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INTERIM REPORT

FIELD STUDY OF A GLASS-REINFORCED PLASTIC PEDESTRIAN BRIDGE

by

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(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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SUMMARY

Discussed are the design, fabrication, and load testing of girders composed entirely of glass-reinforced polyester (GRP) resin. The girders were 4.9 m (16 ft.) long and had geometric features which included trussed webs, a solid flange plate, and a triangular-shaped cross section. Three of the girders were attached laterally by a GRP cover plate in the laboratory to provide a complete superstructure for a pedestrian bridge 4.9 m (16 ft.) long by 2.1 m (7 ft.) wide. The lightweight, high strength, and formability of the GRP materials permitted fabrication and handling of both the components and the completely assembled structure without the use of heavy equipment. The weight of the GRP bridge superstructure was approximately 364 kg (800 lb.). An exposed aggregate concrete deck and wooden handrails will be used for the bridge to blend with the natural setting of the recreational area site for the bridge.

Outdoor live load tests were performed to measure strains, deflections, and the creep behavior of one full-scale girder exposed to prevailing climatic conditions in Charlottesville. A finite element analysis was conducted for the single girder to provide comparisons between experimental load test data and analytical predictions. The experimental results were approximately 25% lower than the predicted values for deflections. Creep test results from a laboratory specimen indicate mid-span deflections to be less than 0.88 mm/year (0.03 in./year) under a live load greater than the AASHTO design value of 4,063 N/m^2 (85 psf).

Observations of the bridge are planned for a period of five years to monitor its structural behavior, the effects of weathering, and user abuse.

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INTRODUCTION

This study resulted from laboratory investigations of the performance of a glass-reinforced plastic girder. Development of the girder configuration evolved over the period September 1971 to June 1976 and is described in several publications issued in recent years.[1,2,3,4] Specific recommendations were made to pursue a study of a full-scale, in-service bridge with investigations to include variables of force, temperature, moisture, sunlight and abuser use.[2] Approval for the project was obtained under the classification of an "Experimental Feature" in the construction of a recreational rest area located along Interstate Highway 66 approximately 5 km (3 miles) northwest of Front Royal, Virginia. The project was sponsored jointly by the Virginia Department of Highways and Transportation and the Federal Highway Administration, with administrative oversight being provided by the Virginia Highway and Transportation Research Council.

Functional and esthetic considerations of the site dictated a pedestrian bridge 4.9 m long by 2.1 m wide (16 x 7 feet), which would span a small creek and fit into the natural appearance of a wooded area. The esthetic requirements were satisfied by using exposed aggregate concrete as the bridge deck to match adjacent gravel paths, timber handrails, and green colored glass-reinforced plastic (GRP) plates over the trusses to blend with the trees and summer flora.

The project was divided into two phases. Phase I was devoted to the fabrication and load testing of a single full-scaled girder. Phase II dealt with the fabrication, erection, load testing and related matters of the prototype bridge structure. The entire bridge was fabricated and completely assembled in the Composite Materials Laboratory at the University of Virginia. Precast concrete seats for the bridge were also cast and shipped with the structure to insure an accurate fit and to minimize erection time of the superstructure at the site. The erection of the bridge was scheduled for late summer 1977. This paper, therefore, reports only the work completed in Phase I of the project and a brief description of the prototype structure.

OBJECTIVES

Specific objectives of Phase I were:

1. To perfect design and fabrication procedures for a girder, which was dimensionally scaled up from previous specimens.
2. To compare analytically predicted deflections with actual deflections which occurred when the concrete slab was cast on the top plate of the girder.
3. To observe the behavior of the composite girder during a live load test of 4062.1 N/m^2 (85 psf).
4. To observe the behavior of the composite girder under static load for an extended period of time.

Specific objectives of Phase II were:

1. To design, fabricate, assemble and instrument the complete bridge structure.
2. To establish procedures for expeditious field erection of the superstructure, placement of concrete deck, and installation of handrails.
3. To observe the behavior of the bridge under a live load test equivalent to the design load of 4062.1 N/m^2 (85 psf).
4. To monitor the bridge for a period of five years to determine the effects of materials creep, weathering and user abuse.

PHASE I — PRELIMINARY TEST GIRDER

Design

The preliminary test girder was designated TTG-13 (triangular-trussed girder specimen number 13) and was basically a scaled-up version of TTG-12, which is described in Reference 1.

Figure 1* is a view of TTG-12 identifying principal features of the girder. The overall length of TTG-13 was 4.9 m (16 ft.), the width of the top cover plate was 71.1 cm (28 in.) and the depth from the top plate to the lower chord was 40.6 cm (16 in.). The depth was considered to be relatively shallow compared to the width, but was adopted to satisfy anticipated site conditions.

A criterion of 1.3 cm (0.5 in.) deflection at midspan under a uniformly distributed live load of 4062.1 N/m^2 (85 psf) was used for designing the fiberglass elements. The concrete deck slab was assumed to act as a portion of the top flange of the girder in meeting the criterion for live load. Because of the shallow depth of the girder and the deflection limitation, the predicted design stresses in the tension elements were low relative to a potential working stress of 172.2 M Pa (25,000 psi). The maximum computed stress was 36.5 M Pa (5,300 psi) and occurred in a lower chord element. The final sizes of the bottom chord and web diagonals were determined by means of a computer program based on the finite element analysis described in Appendix A. The total number of strands of glass roving used in each element is shown in Figure 2. A description of the glass, resin and other materials used is given in Appendix A.

