LABORATORY AND FIELD STUDIES OF A PEDESTRIAN BRIDGE COMPOSED OF GLASS REINFORCED PLASTIC

Fred C. McCormick, Consultant, Virginia Highway and Transportation Research Council

The design, fabrication, and load-testing procedures are described for girders composed entirely of glass-reinforced polyester (GRP) resin. The girders are 4.9 m (16 ft.) long and have geometric features which include 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 x 2.1 m (16 x 7 ft.). 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 superstructure was approximately 360 kg (800 lb.). Laboratory tests were conducted to ascertain the load transfer characteristics at the GRP plate and concrete deck interface. Live load tests were performed to measure strains, deflections, and the creep behavior of a full-scale girder exposed to prevailing climactic conditions. Comparisons of theoretical and experimental results ranged from nearly zero up to 20% in which the experimental results were usually lower than the predicted values for deflections and strains. The bridge is scheduled for field erection in 1978 and observations will be made for five years to monitor its structural behavior, the effects of weathering, and user abuse.

The structural use of glass fiber-reinforced plastics (GRP) has been evident in the automotive, aircraft, and boat industries for a number of years. Civil engineers in general, and bridge engineers in particular, have been slow in exploring the application of structural plastics. Three principal factors which discouraged a consideration and earlier use of synthetic composite materials were (1) a deficiency of basic data on the properties of the materials and field experience necessary for design, (2) a ready source of available materials in familiar geometric configurations, and (3) a continuing source of the relatively cheaper conventional construction materials. In recent years, these factors have changed to the extent that a consideration of synthetic composites as an alternate construction material has become feasible. This is not to say that all former inhibitions and obstacles to the selection of a composite material for a given structure or bridge have been eliminated. There is still a great deal to learn about both the short- and long-term properties of these materials, but each passing year catalogues additional valuable data gained from in-service structures upon which confidence and design procedures can be established. A number of professional groups from the camps of both the producers and users have issued guidelines and specifications which are useful to designers. Chief among these efforts is the forthcoming ASCE Manual for the Design of Structural Plastics prepared under the direction of the Structural Plastics Council of the American Society of Civil Engineers.

The introduction over a decade ago of GRP shapes similar in geometry to structural steel shapes provided some motivation for designers to consider synthetic composites for structural applications where appropriate. However, the low elastic modulus (approximately 10% that of steel) and flamability characteristics of these products have limited structural applications to services demanding high corrosion resistance and electrical insulation.

Cost considerations for GRP materials were improving rapidly until 1974, when the price of petrochemical products increased drastically. Currently, the rates of rise in costs are approximately equal for both synthetic resins and the conventional materials of construction. The raw material components in popular GRP systems now cost approximately $1.32 per kg ($0.60 per pound) and fabricated structural shapes in the range of $3.30 per kg ($1.50 per pound). Even though these unit costs are higher than those for steel and concrete materials, strength-to-weight ratios, fabrication procedures, lifetime maintenance costs, and other considerations provide a cost-benefit advantage for GRP composites in a growing number of structural applications.

An experimental study of a GRP light-duty bridge is described in this paper. A reduction in cost from that incurred in the use of conventional materials and construction procedures was anticipated from (1) the use of industrialized fabrication techniques, (2) a short erection time, (3) reduced dead weights, and (4) a maintenance-free life of many years. The development of a trussed girder as the principal structural member of the bridge evolved over the period from September 1971 to June 1976 and is described in several publications.(1,2,3,4) Recommendations have been made to pursue a study of a

full-scale, in-service bridge in order to investigate variables of force, temperature, moisture, sunlight, and abuser use.(2) Approval for the project has been obtained under the classification of an "Experimental Feature" in the construction of a recreational rest area located along Interstate 66 approximately 5 km (3 miles) northwest of Front Royal, Virginia.

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

The research project was planned to be carried out in two phases. Phase I, which has been completed, was devoted to the fabrication and load testing of a single, full-scaled girder. Phase II deals with the fabrication, erection, load testing, and related matters of the prototype bridge structure. The entire bridge has been fabricated and completely assembled in the Composite Materials Laboratory at the University of Virginia. Precast concrete seats for the bridge have been cast to ensure an accurate fit and to minimize the time needed to erect the superstructure at the site.

