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FINAL REPORT

FIELD STUDY OF A PEDESTRIAN BRIDGE OF REINFORCED PLASTIC

by

Fred C. McCormick
Faculty Research Scientist

(The opinions, findings, and conclusions expressed in this report are those of the author and not necessarily those of the sponsoring agencies.)

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ABSTRACT

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 4.0 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 upon 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 3.0 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.

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INTRODUCTION

The design, development, fabrication and laboratory testing of the components for the pedestrian bridge described in this report have been discussed in previous publications.[1,2,3,4,5]

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 GRP (glass-reinforced plastic) plates bonded to the top flange of each girder. An overview of the superstructure is shown in Figure 1 and a detailed description of the configuration is presented in reference 1. The foundation structure was reinforced concrete and consisted of footings, backwalls, and precast seats. The seats had been formed to match the triangular shape of the bearing surfaces of each of the bridge girders.

A multiple layer polymer concrete 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 1/2 in and added a weight of approximately 1,000 lb to the bridge. The estimated weight of the superstructure prior to the placement of the polymer concrete was 900 lb. The original design for a wearing course and curb was a 4 in thick concrete slab which also functioned as the principal flexural compression element for the structure. The replacement of the structural deck with a nonstructural wearing course reduced the calculated design capacity of the bridge by 98% (from 85 lb/ft^2 to 1.9 lb/ft^2) for equal deflections.

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 principal use of the bridge was for pedestrians and golfers using electric carts weighing approximately 1,000 lb fully loaded. The superstructure was erected on March 29, 1985, as will be described later. A wearing course was applied from April 3 to April 5 and the load test initiated on April 17. On May 24, a heavy rainfall in the park caused severe erosion of the region adjacent to the bridge foundation. Because of anticipated

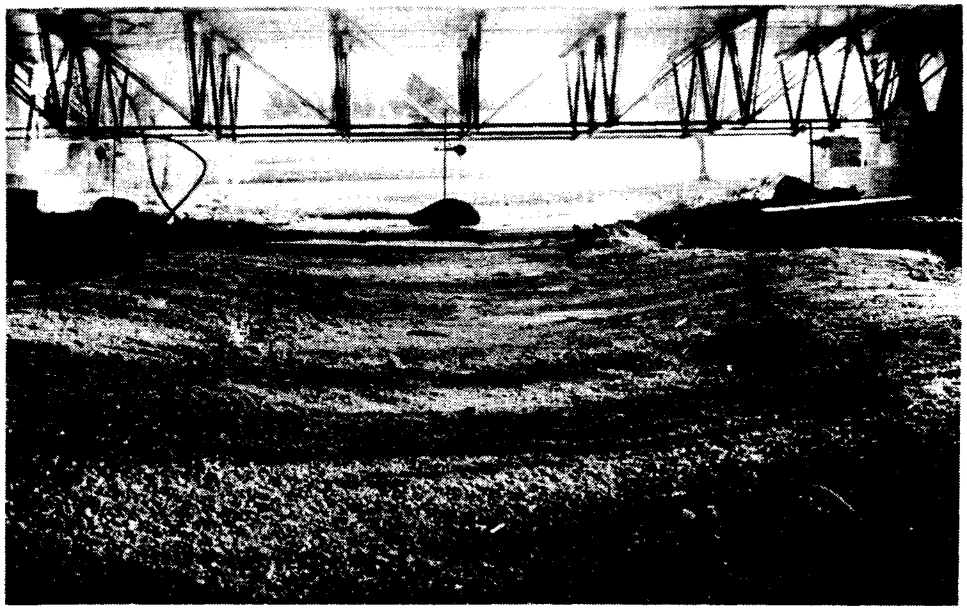


Figure 1. Pedestrian bridge in place over the discharge channel of an irrigation pond at Pen Park.

continued erosion that might lead to the destruction of the bridge, and for other reasons, the load test was terminated on June 17. On June 19, the superstructure was removed from the abutments and transported to a storage site in Pen Park. The bridge may be relocated in the park or elsewhere by the city when a suitable site is determined. While it was intended that a weathering and service fatigue study would be conducted following the load tests, an unexpected bonus of information was provided by the movement of the superstructure, as will be discussed later.

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. The effects of weathering and service loads will not be determined until the structure is relocated at another site.

ERECTION PROCEDURES

The precast concrete bridge seats were positioned on footings and anchored by casting the backwalls against them as shown in Figure 2. Elastomeric (75 durometer neoprene) pads were placed on the bearing surfaces of the seats prior to installing the superstructure. These pads (2 layers, 1/4 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 (900 lb) of the bridge, it was moved manually from a staging area in the park and positioned on the seats without the assistance of mechanical equipment. Approximately one hour was required to assemble a crew of 12 workmen, remove the wooden shipping braces, and install the bridge on the seats. Figures 3, 4, and 5 show the sequence of installation.

Successive layers of polymer concrete were applied at intervals of approximately 2 hours to allow sufficient time for the resin binder to cure. Figure 6 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. Figure 7 shows the texture of the surface of the polymer concrete after it had cured. Excess sand was removed after each layer was cured in preparation for the next resin coating. Eight layers of sand and resin were applied over various areas of the deck to provide a crown along the center of the deck to facilitate water drainage to the sides of the otherwise flat surface. Application of the polymer concrete in successive layers also permitted the buildup of a greater thickness at mid-span 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 #1 grading was used on layers 1-4 and a Morie #2 on the top 4 layers. 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)

Figure 2. Concrete abutments and precast concrete seats prior to installation of superstructure.

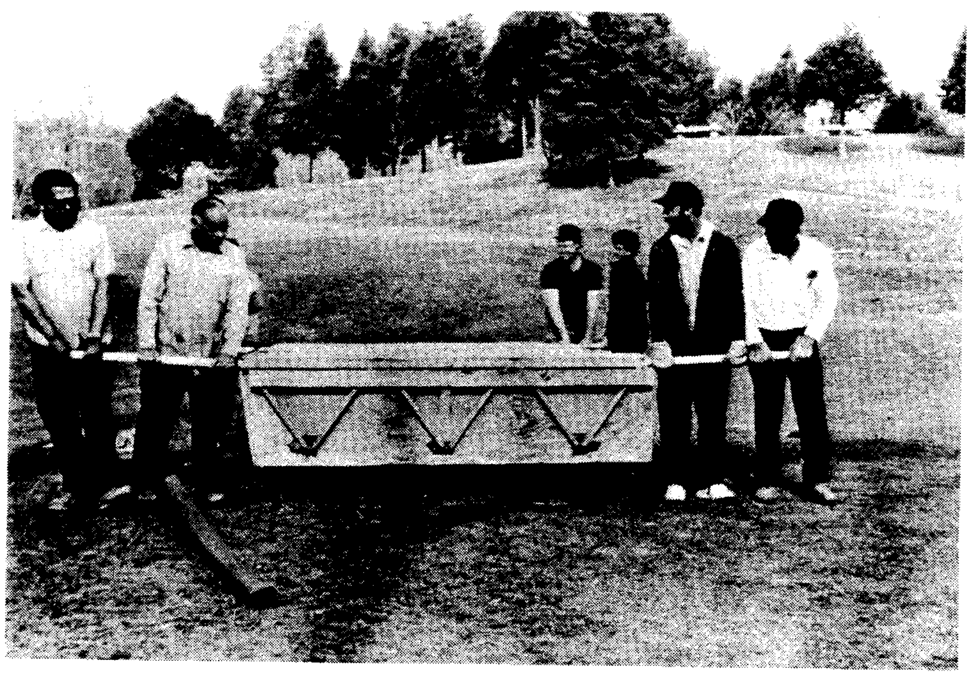


Figure 3. Movement of superstructure from staging area to abutments.

Figure 4. Lifting superstructure onto the precast concrete seats.

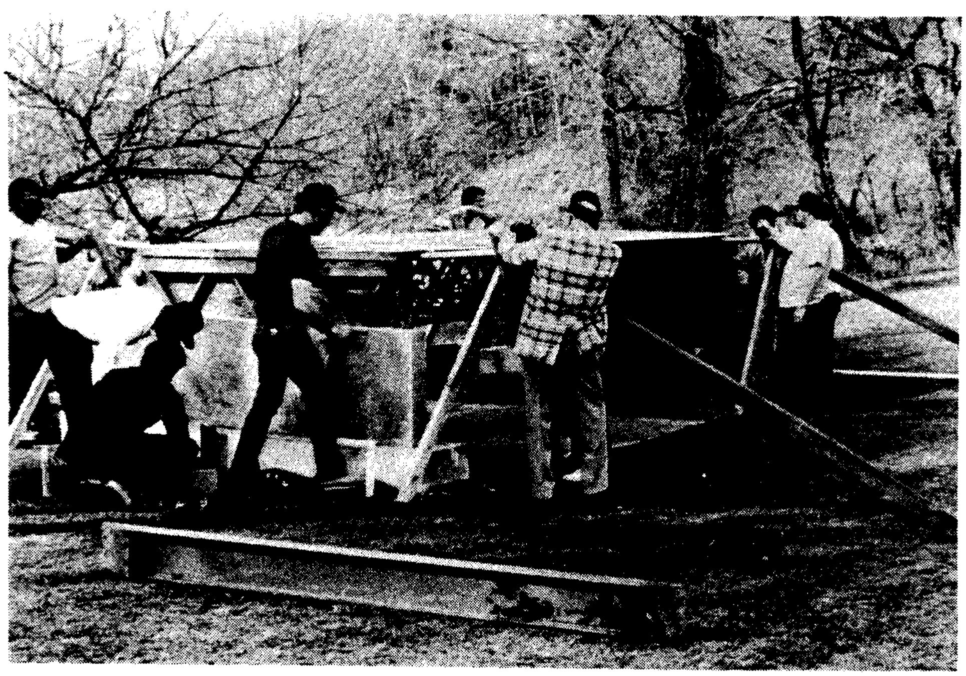


Figure 5. Adjusting bridge and bearing pads prior to final installation.

- Figure 6. Sand aggregate applied to polyester resin to form polymer concrete wearing course.