Fabrication

Specimen TTG-13 was fabricated in essentially the same way as previous girders had been fabricated: cutting and fitting of pultruded rods, tubes and lower chord connectors; bonding the top plate and stiffeners; mounting the top plate on the mandrel and attaching vertical stiffeners and lower chord connectors; and finally, winding and curing the tension elements. Figure 3 shows the specimen mounted on the mandrel just prior to winding and Figure 4 shows the completed girder in the curing oven. Both views are of the girder in an inverted position.

No metallic fasteners were used in any of the connections. Resin-impregnated wooden bolts were used along with an epoxy adhesive to attach the top plate to the transverse stiffeners. Pultruded fiberglass rods .9 cm (3/8-in.) in diameter were used to fasten the ends of the vertical stiffeners to the transverse stiffeners, and .6 cm (1/4-in.) diameter rods were used to fasten the stiffeners and lower chord connectors.

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*Figures follow Appendix.

Two strands (6,120 fibers per strand) of glass roving were simultaneously impregnated with resin and used to build up the tension elements by continuously winding the strands manually around anchor points formed by the transverse stiffeners and the lower chords connectors. Winding paths and the number of strands used in the development of the tension elements are included in Figure 2. Cross diagonals were included in all of the panels in TTG-13 to satisfy anticipated shear force requirements generated by partial live loading on the top plate. However, with the subsequent application of the concrete deck, it was determined that the design live load did not impose a tensile stress upon the cross diagonals, so they were not included in the prototype structure. A cover plate .6 cm (1/4-in.) thick was bonded to the top plate of the girder prior to placing the concrete deck.

The manpower requirement for the fabrication of the 4.9 m (16-ft. long specimens was limited to one person for all cutting and assembly operations, except on occasion when the cumbersome mandrel or top plate had to be moved. Winding the tension elements was most efficiently accomplished by the use of three persons, one of whom rotated the mandrel and kept count of the number of strands as they were placed. Approximately six man-hours were required to "wind" TTG-13.

Instrumentation

Six electrical resistance strain gages were bonded to selected elements in the specimen. The gages were Type EA-06-250-BF-350, supplied by the Micro-Measurements Company, and were bonded with M-bond 200 adhesive. Deflections at several panel points during load tests were measured by mechanical dial indicators with least readings of 0.025 mm (.001 in.). Data from these instruments will be presented and discussed later.

Concrete Deck Placement

The test specimen was supported at both ends by wooden frames built to fit the V shape formed by the inclined stiffeners. No support was applied directly to the lower-chord stiffener connector. A wooden side form for the 7.6-cm (3-in.) thick concrete slab was placed adjacent to the outside edges of the top plate and supported along its length so that the member could deflect independently from the form as the weight of the concrete was applied to the girder. Consequently, the finished depth of the concrete varied from 9.8 cm (3-7/8 in.) at the center to 8.3 cm (3-1/4 in.) at the ends of the girder. Just prior to placing the concrete, the fiberglass plate was sanded, cleaned with an acetate solvent, and coated with an epoxy adhesive which was especially formulated to bond fresh concrete to solid materials. With a unit weight of 2,224 Kg/m^3 (139 pcf), the concrete slab weighed approximately 6,660 N (1,500 lb.). Cylinder tests of the concrete indicated a compressive strength of 36.5 M Pa (5,300 psi) and a compressive modulus of elasticity of approximately 24,804 M Pa (3,600,000 psi) after moist curing for 32 days. Using the rule of mixtures for the concrete and plate, an equivalent modulus for the top flange of the girder was computed as 23,426 M Pa (3,400,000 psi). This value was used in the analytical determination of live load deflections and stresses. Figure 5 shows placement of the concrete slab on the girder.

Live Load Tests

A uniformly distributed live load of 3680.6 N/m^2 (77 psf) was applied to the girder by placing steel barrels on the concrete slab and filling them with water. Figure 6 shows this test in progress. The load was removed from the girder as soon as strain and deflection measurements were recorded. No signs of distress in the girder were noted during or after the test. Figure 7 shows a comparison of the measured and computed deflections at various locations on the girder. The experimental data indicate a nearly linear relationship between load and deflection. These curves also indicate that the actual deflections were approximately 70% of the computed values.

Subsequently, the barrels were filled with sand to apply an equivalent uniform live load of 4302 N/m^2 (90 psf) to the slab. These loads were applied progressively from one end of the girder to observe the behavior of the cross diagonals throughout the member. Computations had predicted that the cross diagonals would go into tension under a live load of 4062 N/m^2 (85 psf) applied to one-half of the girder length. However, the partial load caused no shear reversal in the unloaded panels so the cross diagonals remained in compression during the test. It was planned to leave this load on the girder for an indefinite period of time to observe possible creep deflections and weathering effects. However, four days after loading, the joint at the lower chord connectors and the stiffeners at both ends of the member failed due to the lateral force exerted on the stiffeners by the "V" supports. Figure 8 shows the displacement of the stiffener and distortion of the joint after failure occurred. While this was considered to be a serious failure of the girder, there was no apparent loss of strength nor stability by the member. Therefore, the load was not removed. Periodic creep deflection measurements and inspections for weathering continued to be made for several months. Figure 8 shows the net deflection of the top surface of the girder at midspan and at the supports over a period of time. The large fluctuations observed at the supports are believed to be due to movements of the underlying material caused by frost heave and subsidence during the winter and summer months. The small net displacements at the centerline of the girder were attributed to thermal effects or measurement variations. There was no indication of progressive displacement of the girder as would be expected from creep mechanisms within the GRP material or from a bond failure at the interface between the concrete slab and the cover plate. Deflection data shown in Figure 9 were obtained with a surveyor's level and rod.