Objectives

Specific objectives of Phase I were:

1. To perfect design and fabrication procedures for a girder dimensionally scaled up from previous specimens.
2. To compare analytically predicted deflections with 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 4,060 Pa (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 are:

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 4,060 Pa (85 psf).
4. To monitor the bridge for a period of five years to determine the effects of materials creep, weathering, and user abuse.

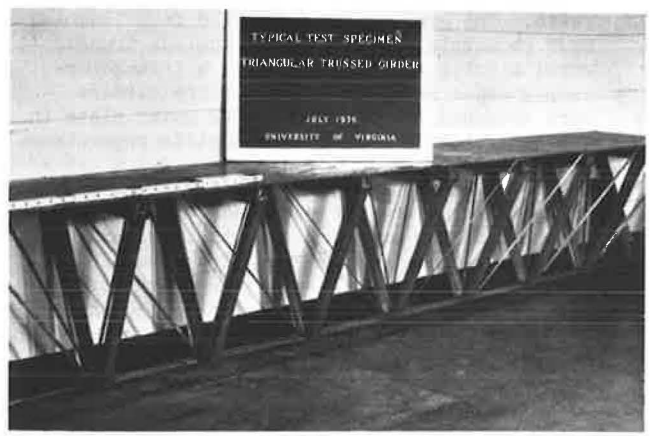
Phase I — Preliminary Test Girder

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 with the principal features of the girder identified. The overall length of TTG-13 was 4.9 m (16 ft.), the width of the top cover plate was 71 cm (28 in.), and the depth from the top plate to the lower chord was 40 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 13-mm (0.5-in.) deflection at midspan under a uniformly distributed live load of 4,060 Pa (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 M Pa (25,000 psi). The maximum computed stress was 36 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 reference 2. The total numbers of strands of glass roving used in the elements are shown in Figure 2.

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



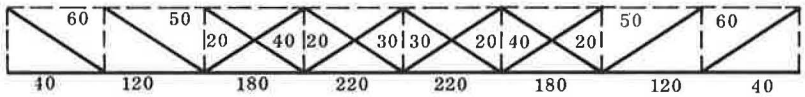
Fabrication

Specimen TTG-13 was fabricated in essentially the same way as previous girders had been fabricated; namely, by cutting and fitting the pultruded rods, tubes, and lower chord connectors; bonding the top plate and stiffeners; mounting the top plate on a mandrel and attaching vertical stiffeners and lower chord connectors; and winding and curing the tension elements. (The pultruding process is used to form structural shapes by simultaneously pulling a number of continuous strands of resin-impregnated glass roving through a heated die which cures the resin within several minutes.) 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 show the girder in an inverted position. A description of the glass, resin, and other materials used is given at the end of the paper.

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 0.9 mm (3/8 in.) in diameter were used to fasten the ends of the vertical stiffeners to the transverse stiffeners, and 0.6 mm (1/4 in.)

Figure 2. Comparison of winding paths and number of strands for TTG-13 and TTG-WC.

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.)