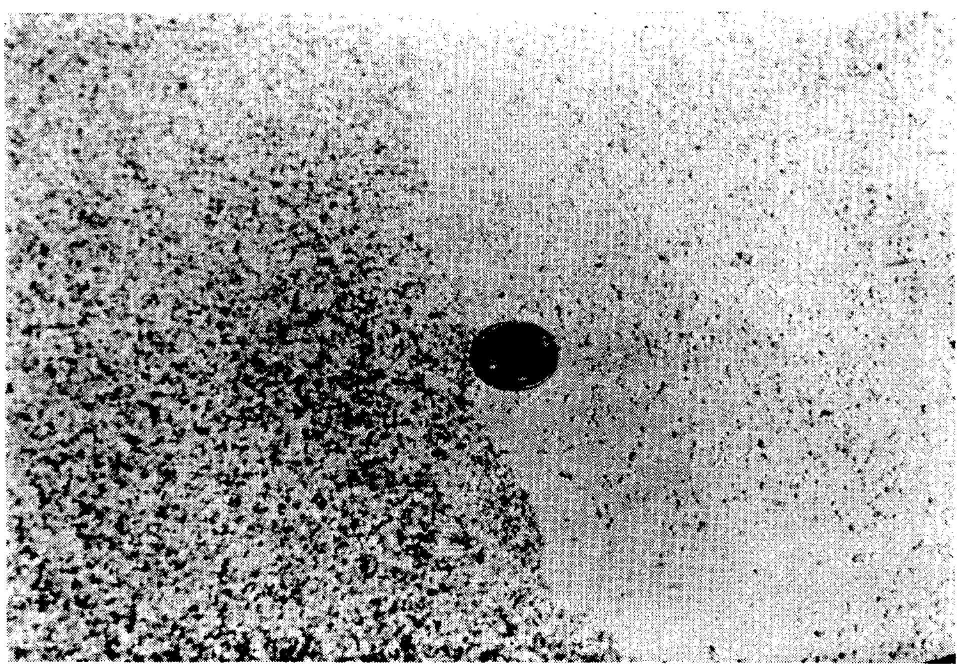


Figure 7. Texture of the surface of the cured polymer concrete compared with an uncoated surface of the GRP cover plate. Note loose particles of sand on cover plate.

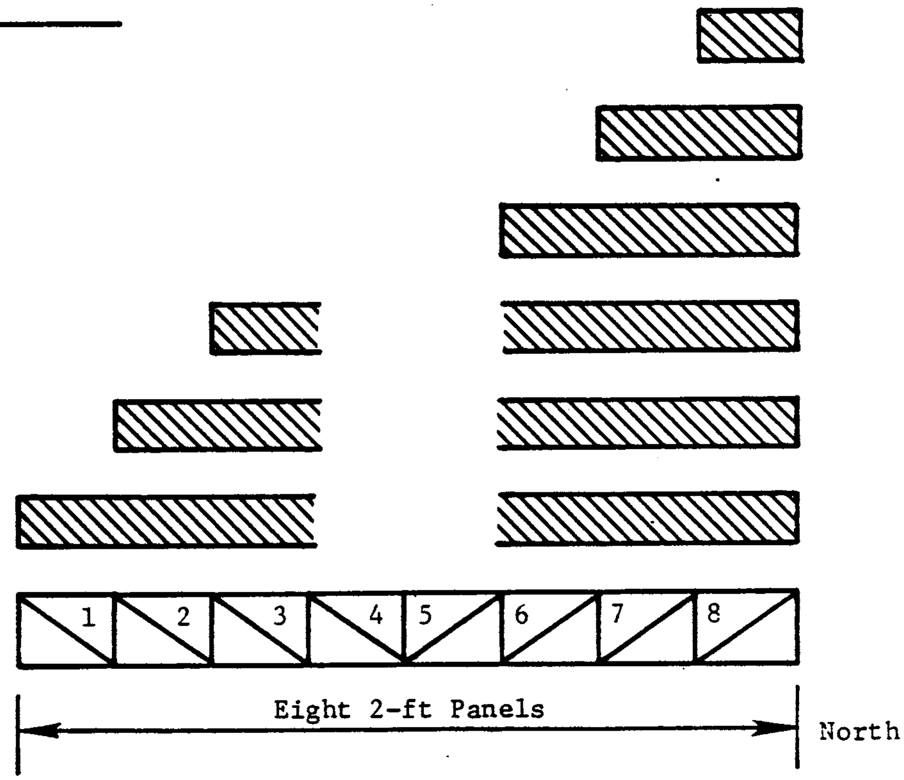
Consideration had been given to placing the polymer concrete wearing course on the deck prior to moving the bridge to Pen Park. However, this alternative was rejected because it would increase the weight considerably and because it was questionable if the bond between the concrete and the deck plate 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 $\approx$ 500 lb total per drum) in the sequence shown in Figure 8. Note that the two center panels of the bridge were not loaded. The progressive manner of loading provided a "moving" load 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 cover plates, and 1/2 in thick polymer concrete) 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 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.

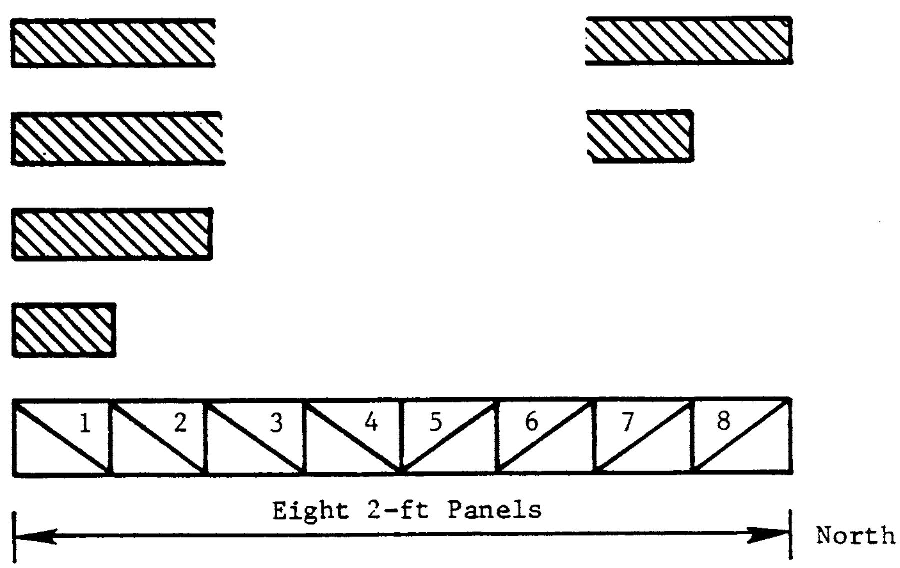
A live load of 8,000 lb on the deck provided an average load of 71.4 lb/ft^2 based on a total surface area of 112 ft^2, or 83.3 lb/ft^2 based on the usable surface of 96 ft^2 between curbs. Since the structure had been designed for a live load of 85 lb/ft^2 with the 4 in thick portland cement concrete 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 polymer concrete confirmed the expectation that the structural contribution of the wearing surface would be slight, particularly during the long-term creep test of the bridge.

Load Sequence	Total Weight, lb
1	1,500
2	3,000
3	4,000
4	5,000
5	6,000
6	8,000



Progressive application of test loads.

Load Sequence	Total Weight, lb
Initial	6,000
1	4,500
2	3,000
3	1,500

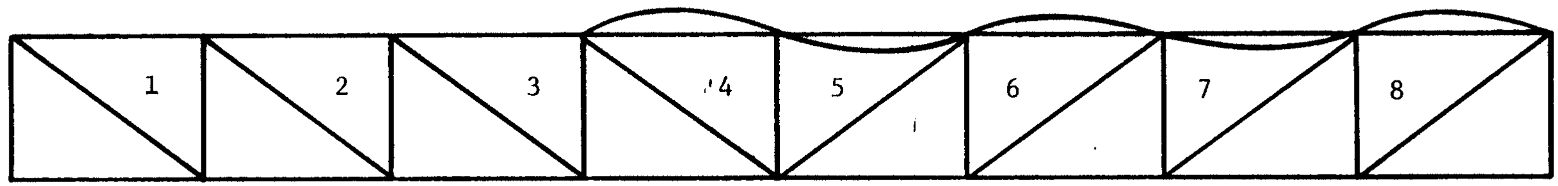


Progressive removal of test loads.

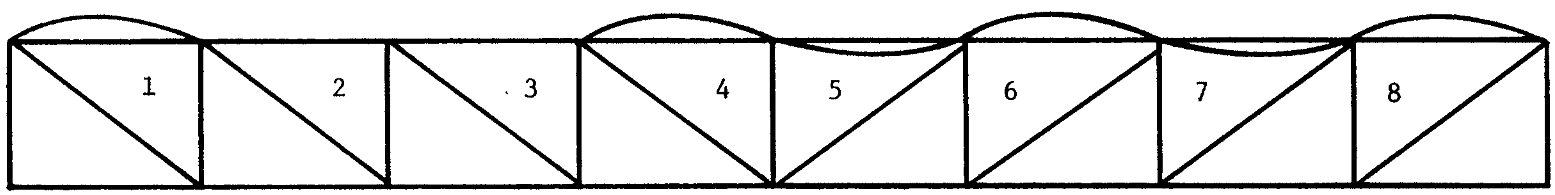
Figure 8. Sequence of application and removal of loads to bridge deck by means of water-filled drums.

Early evidence of excessive compressive stresses in the flanges was manifested in a slight buckling of the flanges as sketched in Figure 9. The directions of the plate displacements were determined visually with the aid of a straightedge. No quantitative measurement of the displacements was attempted since most amplitudes were very slight. The maximum amplitude was estimated at 0.10 in and occurred in panel 8 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. As shown in Figure 9(c), the transverse stiffeners typically were bowed downward at the center of the bridge. 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, on April 23, two drums of water were removed from panel 3 and from panel 6 (Figure 8) 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.

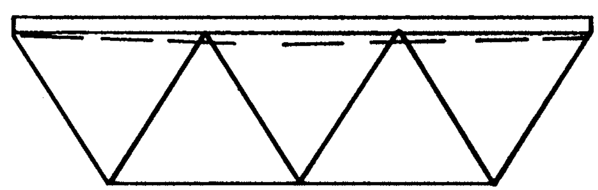
After 61 days under live load, all drums were removed, and rebound deflections of the bridge were measured over a period of 46.5 hours. The unloading sequence for live load removal was depicted in Figure 8. Figure 10 shows drums being emptied on the deck to minimize probable disturbance of the deck due to the shifting of heavy drums to the edge of the deck. 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 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 had occurred in the opposite end of the girders when the load was applied.



(a) Exterior girder on east side



(b) Exterior girder on west side



(c) Typical displacement of cross section of deck.

Figure 9. Exaggerated representation of elastic buckling and displacement of deck assembly due to live load application of 8,000 lb. Maximum amplitude estimated at 0.10 in.