Quantitative weathering measurements (such as weight loss of coupons) were not made. However, it was noted that bleaching of the original greenish tint of the resin occurred gradually with time. Some "blooming" of the fibers (exposure of glass fibers due to erosion of the resin from the surface) was also noted after about six months on exposed surfaces of the pultruded shapes. Fiber blooming increased progressively over the test period but did not appear to influence the serviceability nor the performance of the girder in any way.

PHASE II — PROTOTYPE BRIDGE

Design

The prototype bridge (TTG-WC) was 4.9 m (16 ft.)long and 2.1 m (7 ft.) wide. It consisted of three identical girders, similar to TTG-13, which were connected by bonding a cover plate to each of the top flange plates and by tieing the lower chords together with strands of resin-impregnated glass roving. Each girder was 71.1 cm (28 in.) wide and 45.7 cm (18 in.) deep. Elimination of the cross diagonal elements included in TTG-13 required some changes in the lay-up pattern of the roving in the chord and diagonal elements. The final configuration used is shown in Figure 2 in comparison with that used for TTG-13. Approximately the same amounts of glass were used in the elements for both patterns. An increase in depth to 45.7 cm (18 in.) from the 40.6 cm (16 in.) used for TTG-13 reduced the predicted deflections and stresses from those determined for TTG-13. A comparison of the actual with predicted stress and deflection values will be presented upon completion of the load tests. The design dead load was 2256 N/m^2 (47.2 psf) and the design live load was 4062 N/m^2 (85 psf).

SUMMARY

Fabrication and load testing of the TTG-13 girder provided valuable experience and performance data which were applied to the design and assembly of the prototype pedestrian bridge. Upon completion of the installation and load testing of the full-scale structure, a final report will be submitted describing all aspects of Phase II of the project. It is anticipated that the report will be issued by June 1, 1978, if the construction contract is awarded on schedule.

RECOMMENDATION

The current project provides for visual inspections and deflection measurements of the pedestrian bridge over a period of nearly five years. These observations will be helpful in assessing the in-service behavior of the structure. However, experience to date has indicated that further modifications to the structural concept would result in cost reduction and performance improvements. Several of these modifications are listed as follows:

1. Eliminate the concrete slab as an integral part of the structure to reduce the overall dead weight. A bonded-aggregate wearing surface would suffice for most applications for pedestrian use.

2. Replace the GRP top plate with a cored or other type of section. This would improve rigidity of the flange without increasing the weight. Cored GRP sections are available, but other noncorrosive materials such as aluminum should also be investigated.

3. Utilize fiberous reinforcements with higher moduli than glass for the tensile elements. Polyamide or graphite fibers could be used selectively and sparingly to increase the rigidity of the girder with a minimum increase in material cost.

4. Extend design concepts and fabrication procedures to include continuous girders over intermediate supports. All development work so far has been limited to simply supported members.

REFERENCES

1. McCormick, F. C., "Modification Studies for a Bridge Girder of Reinforced Plastics," Virginia Highway & Transportation Research Council, Report No. 77-R5, July 1976, 29 pp.

2. McCormick, F. C., and H. Alper, "Further Studies of a Trussed-Web Girder Composed of Reinforced Plastics," Virginia Highway & Transportation Research Council, Report 76-R16, November 1975, 78 pp.

3. McCormick, F. C., "Study of a Trussed Girder Composed of a Reinforced Plastic," Virginia Highway & Transportation Research Council, Report 75-R6, August 1974, 40 pp.

4. ________________, "Initial Studies of a Flexural Member Composed of Glass-Fiber Reinforced Polyester Resin," Virginia Highway & Transportation Research Council, July 1973, 28 pp.

APPENDIX A

STRESS ANALYSIS AND DESCRIPTION OF COMPUTER PROGRAM

Axial stresses in the web and lower chord elements and displacements at the joints were computed theoretically by the finite element method utilizing plate bending elements to represent the top plate and space truss elements for the web and lower chord. The relationship between the forces and displacements at the nodes of an element is

$$q_i = \sum_{j=1}^{n} k_{ij} d_j$$

where q_i is a force in direction i; the stiffness coefficient, k_{ij}, is the force that must be applied in direction i to produce a unit deformation in direction j when no other deformations occur in the element; and d_j is the deformation in direction j. Expressed in matrix notation, the relation becomes

$$\{q\} = [k] \{d\}$$

The stiffness matrix [k] of a space truss element, with reference to the global coordinates as shown in Figure A-1, is

$$[k] = \frac{AE}{L} \begin{bmatrix} l^2 & & & & & \\ lm & m^2 & & \text{symmetric} & & \\ ln & mn & n^2 & & & \\ -l^2 & -lm & -ln & l^2 & & \\ -lm & -m^2 & -mn & lm & m^2 & \\ -ln & -mn & -n^2 & ln & mn & n^2 \end{bmatrix}$$

where $l = \dfrac{X_b - X_a}{L}$, $m = \dfrac{Y_b - Y_a}{L}$, $n = \dfrac{Z_b - A_a}{L}$

The stiffness matrix for the rectangular plate bending element shown in Figure A-2 was taken from Przemieniecki.* It was derived using a displacement function that ensures both deflection and slope compatibility on adjacent elements.

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*<u>Theory of Matrix Structural Analysis</u>, McGraw Hill, 1968.