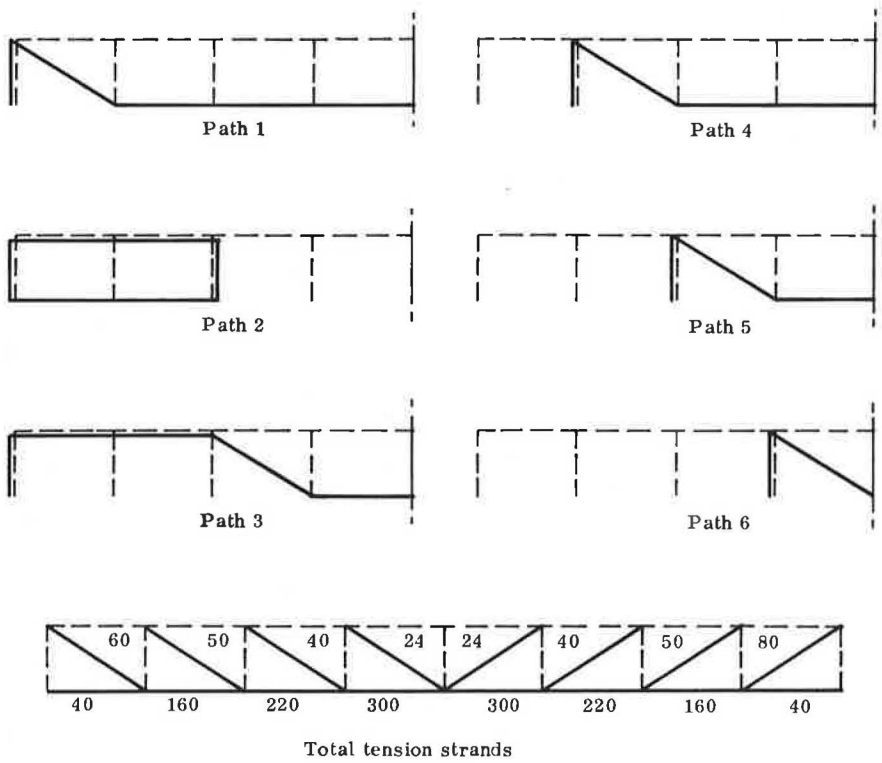
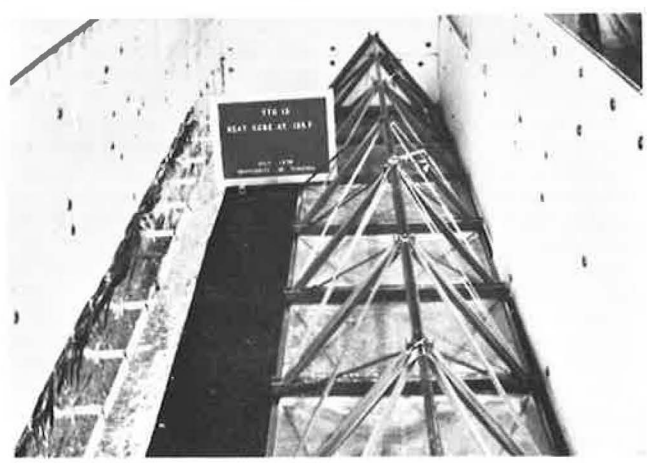


Figure 3. Stiffener and flange plate mounted on mandrel prior to winding the tension elements.

Figure 4. A completed girder in an oven for post-curing resin at 54.4°C (130°F).



diameter rods were used to fasten the stiffeners and lower chord connectors.

Two strands (3,120 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 chord connectors. The 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 0.6 mm (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 more 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 with an adhesive 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 (Methyl-2-cyanocrylate adhesive). Deflections at several panel points during load tests were measured by mechanical dial indicators with least readings of 0.025 mm (0.001 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 76 mm (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 concrete was applied to the girder. Consequently, the finished depth of the concrete varied from 98 mm (3-7/8 in.) at the center of the girder to 83 mm (3-1/4 in.) at the ends. Just prior to placing the concrete, the fiberglass plate was sanded, cleaned with an acetate solvent, and coated with an epoxy adhesive 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 680 kg (1,500 lb.). Cylinder tests of the concrete indicated a compressive strength of 36 M Pa (5,300 psi) and a compressive modulus of elasticity of approximately 24,800 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 23,400 M Pa (3,400,000 psi). This value was used in the analytical determination of live load deflections and stresses.

Live Load Tests

A uniformly distributed live load of 3,680 Pa (77 psf) was applied to the girder by placing steel barrels on the concrete slab and filling them with water. Figure 5 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 6 shows a comparison of the measured and computed deflections at various locations on the girder. The experimental data indicate nearly linear relationships between the load and deflection. These curves also indicate that the deflections were approximately 70% of the computed values.

Subsequently, the barrels were filled with sand to apply an equivalent uniform live load of 4,300 Pa (90 psf) to the slab. The barrels were filled 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 4,060 Pa (85 psf) applied to one-half of the girder length. However, the partial load caused no shear reversal in the unloaded panels so that 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. 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 7 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, the member apparently lost no strength nor stability. Therefore, the load was not removed. Periodic creep deflection measurements and inspections for weathering were continued 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. These data were obtained with a surveyor's level and rod. The settlement at the supports was due to deformation of an asphalt pavement beneath the supports. The fluctuations observed at the centerline of the girder were attributed to thermal effects or measurement errors.

Figure 5. Live load test of TTG-13 with water-filled barrels for a distributed load of 3,680 Pa (77 psf).

Figure 6. Comparison of theoretical and experimental deflections for live load test of TTG-13. 25.4 mm = 1 in.; 47.8 N/m^2 = 1 psf.