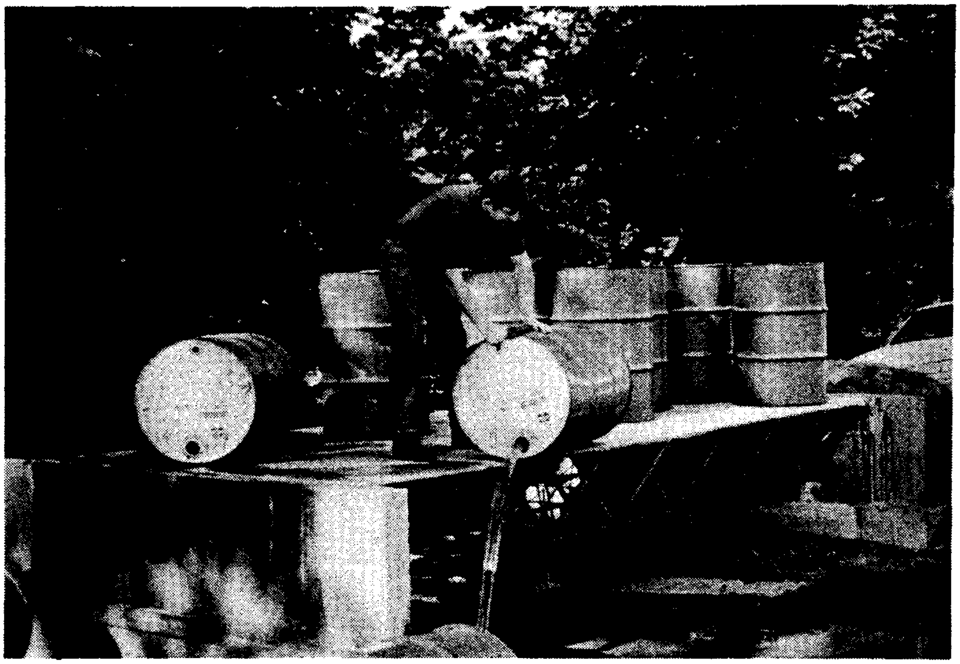


Figure 10. Removal of live load from the bridge deck.

Elastic Deflections

Lower chord vertical deflections were measured by dial-gage indicators (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 gages remained in place for only a short time following the application and removal of the live load. Figure 11 is a photograph of the loaded bridge showing the dial gages and strain gage instrumentation in position. The locations of the gages beneath the girder and the measured deflections of the lower chord are shown graphically in Figure 12.

All deflection measurements included movement of the end supports in the seats due to compression of the elastomeric 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 gages 1 and 3. This supposition is supported by the deck deflection data as discussed later. The larger deflection, shown by gage 1 as compared with that by gage 3, resulted from the application (and removal) of the load progressively from the northern end of the bridge to the southern end.

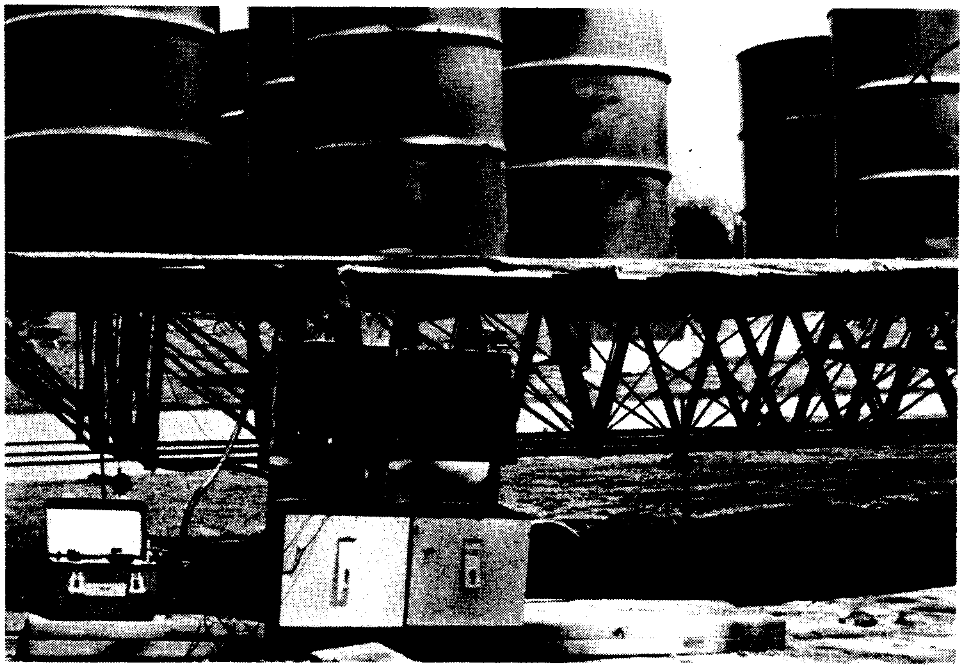


Figure 11. Loaded bridge with strain gage instrumentation and dial gages in position.

It was expected that the deflection at gage 3 would reach that of gage 1 when the full load was applied, since an effort was made to distribute the filled drums symmetrically about mid-span. 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 gages (1 and 3) and gage 2 was 0.324 in for the full load of 8,000 lb. If it is assumed that approximately half (0.12 in) of the average deflection of the end panels (0.24 in) 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.* The computations for the range of values will be discussed later.

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*See Reference 3 for a description of the finite element model and solution for the three-girder bridge configuration.

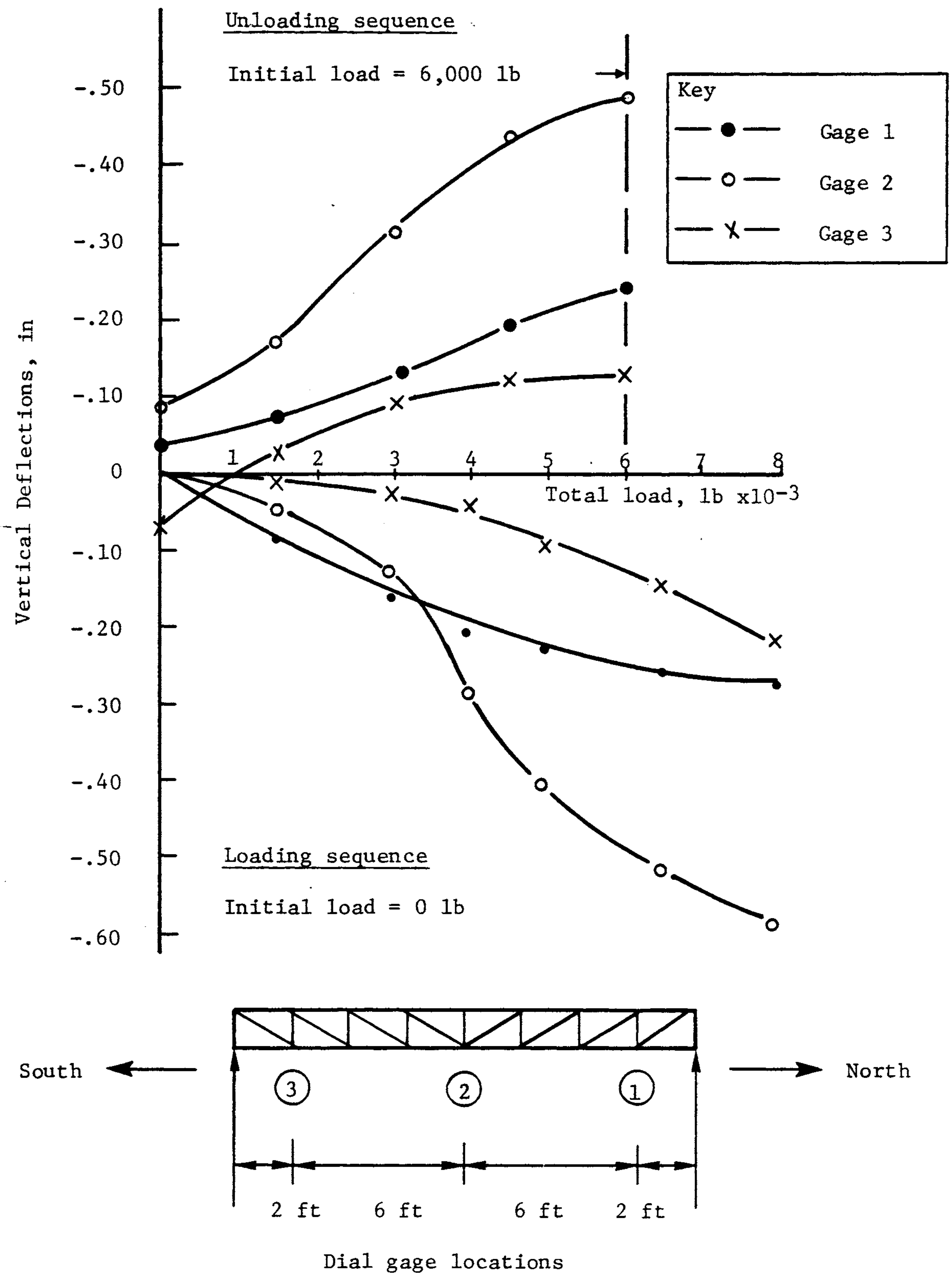


Figure 12. Deflections of the lower chord of the center girder as a function of live load.

Deflections measured during the unloading cycle of the test mirrored the pattern observed during the loading cycle. Both gages 1 and 2 indicated a residual net deflection, while gage 3 showed a greater elastic recovery than that measured during the loading cycle. The residual deflection values recorded when the load had been removed were somewhat arbitrary, since the starting values (indicated as 6,000 lb in Figure 12) were selected as equal to those measured at 6,000 lb during the loading cycle. The actual deflected positions of the gage 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 gage 2 and the average values of gages 1 and 3 would be affected less by these variables than were the direct readings from the individual gages. 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 which 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 as shown in Figure 13 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 which 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 14 presents the variation in air temperature, the average displacement of the supports (points 1 and 7) and the average displacement of the midspan of the bridge (points 4, 5, and 6). Readings of the other four reference points were prevented by the location of the drums.

Initial creep readings were taken within 30 minutes after the final load increment was applied and correspond to the zero deflection value at time zero in Figure 14(b) and (c). Since reference points 1 and 7 were located very close to the supports, the deflections shown in Figure 14(b) may be attributed principally to the distortion of the elastomeric bearing pads beneath the supports. The data of Figure 14(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.