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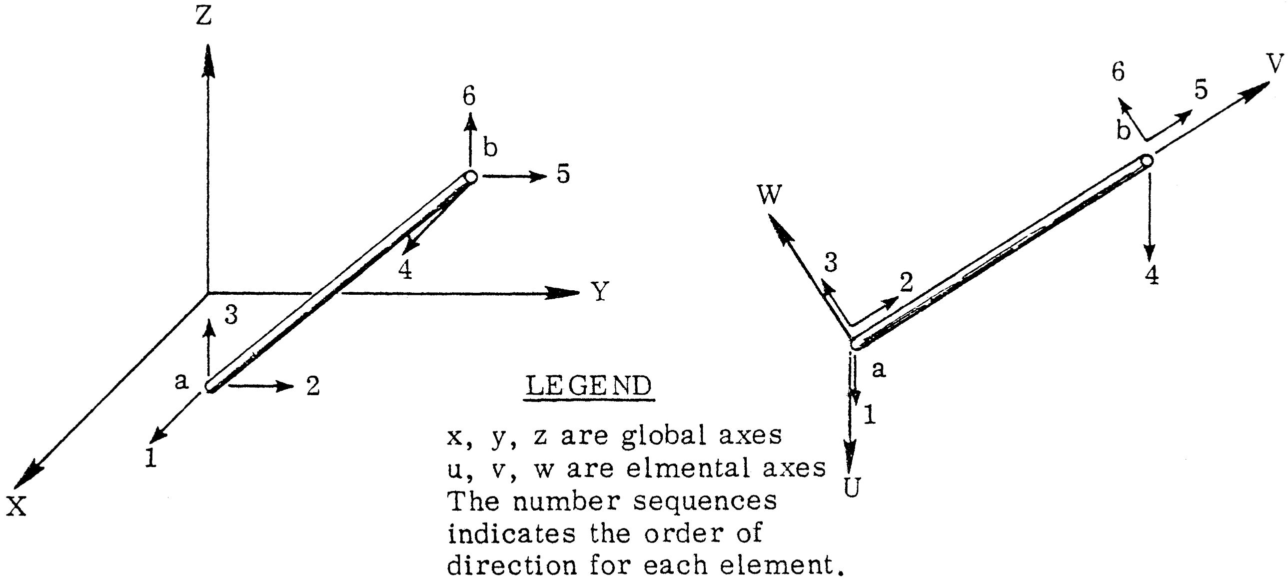
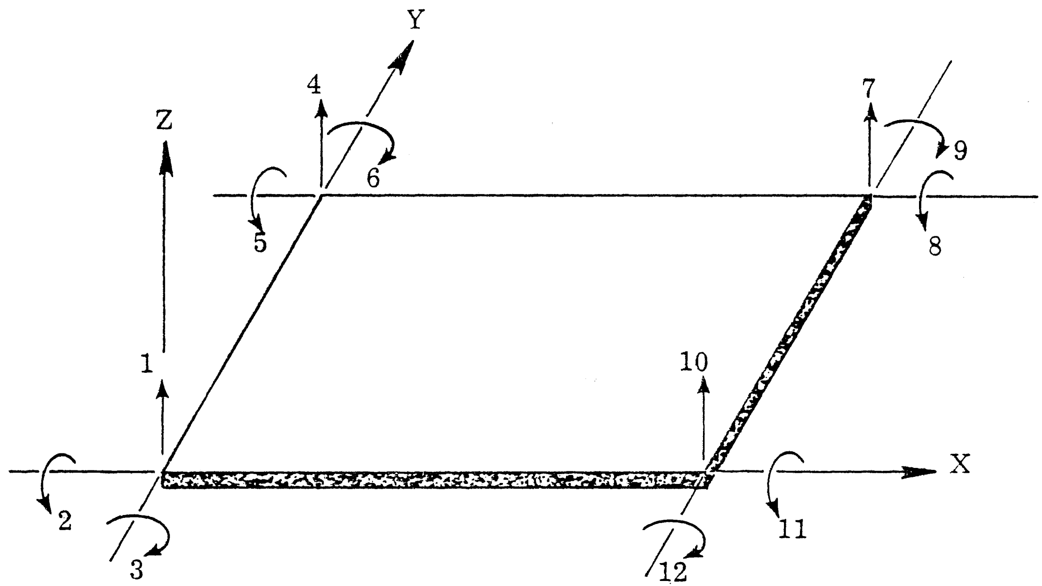


Figure A-1. Global and element axes for space truss element.



LEGEND
x, y, z are plate axes
The number sequence indicates the order
of direction for each element.

Figure A-2. Rectangular plate bending element.

The stiffness matrix [K] for the entire structure (one or more girder sections) is generated by superposition of the matrices of each element. If the external forces and corresponding displacements of the joints in the truss are denoted by {Q} and {D} respectively, the matrix equation for the load-displacement relationship of the entire structure may be shown as

$$\{Q\} = [K] \quad \{D\}$$

Figure A-2 shows schematically the set of directions by which the forces and displacements were defined. Two planes of symmetry permitted use of only one-quarter of the structure in the analyses, and thereby reduced the number of the numerical calculations. For a given load vector {Q}, this set of linear simultaneous equations may be solved for {D}. Thereafter, the deformation vector (d_{xyz}) of each element can be obtained. Subsequent multiplication of $\{d_{xyz}\}$ by $\{k_{xyz}\}$ gives values for $\{q_{xyz}\}$ as desired. The final axial force in each bar is obtained by transforming the xyz components into the element axes (or uvw direction) by

$$\{q\} = [T] \{q\}_{xyz}$$

where [T] is a transformation matrix as defined below:

$$[T] = \begin{bmatrix} [t] & \\ & [t] \end{bmatrix}$$

$$[t] = \begin{bmatrix} \frac{m}{Q} & \frac{-1}{Q} & 0 \\ 1 & m & n \\ \frac{-1n}{Q} & \frac{-mn}{Q} & Q \end{bmatrix}$$

$$Q = \left|\sqrt{1 - n^2}\right|$$

For vertical bars, Q is zero because $n = \pm 1$, and some quantities in [t] become indefinite. Therefore, for vertical bars

$$[t] = \begin{bmatrix} 0 & -1 & 0 \\ 0 & 0 & n \\ -n & 0 & 0 \end{bmatrix}$$

Final values are presented in unit stresses and strains.