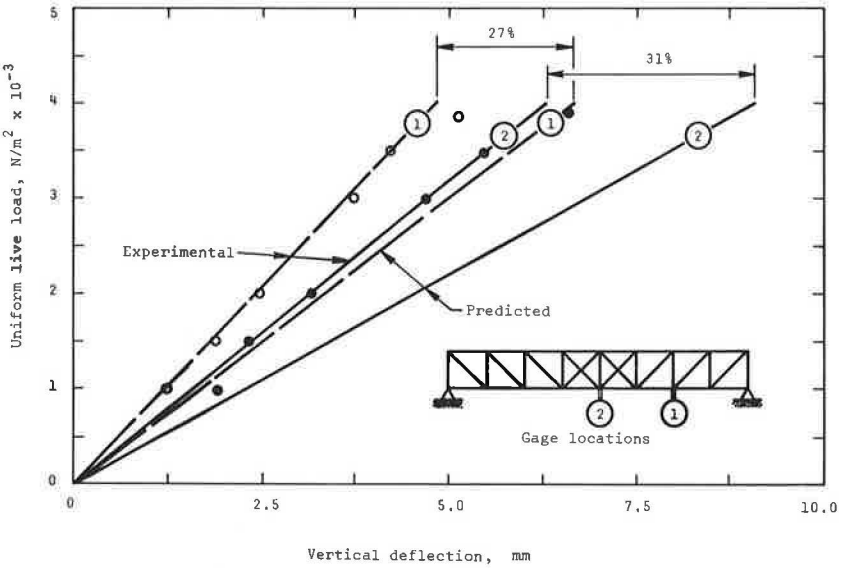
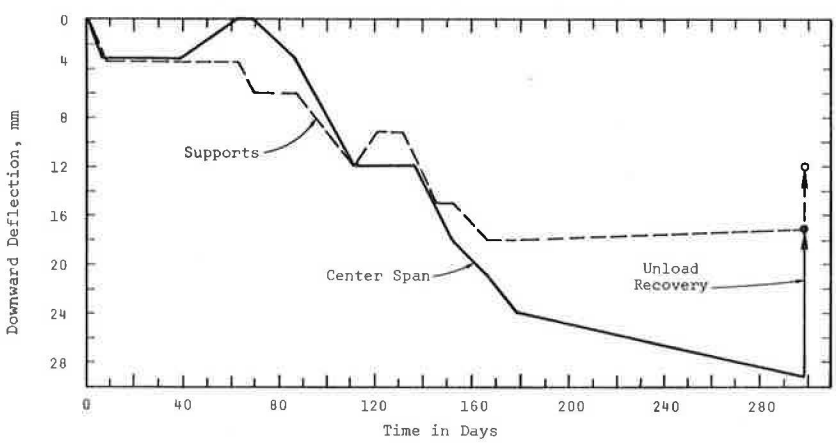


Figure 7. End view of girder TTG-13 showing the failure of a joint at a lower chord connector due to the seat bearing load.

Figure 8. Creep of TTG-13 at center span and settlement of the supports with a constant live load of 4,300 N/m^2 (90 psf).



There appeared to be little, if any, creep deflection at the center of the girder for the first three months of the test. Over the next three months, however, a fairly constant rate of approximately 0.23 mm (0.009 in.) per day was observed. Subsequently, the rate decreased to approximately 0.04 mm (0.002 in.) per day. During this time it was noticed that the wooden supports were deforming at the joints and altering the bearing conditions of the girder appreciably. Therefore, it is not known whether the observed flexural creep of the girder was due to (uncompensated) movement of the supports or the yielding of the GRP materials within the girder. Figure 9 shows the terminal deflection of 76 mm (3 in.) of the center of the girder just prior to removal of the live load.

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.

Figure 9. Deflection of TTG-13 at midspan relative to the ends of the girder just prior to removing the live load.

Creep Test of TTG-12

Experience with the fabrication and testing of TTG-13 indicated the need to study further the consequences of casting the concrete deck slab on a girder with temporary intermediate supports. Therefore, an experiment was conducted with specimen TTG-12 to investigate stress transfer from the fiberglass girder to a concrete slab cast on the top plate. An effort was made also to evaluate the integrity of the epoxy joint between the girder plate and the concrete slab as a load was applied to the composite girder. It was recognized that the deflection of the girder would be excessive under the weight of the fresh concrete because composite action of the fiberglass girder and the bonded concrete slab would not become effective until after the concrete developed compressive strength. It was reasoned that the deflection of the girder should be reduced by supporting it with falsework along its length until the concrete hardened, but it was not known how the top plate of the girder and the concrete slab would interact when the falsework shoring was removed. The experiment with TTG-12 was therefore planned to observe the behavior of the girder (a) as the concrete slab was cast, (b) when the interior supports were removed, and (c) following the load transfer to the slab.

