars, the fractures were not extensive and were easily repaired.

2. The bond between the polymer concrete wearing course and the GRP deck plate remained intact throughout the removal of the superstructure from the abutments. No signs of distress or spalling were detected in spite of severe distortion of the deck assembly.

3. The method of supporting the bridge by distributing the bearing pressure from the seats to the faces of the vertical stiffeners at the ends of the girders was quite satisfactory. The elastomeric bearing pad apparently assisted in distributing reactive forces uniformly. However, it was inconvenient from a handling standpoint to be unable to rest the structure temporarily on a horizontal surface.

4. The influence of temperature changes was reversible but pronounced on strains in the elements and deflection of the bridge. The likelihood of large secondary stresses in the superstructure is high if thermal distortions are constrained by supports.

5. Strains appeared to be reasonably uniform along the different elements monitored during both the elastic and creep load tests. Stresses computed from measured strains did not exceed 10,000 lbf/in.^{2} at full live load.

6. Creep deformations in the elements and joints of the superstructure appeared to be nonexistent over the test period of 61 days.

7. The ratio of live to dead load for the creep test was over 3:1 and for the elastic test over 4:1. Even though the replacement of the structural concrete deck in the original design with a nonstructural wearing course reduced the calculated elastic strength capacity of the structure by 98 percent, the deflection of the prototype was less than three times that prescribed by AASHTO specifications.

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