MATERIALS USED IN FABRICATION

The following materials were used for the fabrication of the girders.

1. Pultruded square tubes and plates were obtained from Morrison Molded Fiberglass Company, Bristol, Virginia. All materials were grade Extren 500.

2. Glass-fiber reinforcement was obtained from Owens Corning Fiberglass Company, Toledo, Ohio. Type 30, E glass roving was used for winding all tensile elements.

3. Polyester resin, Type E 447, used to impregnate the glass roving was also obtained from Owens Corning Fiberglass Company.

   Small quantities of MEKP were used as the catalyst to provide a gel time of approximately 50 minutes.

4. Bonded joints between pultruded sections and plates with epoxy adhesives furnished by Morrison Molded Fiberglass Company (Kit 502) and H. B. Fuller Company, St. Paul, Minnesota, (Resiweld FE7004).

5. The bonded joint between the cover plates and the concrete slab was made with an epoxy, Sikadur Hi-Mod, obtained from Sika Chemical Corporation, Lyndhurst, New Jersey. This epoxy was a two-component material consisting of 1-1/2 parts epichlorohydrin bisphenol A to 1 part of the reaction product of an aliphatic polyamine and monofunctional epoxide modified with 2.46 tri (dimethylaminomethyl) phenol. The initial necessity for the blended adhesive was specified as 2,000 cps and a tensile strength of 24.1 M Pa (3,500 psi) was to be developed after curing for 14 days.

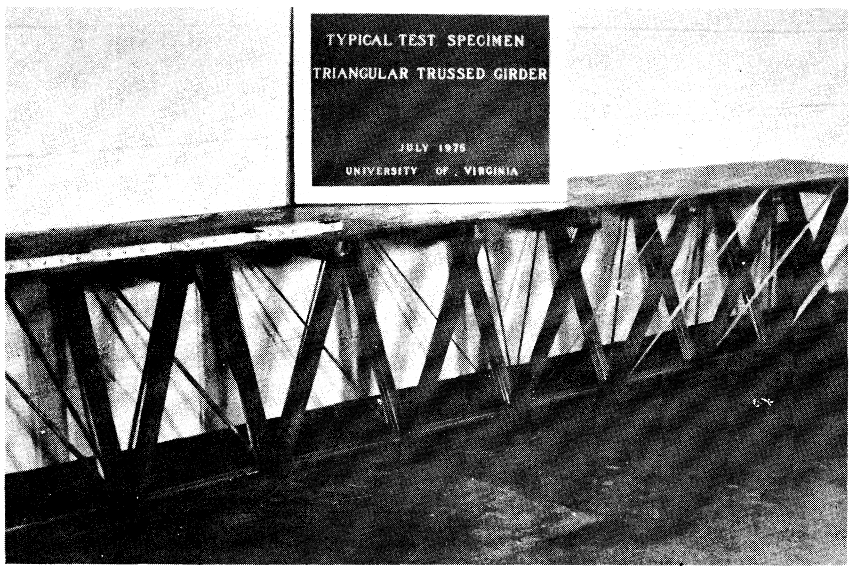
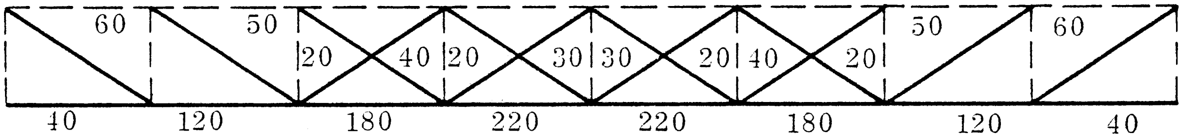
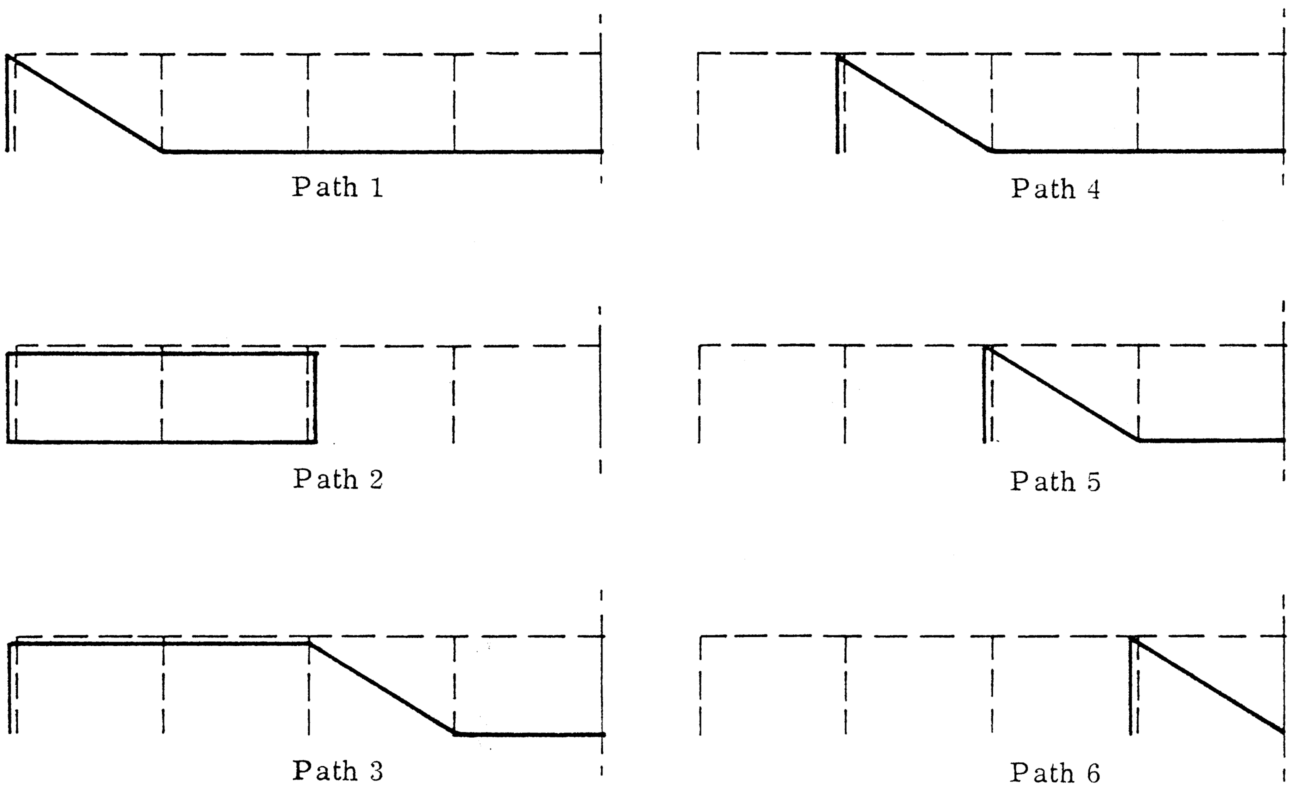