Procedure. The test specimen was instrumented with electrical resistance strain gages and dial indicators and was supported temporarily at the two panel points adjacent to the center stiffeners. Placement of a concrete slab 76 mm (3 in.) thick followed the application of a coating of epoxy adhesive to the top plate of the girder. The slab was moist cured for 8 days during which time the compressive strength reached 21 MPa (3,000 psi). At that time, the interior supports were removed and strain and deflection measurements were initiated. Deflections were measured for 145 days and strain measurements were made for 16 days.

Experimental Results. As was expected, placement of the concrete slab caused downward deflections of the top plate of the girder between supports. The deflections increased equally 0.3 mm (0.012 in.) at each dial indicator during the first 48 hours after placement. Additional deflections of approximately 0.25 mm (0.010 in.) were measured at each gage over the next 6 days. Removal of the interior supports caused a fairly uniform downward displacement of the girder of approximately 1.5 mm (0.060 in.). Displacements occurring after the interior supports were removed are shown in Figure 10.

Discussion of Results. Overall, the test data indicated that considerable time was required for the test specimen to be transformed from a three-span continuous member with an inactive (concrete slab) compression flange to a single-span member with the concrete slab acting as the compression flange. The strain data from the top plate clearly showed that the neutral surface of the girder immediately moved into the concrete slab when the interior supports were removed. These data also indicated that the adhesive bond between the concrete and the girder plate initially was adequate to transfer whatever shear stresses developed at the interface. However, the long-term deflection data shown in Figure 10 are subject to speculative interpretation. One interpretation would suggest that the concrete slab continued to cure and shrink, with consequent increased deflection of the girder. It might also be argued that the concrete deformed due to compressive stresses and that the reinforced plastic tension elements also underwent creep deformation, but the relatively low stress levels involved do not support this contention. Relaxation or progressive failure of the adhesive at the concrete-GRP plate interface also could have caused the deflections observed.

The data of Figure 10 are believed to be reasonably accurate but they do present several disparities. First, it would be expected that the midspan deflection would be greater than that at the points closer to the end supports. The data indicate that all three points deflected approximately equally for the first month of observation. Secondly, there appears to be a significant change in the rate of the deflection at two of the gages approximately 30 days from the time the interior supports were removed. This behavior could occur if the adhesive bond at the concrete slab-plate interface failed suddenly, but if the bond did fail, it seems unlikely that only two of the gages would have been affected.

In summary, the data from this experiment did not clearly identify principal mechanisms controlling the behavior of the girder, but the test procedure and results did provide insight for the anticipated behavior and construction procedures for the prototype structure. Modifications in the design and erection procedures for the prototype resulting from these observations will be described later.