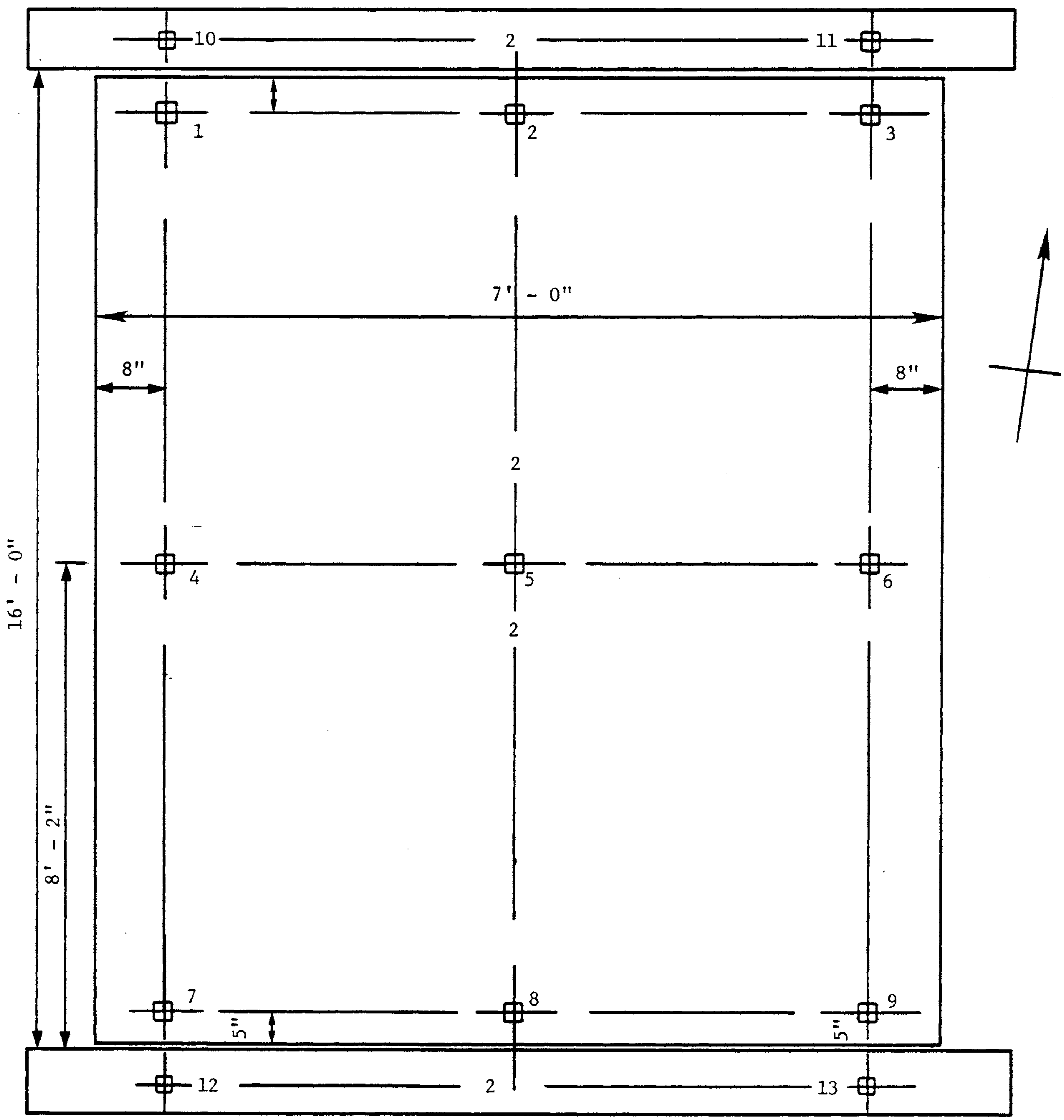
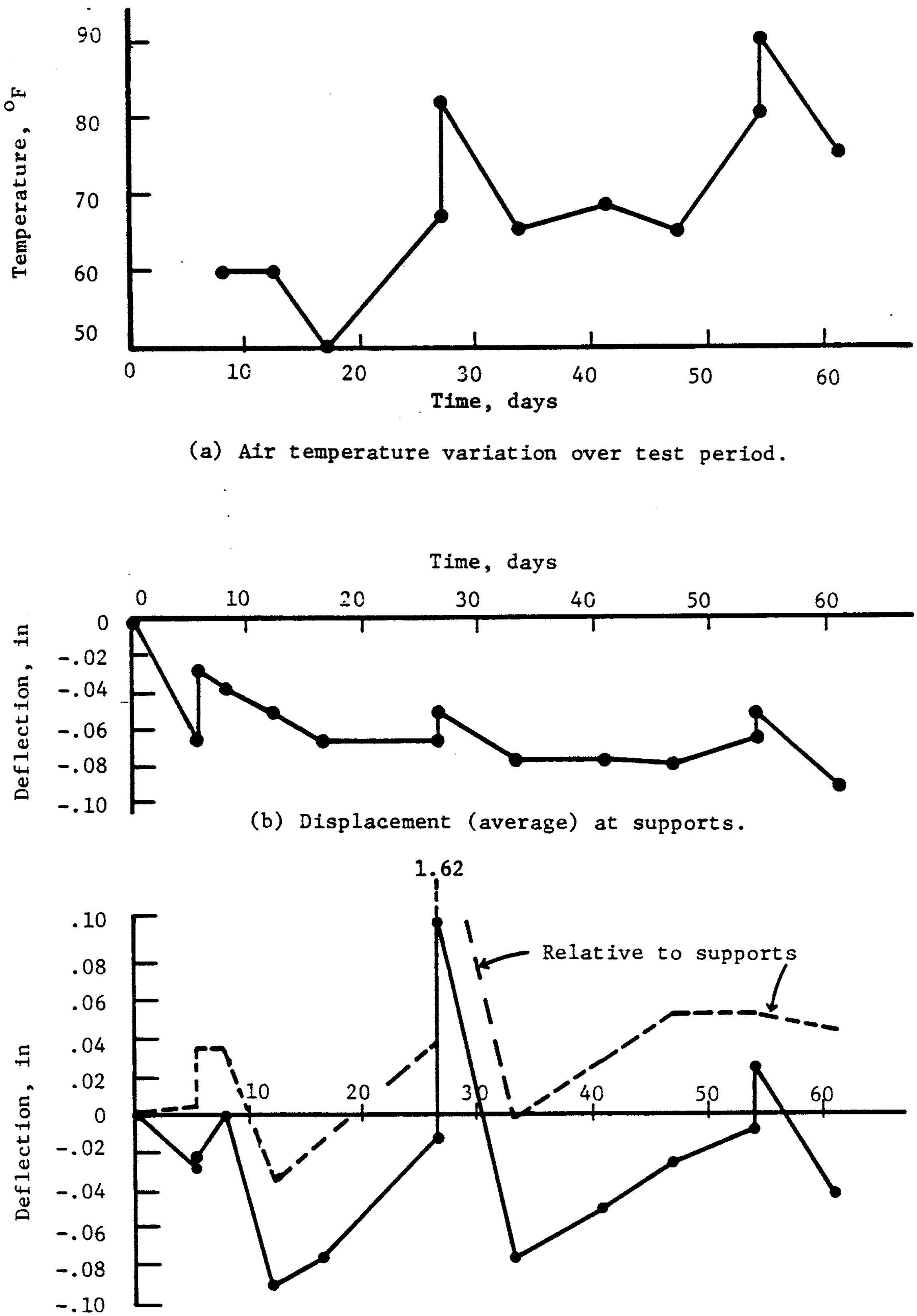


Figure 13. Location of reference points on deck used for deflection measurements.



(a) Air temperature variation over test period.

(b) Displacement (average) at supports.

(c) Displacement (average) at mid-span

Figure 14. Air temperature fluctuation (a) and deflections of the deck at the supports (b) and at the mid-span of the bridge (c).

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 may be expected, the movement of the mid-span 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 15 charts the creep recovery (sometimes referred to as an "elastic after affect") 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 gage readings at times of zero, 24.0, 37.0, 45.5, 46.0 and 46.5 hours. Lines connecting the points are not intended to represent the variation in readings, except for the period from 45.5 to 46.5 hours, when the gages 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 temperature values shown were recorded from thermometer readings at the site or from the readings made in the Charlottesville area. The effect of temperature was clearly demonstrated by the increased deflection (an upward movement of the lower chord) over a period of 1 hour (from 7 to 8 a.m. on June 19, 1985), during which time the ambient temperature increased from 60° to 70°F. Values of 0.02, 0.03 and 0.01 in occurred at gages 1, 2 and 3, respectively, during that period of time. The movement due to temperature in 1 hour represents approximately 15% of the maximum creep recovery measured in 37 hours. While the change in geometry of the 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 was 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 form temperature fluctuations, the movement of gage 2 relative to gages 1 and 3 was nearly equal over the observation period of 46.5 hours. 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 as reported in reference 1.