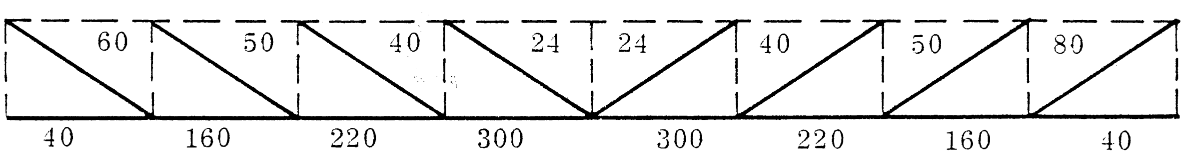
Figure 1. Typical test specimen of a triangular trussed girder showing principal features.

Total tension strands and winding pattern for TTG-13. Dashed lines are stiffeners and plate.)



Winding paths and total tension strands for TTG-WC. (Dashed lines are stiffeners and plate.)





Total tension strands

Figure 2. Comparison of TTG-13 and TTG-WC.

Figure 3. Stiffeners for girder mounted on mandrel prior to winding tension elements.

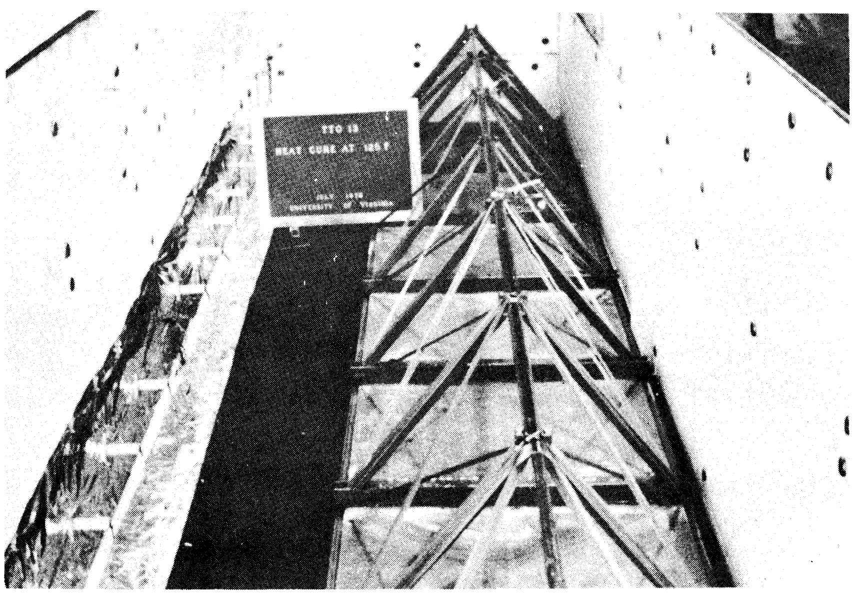


Figure 4. Completed girder in oven for postcuring resin at 54.4°C (130°F).

Figure 5. Placing the concrete deck on top flange plate.