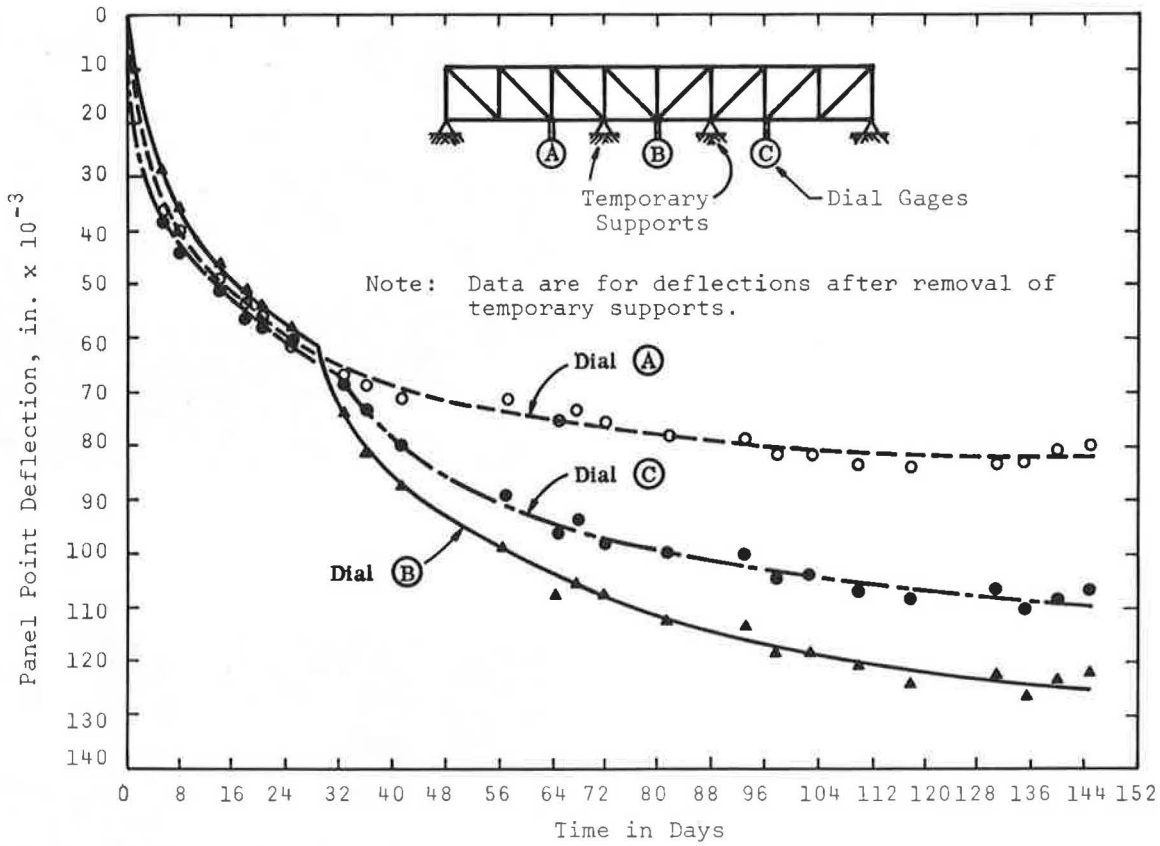
Phase II – Prototype Bridge

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, 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 cm (28 in.) wide and 46 cm (18 in.) deep. Elimination of the cross diagonal elements included in TTG-13 required some changes in the layup 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 46 cm (18 in.) from the 40 cm (16 in.) used for TTG-13 reduced the predicted deflections and stresses from those determined for TTG-13. The design dead load was 2,250 Pa (47 psf) and the design live load was 4,062 Pa (85 psf).

Several modifications in the design were made to TTG-WC as a result of the tests conducted with TTG-12 and TTG-13.

1. The cross diagonal elements were omitted in all of the panels. These were considered unnecessary for the stability of the member.
2. End plates of 12 mm (1/2 in.) thick material salvaged from scraps of the top flange plate were fitted and bonded in the triangular space between the stiffeners at the ends of the girders to preclude the type of failure experienced with TTG-13.
3. Narrow strips of 12 mm (1/2 in.) thick plate were bonded to the top of the cover plate in a direction perpendicular to the length of the bridge. These strips served as mechanical shear connectors between the cover plate and the concrete slab. They also were used as clamps during the bonding of the

Figure 10. Deflections observed in TTG-12 during stress transfer from flange plates to the concrete slab.



cover plate by bolting them securely to the flange plate of each girder.

4. The dimensions of the lower chord connector were altered slightly to provide better end bearing support for the mating stiffeners.

Fabrication of Girders

The fabrication of TTG-WC closely paralleled that of TTG-13 with some modifications. These modifications included the replacement of wooden bolts with brass bolts in the flange-to-stiffener connection, the elimination of positioning pins in the joint between the stiffeners and the lower chord connectors, and sandblasting of all bonded surfaces. A corrosion resistant metallic bolt was used to provide more strength than that available from the wooden bolts. The 6 mm (1/4 in.) diameter pins were omitted from the lower chord connector because they were determined to be structurally ineffective and were not required for the assembly procedure. All mating surfaces which were to be bonded with an epoxy adhesive were sandblasted to assure removal of the release agent used in the manufacture of the pultruded products. A No. 1 silica sand was used in a sandblaster at an air pressure of 275 Pa (40 psi) to clean the surfaces of plates and stiffeners. This operation was time-consuming but provided a better bond surface than previously obtained by belt sanding. The manpower requirement for the winding operation of each of the three girders was approximately the same (six man-hours) as that for TTG-13.