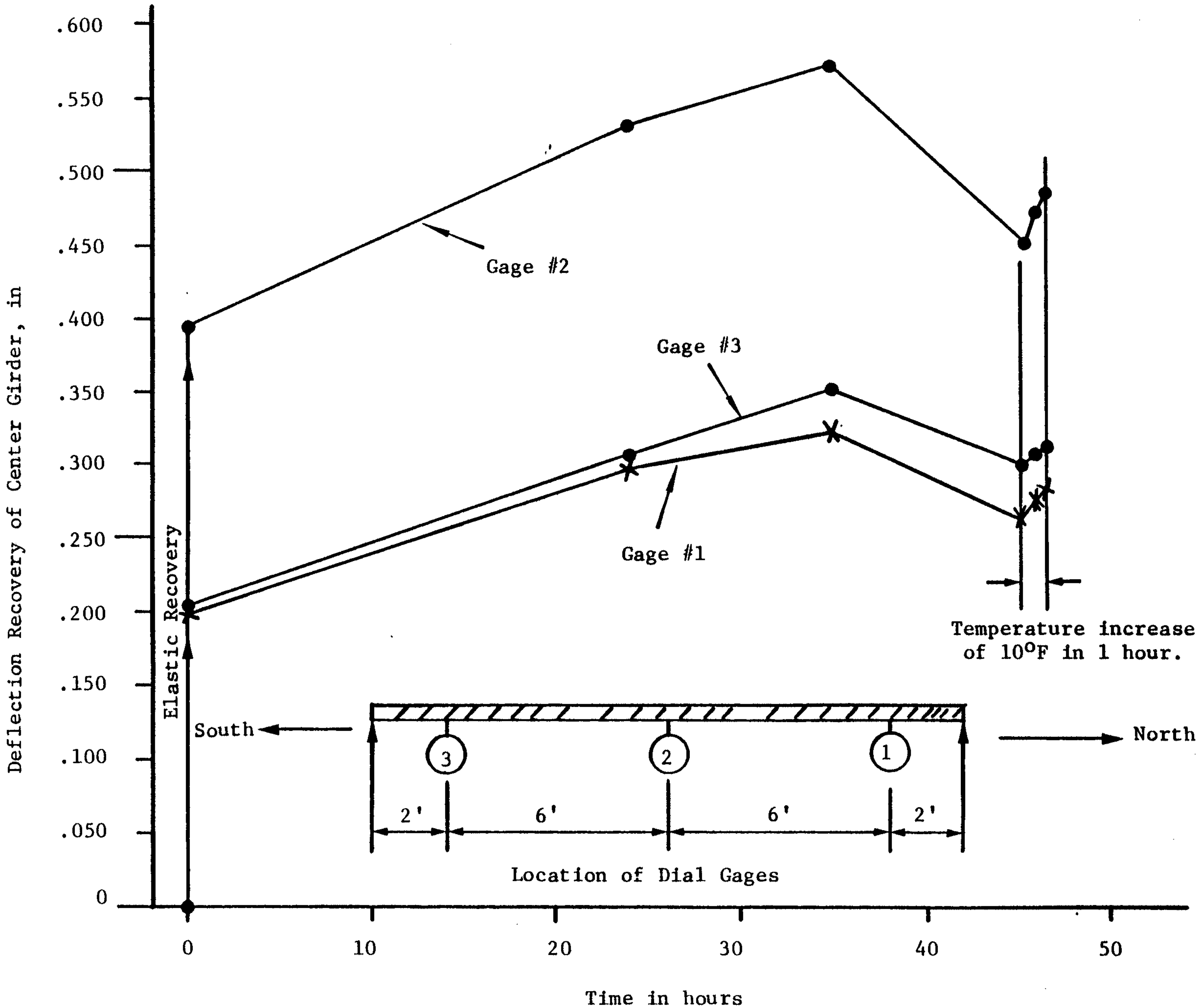


Figure 15. Deflection recovery of the bridge following load removal. The effect of a temperature increase is shown for a period of 1 hour.

Elastic Strains

When the structure was built, 20 electrical resistance strain gages (EA-06-250-BF-350 by Micromeasurements Company) were bonded to various elements of the deck, web, and lower chord as shown in Figure 16. After 8 years in storage, 18 gages remained functional and were attached to two portable switching units and one indicator (Bud Company, Model P350) as was shown in Figure 11.

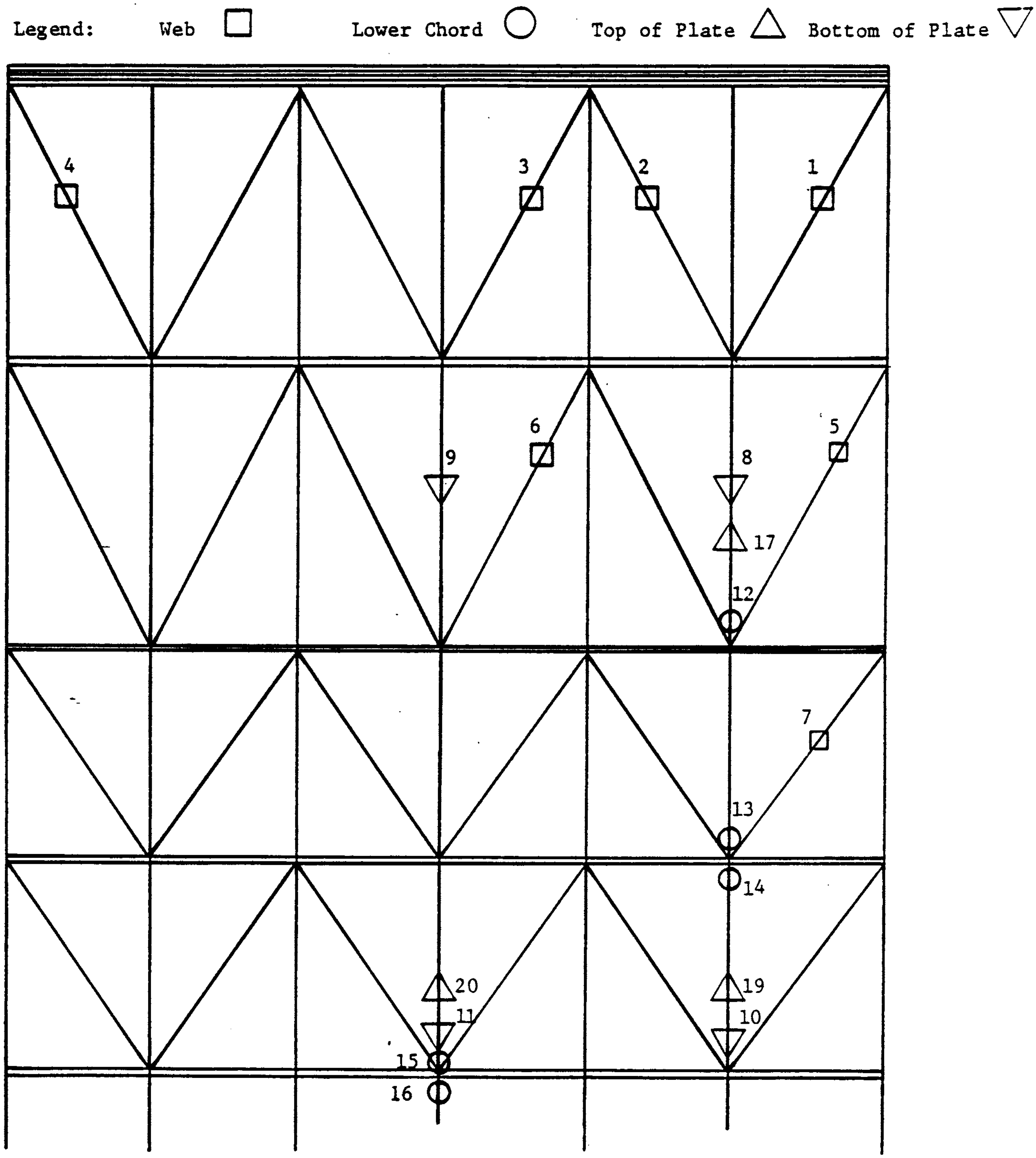
Strain measurements were recorded during the period of the application of live load and at intervals during the period of the creep test. The strain data obtained during the load test were considered reasonably accurate, but the creep strain data were not considered to be quantitatively correct. Several days after completion of the load test, the switching units were exposed to moisture and an uncontrollable drift in the strain values was noted on subsequent readings.

Figure 17 presents data from six gages 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 gages indicated no reversal of strain increase in the end panel. The difference in strains between gages 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 gages 3 and 4 indicate very 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 gages 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 18 presents strain data from five gages mounted on the lower chords. All gages indicated increasing strain values as the load was increased on the bridge. As expected, a sharp reduction in the strain rate in all five gages 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. It is seen from Figure 18 that a relatively narrow range of strains was measured throughout the four panels and two girders monitored by the gages -- 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 as reported in reference 4, the strains shown in Figures 17 and 18 may be converted to axial tensile stress in the elements. The inclined web elements, therefore, developed 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 very 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 gages.



Bottom view of girders

Figure 16. Location of electrical resistance strain gages bonded to elements of the bridge superstructure.

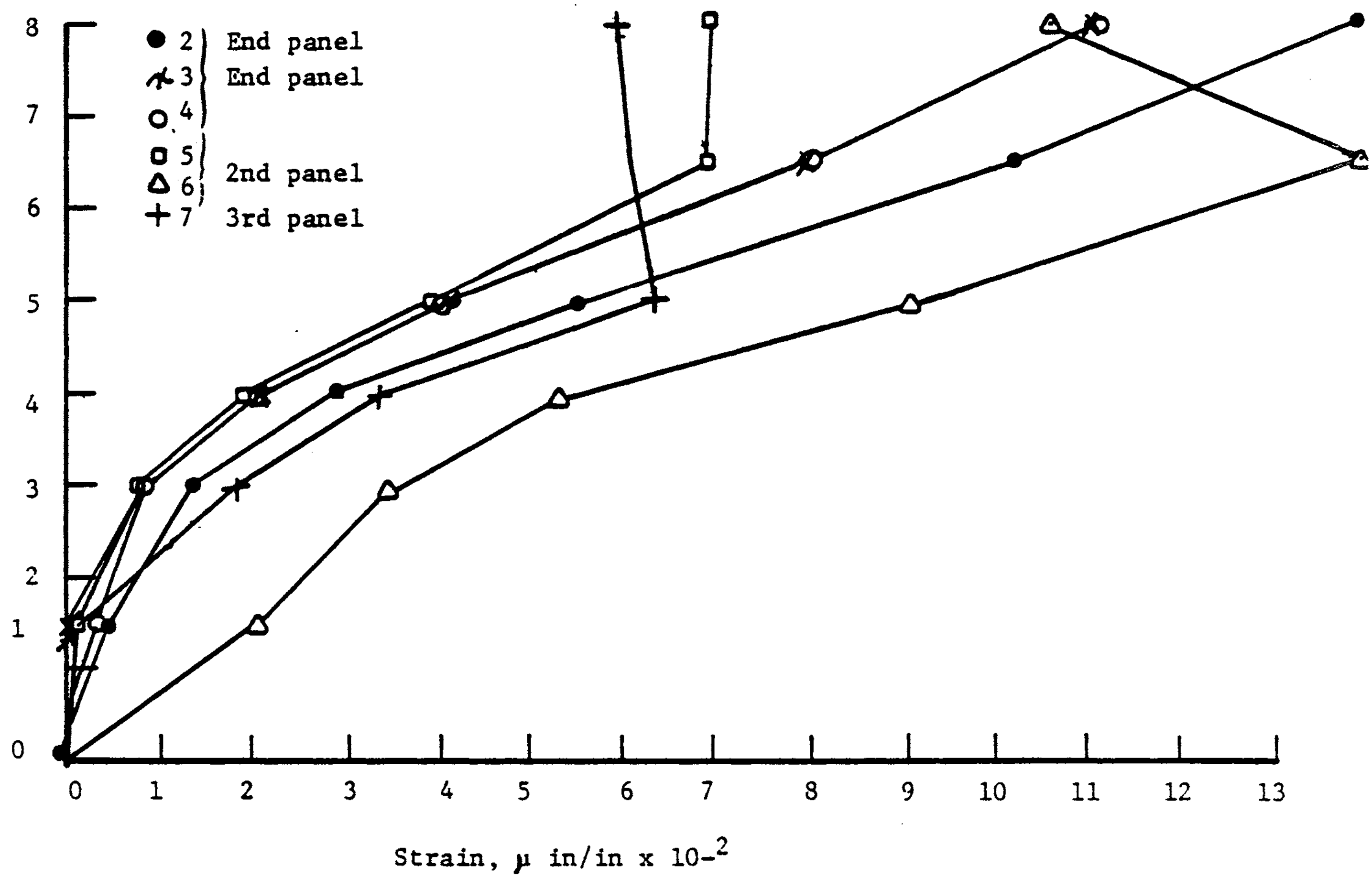


Figure 17. Elastic strain in inclined web elements in the girders due to application of loads.