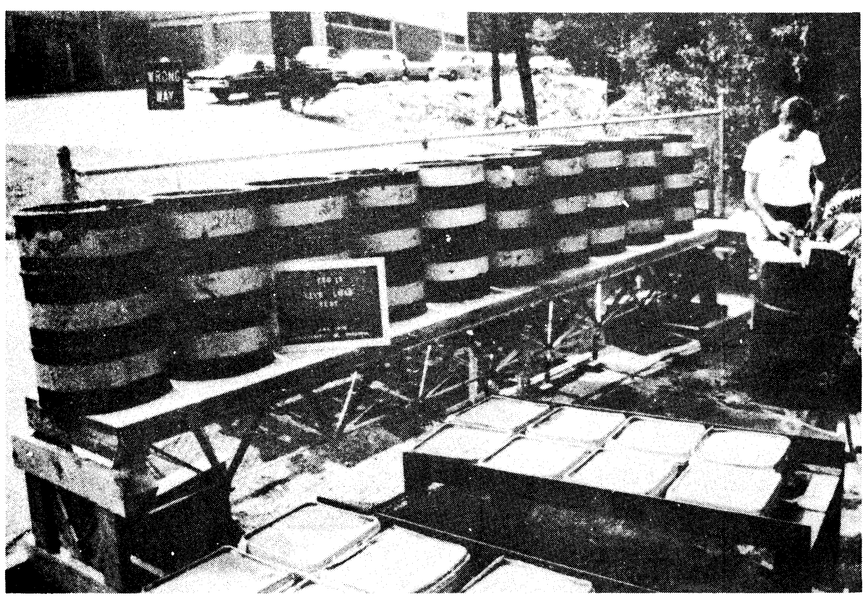


Figure 6. Live load test with water-filled barrels to a maximum of 3680.6 N/m^2 (77 psf).

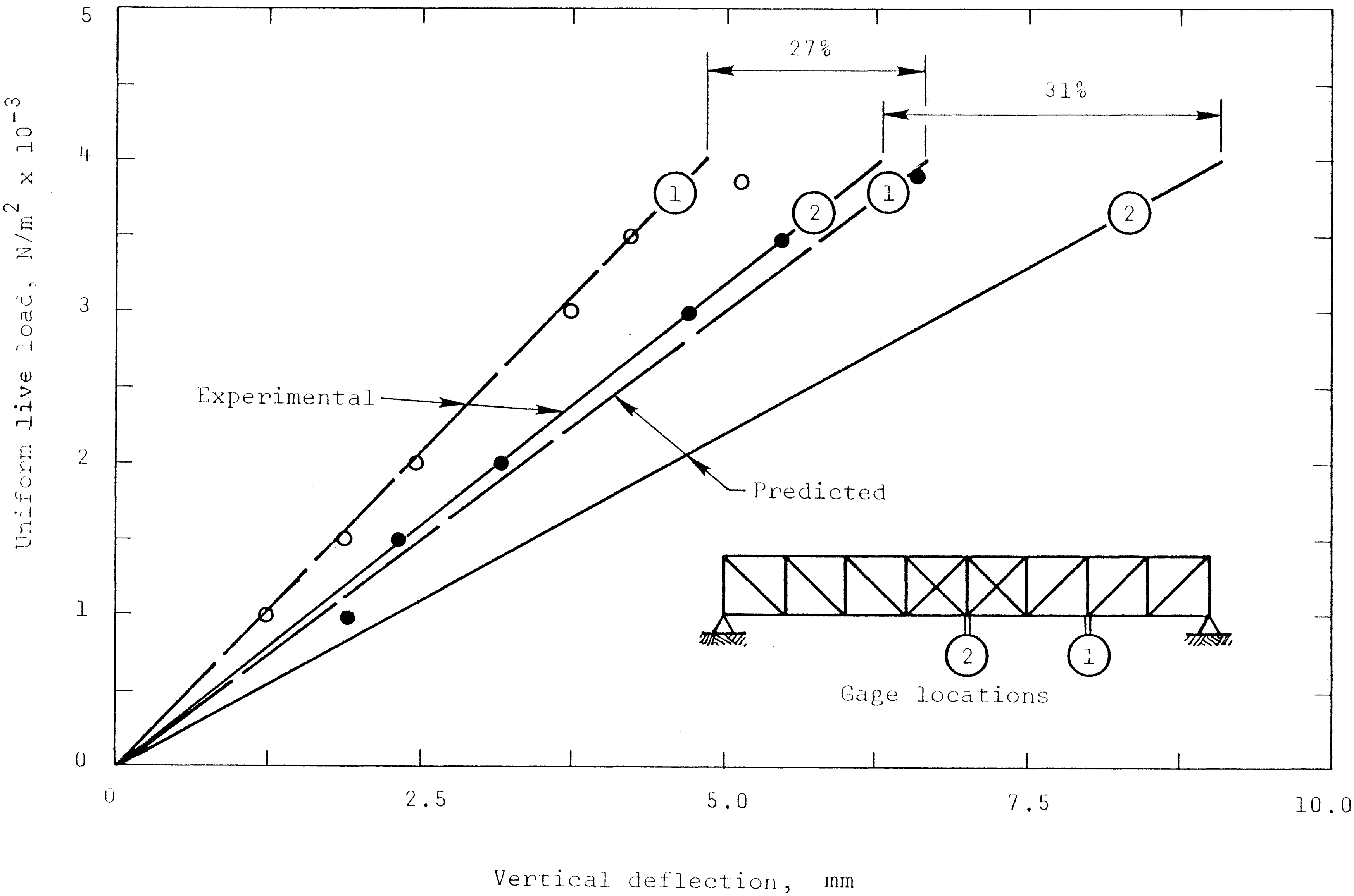


Figure 7. Comparison of theoretical and experimental deflections for live load test of girder. 25.4 mm = 1 inch; 47.8 N/m^2 = 1 psf.

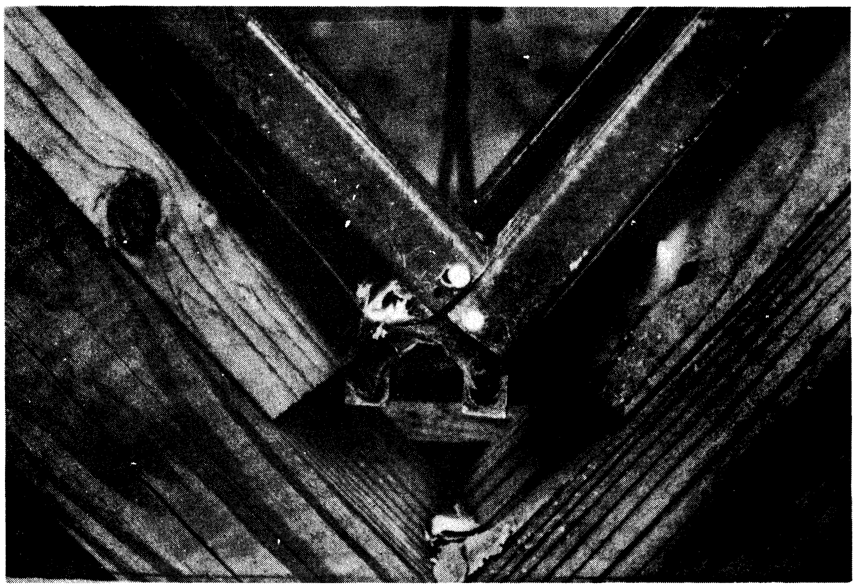


Figure 8. End view of girder showing failure of a joint at a lower chord connector due to seat bearing load.

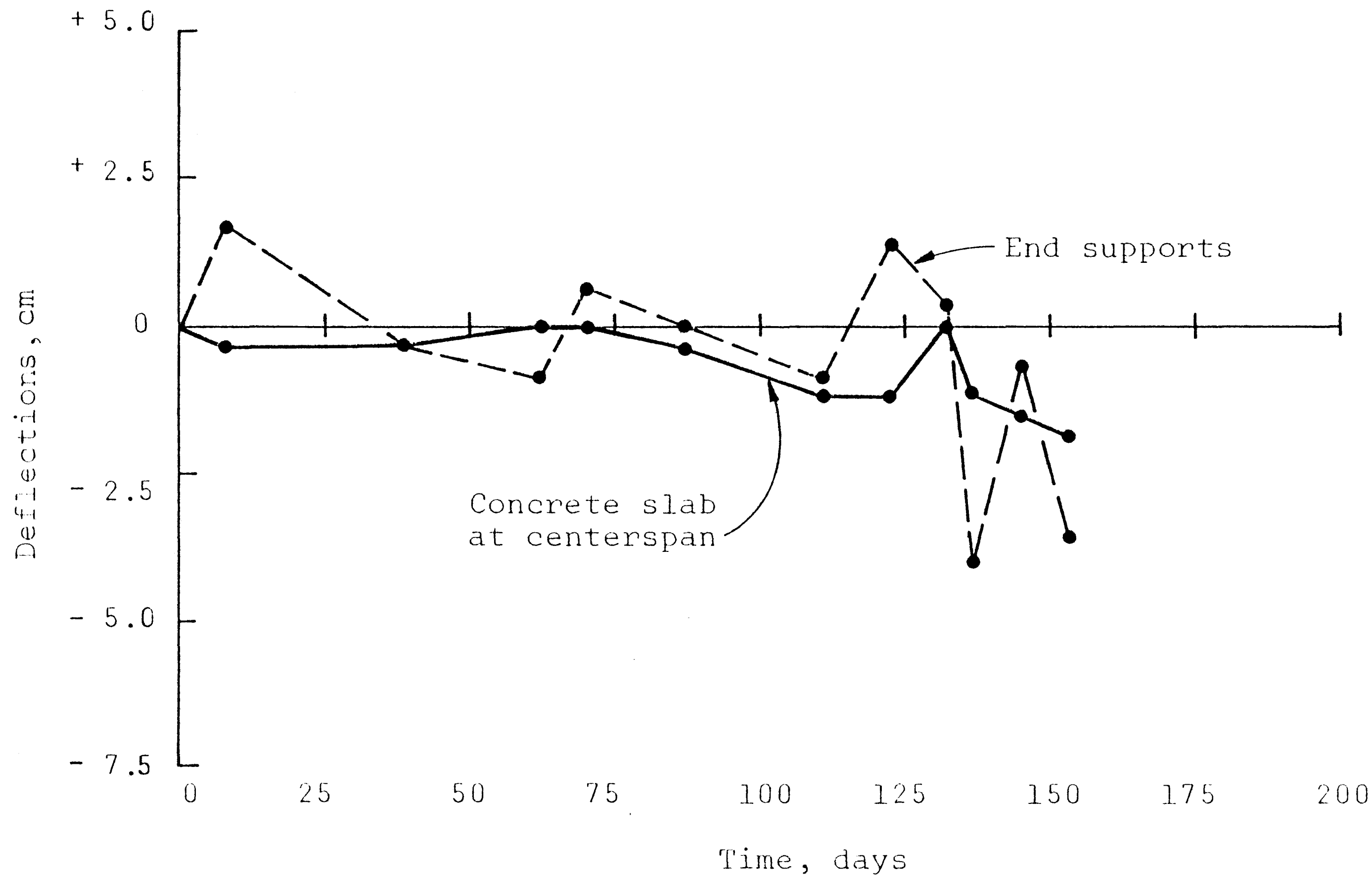


Figure 9. Deflection of the top flange of the girder under a constant load of 4302 N/m^2 (90 psf). 2.54 cm = 1 inch.