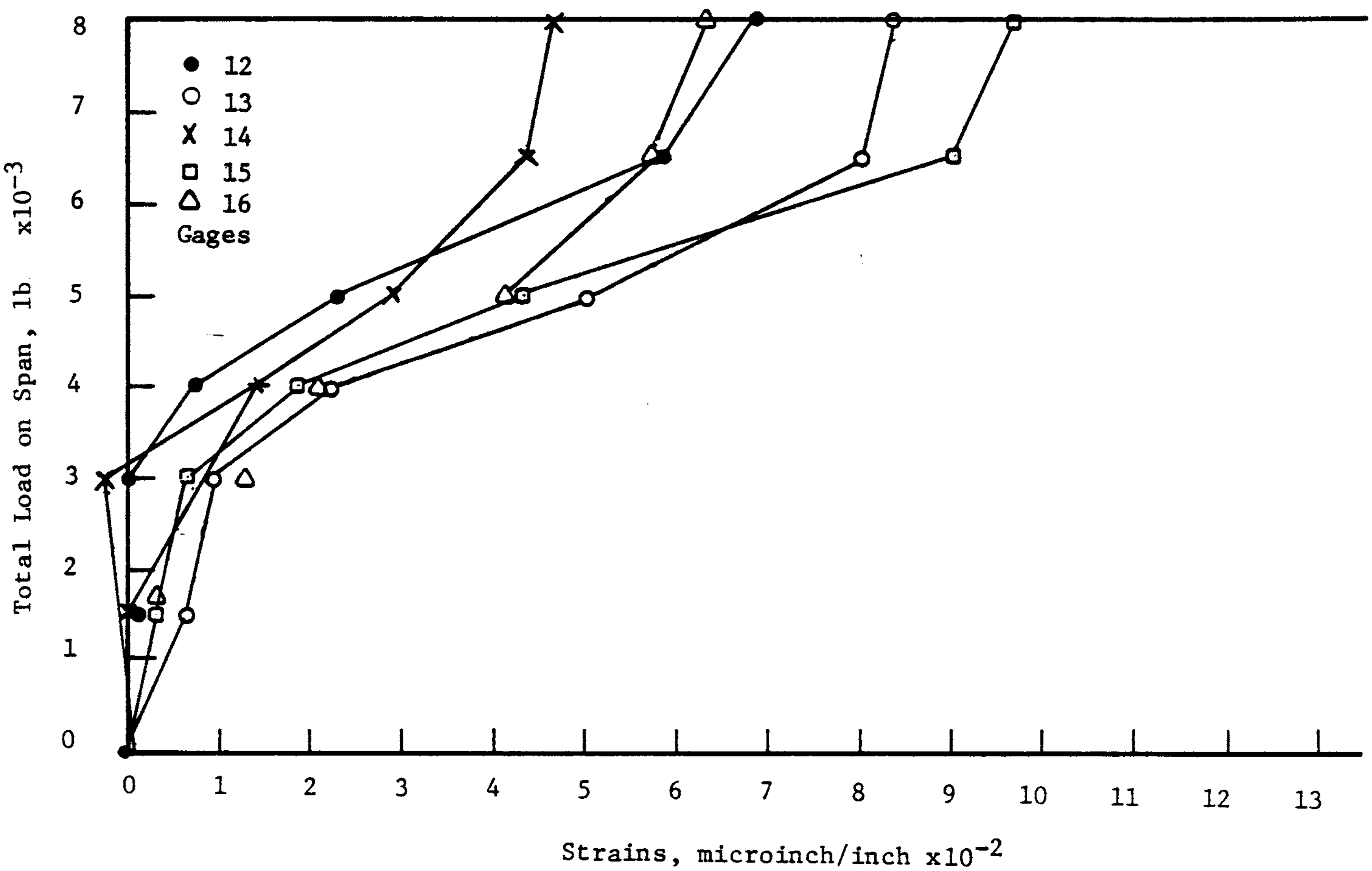


Figure 18. Elastic strain in lower chord elements in the girder due to application of loads.

Creep Strains

Figure 19 presents representative strain data from five active strain gages and a fixed reference circuit over the period 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 19, 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 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 gage 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 15 indicates little to no correlation. Because of the gross deviations of the measurements, the data are worthless for quantitative use. However, it is quite evident that all of the gages underwent the same magnitude of creep strain, whatever it was. If the first 6 days of the creep strain readings might be considered reliable, it appears that no measurable creep occurred in the three lower chord and two web elements shown in Figure 19. The absence of detectable creep strain in the individual elements of the girders corresponds with 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 would account for the absence of creep in the composite material.

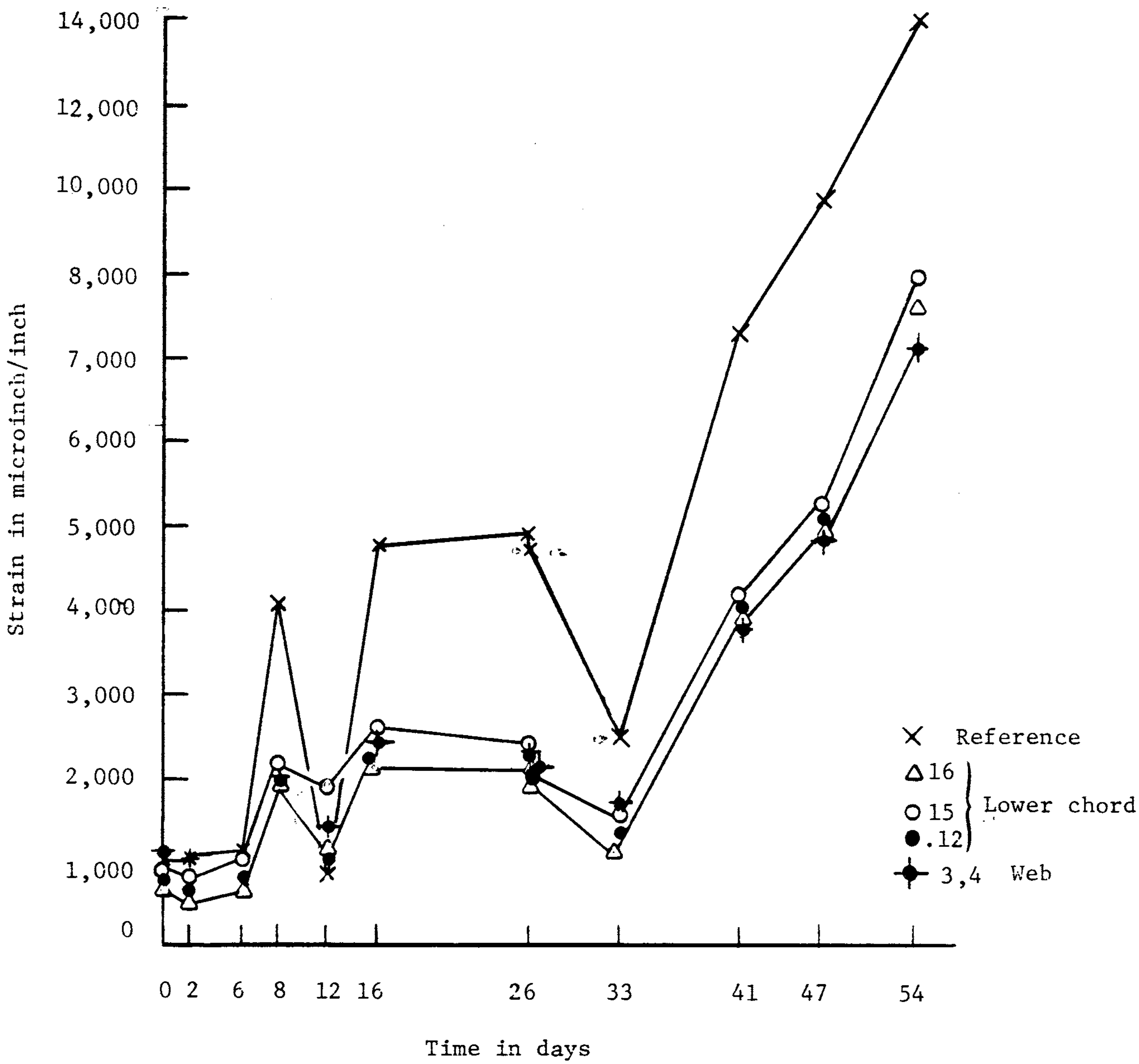


Figure 19. Strain in selected elements vs time with full live load applied to bridge.

REMOVAL AND TRANSPORTATION OF BRIDGE

Unexpected but valuable information was obtained when the superstructure was moved to the test site. On March 11, 1985, personnel from the Parks and Recreation Department of the city of Charlottesville moved the bridge from a storage yard to the erection site in Pen Park. The handling and transporting of the bridge were completed without benefit of procedural instructions from the project director. Being unaware of the fragile nature of the material in the bridge and unfamiliar with special support requirements, the persons moving the structure took few precautions to avoid damaging it. Consequently, a heavy chain was simply wrapped around the bridge longitudinally and attached to a front-end loader for lifting the structure onto a lowboy trailer. Fortunately, the temporary wooden crating attached to the end stiffeners for support during storage also served well as end supports while the bridge was transported approximately 8 miles to the erection site. The bridge was unloaded from the lowboy in the same manner as it had been loaded. During the handling, the outside stiffeners at both ends of the center girder were broken where the lifting chain made contact. The extent of the damage to a stiffener may be seen in Figure 20. 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 as shown in Figure 21.

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 which would not have been conducted intentionally for fear of irreparable damage to the joints and elements.

Upon completion of the creep test, the bridge was removed from the concrete seats and moved again via lowboy to a storage location in Pen Park. The bridge could not be manually lifted from the seats because of the additional weight of the polymer concrete wearing surface. The estimated total weight of the bridge at this time was 1,900 lb. To minimize possible cracking and spalling of the concrete wearing surface, the structure was lifted with nylon cargo straps located at three positions along the deck. A strongback (5 in x 6 in wooden timber) was used to connect the three lifting straps and was also attached with a chain to the bucket of a front-end loader. The deck assembly was reinforced at the locations of the cargo straps with wooden framing positioned laterally across the deck.

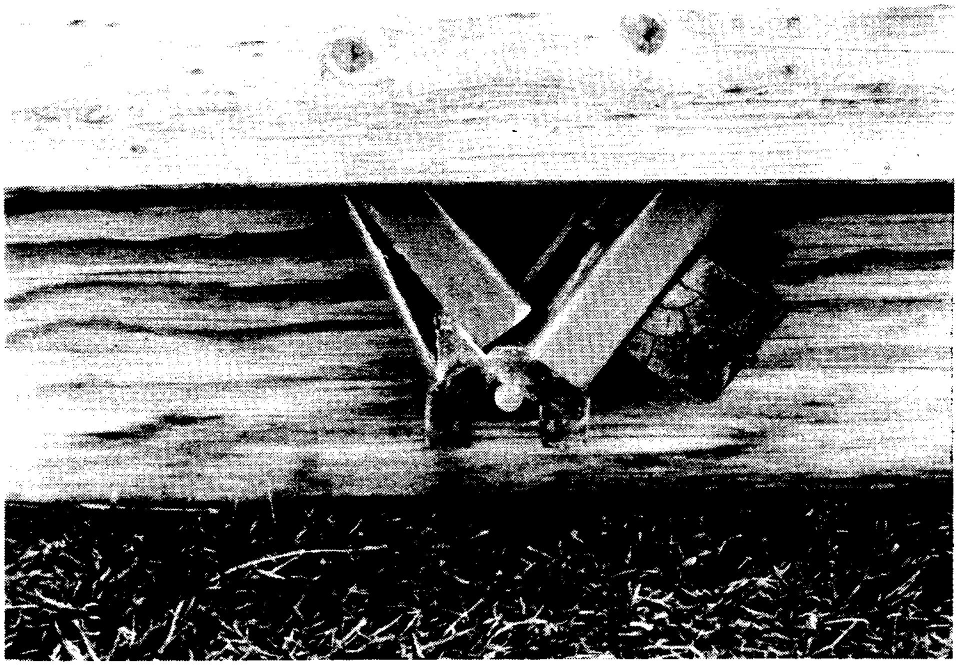


Figure 20. Damaged web stiffener at the end of the center girder.

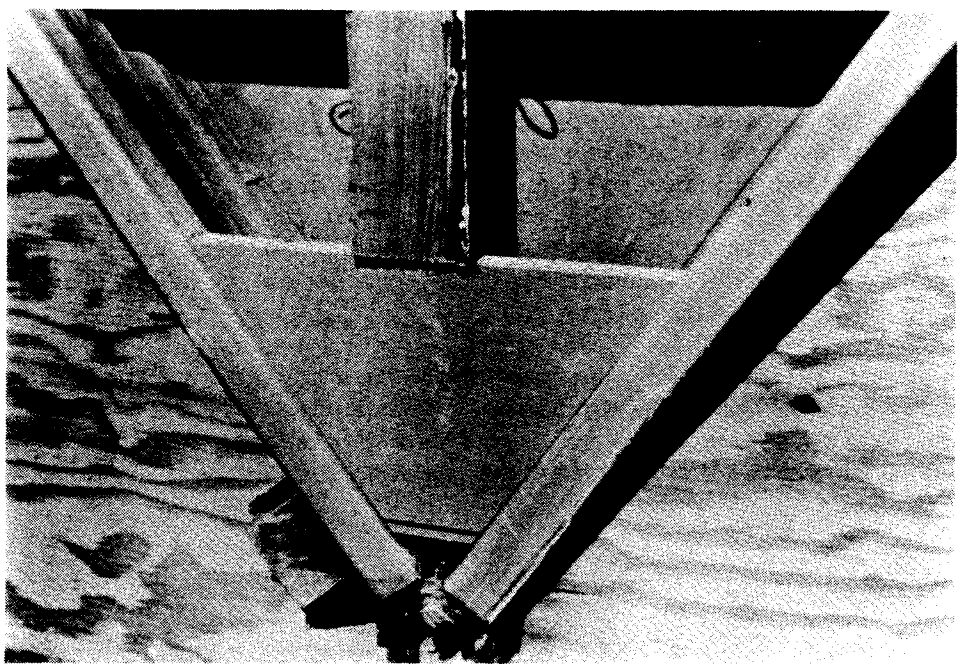


Figure 21. Joint shown in Figure 20 after repair by bonding plate to the stiffeners.

As upward force was applied from the front-end loader to the strongback, it was evident that the ends of the bridge were either wedged into or bonded firmly to the concrete seats. By moving a cargo strap first to one end and then the other end of the bridge and concentrating the lifting force, the supports were loosened from their seats - but not without considerable deformation of the deck and the application of excessive forces to the end stiffeners. The northern end of the bridge was disengaged from the seat without difficulty, but the southern end appeared to defy all efforts of the construction crew to loosen it by preplanned procedures. As frustrations grew, gentle treatment dissolved into action with sledgehammers, long steel prybars, and the application of maximum load capacity from the front-end loader. Just before the bridge broke loose from the concrete seat, the deck was so distorted and the abuse to the fragile pultruded members had become so severe that the project director, distracted and alarmed for the safety of the structure, failed to photograph the extreme measures used. However, a number of photographs were made of the handling of the bridge and several are included in Figures 22 through 24 to document the procedure.

Upon removal of the superstructure from the concrete seats, it was determined that the northern end of the bridge had resisted removal because of the wedge action of the bearing pads on the support elements. However, the southern end had been bonded to the concrete abutments by small quantities of polyester resin which had drained from the end of the deck plate during the application of the polymer concrete wearing course. Figure 25 shows the principal area on the surface of the concrete backwall over which the polyester resin drained. Apparently the clearance between the top transverse stiffener and the backwall of the abutment was small enough to trap a quantity of the resin, which cured and bonded the two mating surfaces securely. The forces and efforts required to break the polyester from the abutment attest to the strength of the binder, and they explain why the deck assembly could be drastically distorted without spalling the layer of polymer concrete from the pultruded deck plate.

Figure 26 shows the superstructure in storage in Pen Park with temporary wooden supports in place. The wooden framing material seen in the photograph was used to protect the web elements and to maintain alignment of the end stiffeners during handling.

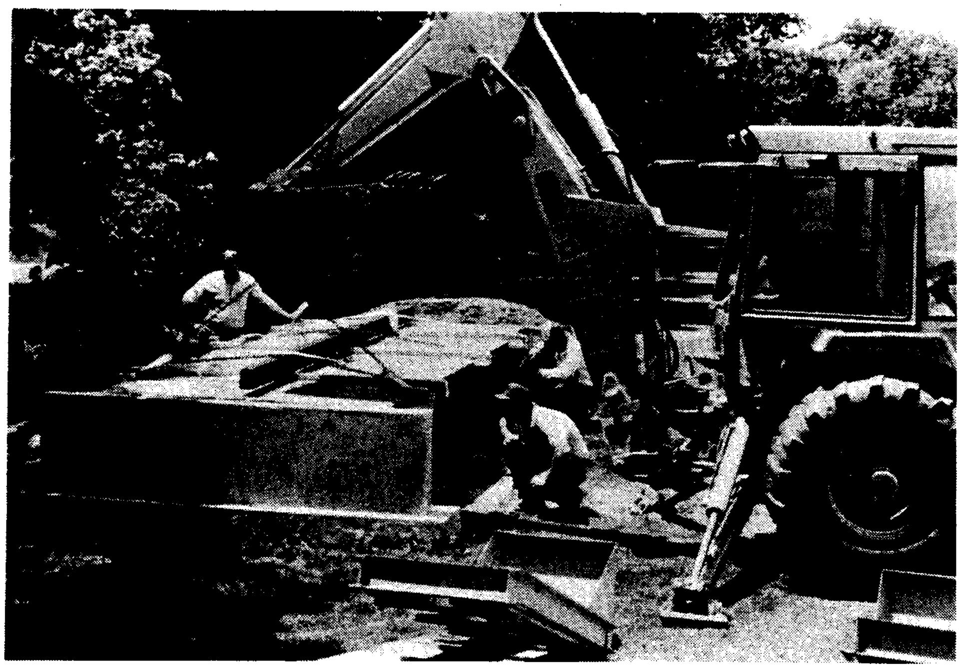


Figure 22. Loosening the northern end of the bridge from the concrete seat with power equipment.

Figure 23. Positioning lowboy trailer beneath suspended superstructure.

Figure 24. Bridge in place on trailer with temporary wooden supports under the end stiffeners.

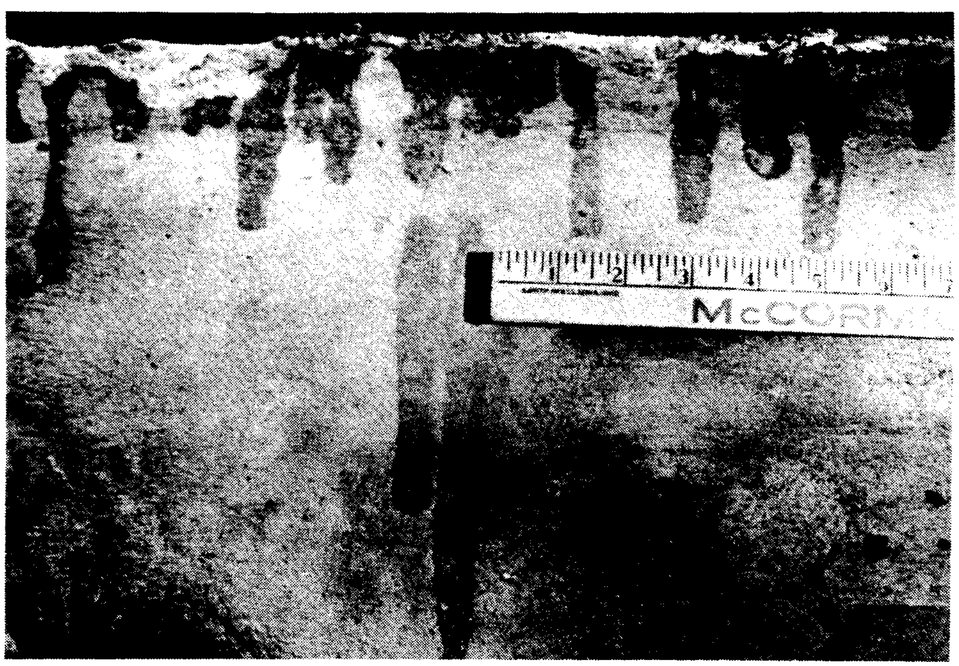


Figure 25. Polyester resin on the surface of the concrete abutment at the southern end of the bridge.

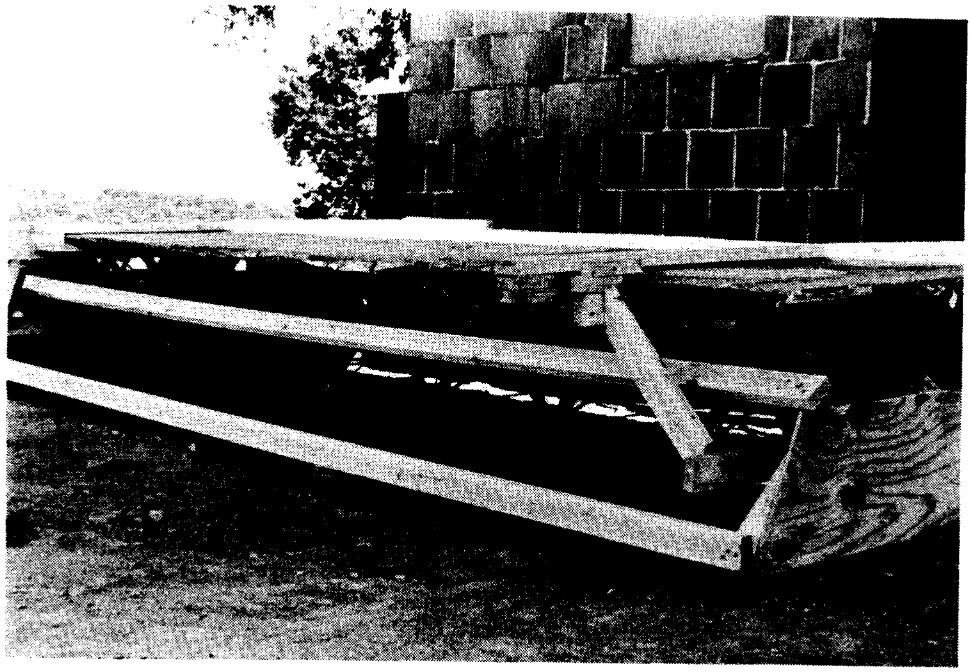


Figure 26. Crated superstructure in storage area in Pen Park.

A careful visual inspection of the bridge following the removal and transportation from the irrigation pond site provided the following information.

1. One of the outside stiffeners on the southern end of an exterior girder was cracked. This stiffener had been used as a contact point to pry the bridge from its support. The stiffener was not displaced from the lower chord connector, and it is not believed that the cracked member will adversely affect the performance of the bridge in the future.

2. The lower chord connector of the center girder, which was also used as a pry point, was abraded but not otherwise damaged.

3. None of the other stiffeners nor connectors were damaged.

4. A crack approximately 6 in long was observed on the inside of the center girder at the end joint between the flange plate and the transverse stiffener. The crack extended from one end of the stiffener inward. It is probable that this crack formed when the deck was distorted during removal of the southern end

of the bridge from the seat. It is unlikely that the crack will propogate through the joint unless the flange of the girder is grossly distorted again.

5. No signs of separation were observed between the flanges and cover plates.

6. The polymer concrete wearing surface appeared intact throughout. Soundings with a small hammer over the deck revealed no obvious nor suspected regions of delamination.

STRUCTURAL CONSIDERATIONS OF THE POLYMER CONCRETE WEARING COURSE

As noted previously, the bridge was designed for a live load of 85 lbf/ft^2 with a normal weight concrete slab serving as the wearing course. The 4 in thick slab was also considered to act as the upper flange for the girders and to provide the majority of the compression area to resist bending stresses from the live load. The compressive elastic modulus of the concrete was assumed to be 3×10^6 lbf/in^2 and the modulus of the GRP flange plates was 2.3×10^6 lbf/in^2. The effective modulus of the composite flange was 2.9×10^6 lbf/in^2 as computed in accordance with the Rule of Mixtures and used in the analytical studies. Therefore, the substitution of a slab of polymer concrete 1/2 in thick with a different compressive modulus altered the structural role of the concrete slab considerably.

Compressive tests conducted on 4-in x 8-in cylinders of the polymer concrete produced the values shown in Table 1 as a function of the temperature of the concrete. A similar range of fluctuations of the compressive modulus of the GRP plates as listed by the material supplier are shown in Table 2.

Using the given values for the moduli at 75°F and the Rule of Mixtures, the effective modulus for the girder and concrete overlay was computed to be 2.1×10^6 lbf/in^2. At 125°F the effective modulus would be 1.2×10^6 lbf/in^2. Because of the varying, nonuniform temperature through the thickness of the deck materials, an effective compressive modulus may range from 1.2 to 2.1×10^6 lbf/in^2 for a deck with a combined thickness of 1.25 in.

Table 1

Variation of Compressive Modulus of Polymer Concrete with Temperature

Temperature, °F	Compressive Modulus $\times 10^6$ lbf/in^2
0	3.3
35	2.3
73	1.7
100	0.5
122	0.3

Table 2

Variation of Compressive Modulus of Pultruded Composite with Temperature

Temperature, °F	Compressive Modulus, $\times 10^6$ lbf/in^2
75	2.3
124	1.8
175	1.4
200	1.2

Figure 27 presents the results of an analytical solution for the deflection of the deck surface and the lower chord at the center point of the bridge with full live load applied and with various values for the effective compressive modulus for the deck and girder flanges. The measured values for the same deflections are plotted in the figure for comparison. From this comparison, it is apparent that the analytical model used to compute the deflections in the structure provides a conservative estimate at the test temperature of 75°F. The difference between the measured and computed deflections diminishes as the test temperature increases. While the difference between the measured value of 0.44 in and the analytical value of 0.25 in appears large, it should be noted that the analytical model does not include known secondary conditions which contributed to the actual deflection of the structure. These variables include bending in the two-force elements, joint and general geometric distortions, deck-plate warping, shear deflections in all of the elements and other synergistic effects between elements and joints. The relative contributions of these secondary conditions to the overall measured deflections are unknown. Certainly, some of the variables cited are more significant than others. It is, therefore, difficult to include a meaningful modifying value in the analytical solution for each affected component to achieve better agreement between the measured and computed values for deflections. Further testing of the structure under conditions more carefully controlled than those for the recently completed field load test should provide some estimate as to which of the variables are significant and to what degree.

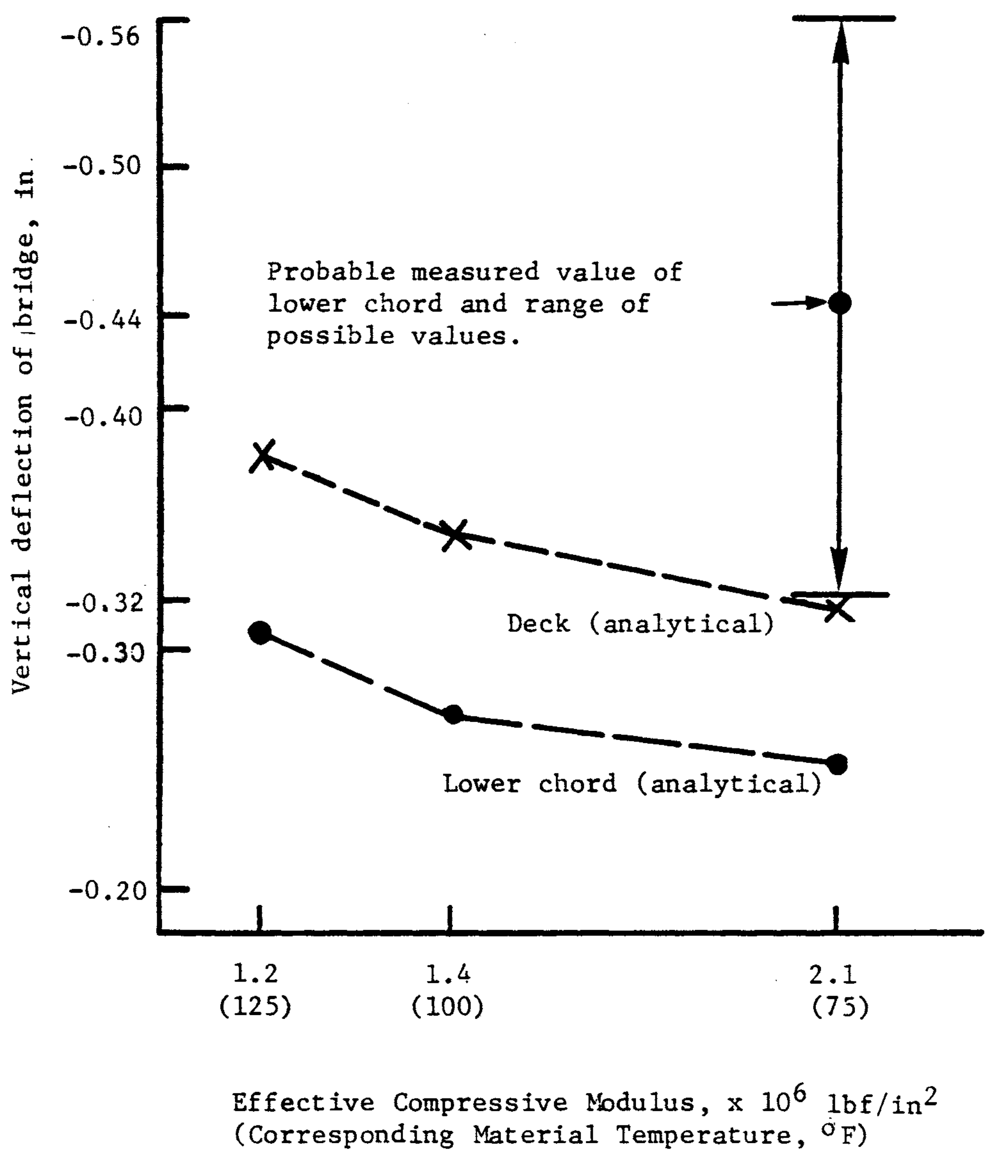


Figure 27. Elastic deflection of center point of bridge at deck and lower chord elements as a function of temperature and material modulus variation for a live load of 8,000 lb.

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. While 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. Manual handling of the superstructure prior to placing the wearing course was adequate for successful erection on the seats. The use of nylon cargo straps to lift the bridge with the wearing course is considered desirable to prevent abrasion and concentrated forces at points of contact with the GRP material.

4. 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.

5. The influence of temperature changes was reversible but pronounced upon 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.

6. Load transfer laterally between girders was adequate and the interaction between girders appeared satisfactory for the live loads investigated.

7. Strains appeared to be reasonably uniform among 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.

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

9. A residual deflection of 0.10 in measured upon removal of the live load appeared to be concentrated in the bearing pads at the supports. A significant portion of the residual deflection was recovered within 48 hours.

10. The ratio of live to dead load for the creep test was over 3/1 and for the elastic test was 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%, the deflection of the prototype was less than three times that prescribed by AASHTO specifications.

11. Measured deflections exceeded the calculated elastic deflection by an estimated factor of 1.8. This was not considered inappropriate in view of the simplifying assumptions included in the analytical model.

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