Assembly of Bridge

The three identical girders were assembled to form the prototype structure by bonding a common cover plate (6 mm [1/4 in.] thick) to the top flange plate and by connecting the lower chords with strands of glass-impregnated roving. It was essential that the adhesive joint between the girder flange and cover plate should be as free from voids as possible to develop the shear strength required for the integrity of the compression element for the bridge. Experience in bonding large surface areas during the assembly of TTG-8 (see reference 2) had provided an indication of the difficulty in achieving a void-free joint. The two primary contributing factors in the assembly problem was the inherent warpage of the pultruded plates and the bending, or distortion, of the top plate resulting from the attachment of stiffeners and tension strands during the fabrication of the girder. While both of these factors were individually small, there was some concern that the distortion could be eliminated in order to provide contact over most of the plate area within the 0.075 mm (0.003 in.) thickness required by the glue line. It was obvious that a normal clamping force would be required to hold the surfaces together while the adhesive cured. Because of the large areas involved and the time required to complete the joint, an epoxy that cured at room temperature was selected as the adhesive.

In view of the above considerations, steel beams were placed transversely beneath all three girders and supported at the ends so that the top plate of each girder rested directly upon the steel beam. Careful leveling of the beams and alignment of the

girders longitudinally achieved a reasonably rectangular and planar configuration for the overall surface for the bridge. The cover plates, 1.2 m (4 ft.) wide, were cut to lengths of 2.1 m (7 ft.) to eliminate all longitudinal joints and to simplify the bonding procedure. Four cover plates were therefore used to cover the entire bridge area. Prior to applying adhesive to both plates, the surfaces were cleaned thoroughly with an acetate solvent. Contact between mating plate surfaces was achieved by applying C-clamps along the edges, bolting three clamping strips per cover plate, and applying pressure through an air bag over the center portion of the plate. Figure 11 shows a typical arrangement for bonding one of the cover plates. The supporting steel beams may be seen in the lower right of the figure. The air bag was in contact over the full width of the bridge along a strip slightly less than 1 m (3 ft.) wide. The adhesive was permitted to cure under pressure for at least 5 days at a room temperature of approximately 23°C (70°F). Inspection of the bonded joint after curing was made by visual observation around the edges of the plates and by scanning portions of the plates with an ultrasonic detector. The visual inspection revealed that most, but not all, of the peripheral joints were free of open cracks.

Several regions which were either considered to be well-bonded or had a likelihood of improper bonds were selected for ultrasonic scanning. The well-bonded regions included the central portions of the contact surfaces over which the air bag had been pressurized. The regions of questionable bonds were along the interior edges of the cover plates, the interior edges of the flange plates, and particularly the "corner" areas formed at the intersections of the joints in the flange and cover plates. As described previously, the inherent curvature in the respective plates and the inability to provide positive clamping along the interior edges of the cover plates during assembly raised some doubts as to the nature of the bonded joint in these areas. The ultrasonic inspection was made with a V-Scope, Model C-4960, manufactured by James Electronics of Chicago, as shown in Figure 12. The V-Scope measured the time required for a high frequency audio wave to pass through the two plates and epoxy joint. Calibration of the instrument with a "good" joint and one with an extensive void established a well-defined measurement of the time delay due to the void. Approximately 50 points along the joined edges of the plates were surveyed. Nearly one-half indicated the presence of voids. However, the voids did not appear to extend beyond 75 mm (3 in.) from the edge of the plate. One point (at one of the corner intersections) strongly indicated a crack or a separation of the plates extending approximately 75 mm (3 in.) from the edge. None of the points checked with the V-Scope in the central regions of the plates (presumably well-bonded areas) showed any indication of voids or cracks. From the visual inspections and ultrasonic surveys of the bonded plates, the plate assembly procedure appeared to be successful even though it was tedious and required intermittent steps.

After the cover plate was bonded to the top plate of the girders, the entire structure was inverted and the lower chord connectors of parallel girders were tied together with strands of roving as described previously. This operation was quite simple and was completed within several hours. Figure 13 shows this operation. However, inverting the structure without exerting undesirable forces or distortion on the plates or chords required special efforts and fixtures. Wooden braces were attached

Figure 11. Bonding of cover plates on TTG-WC with pressure applied by C-clamps and by means of an air bag.

Figure 12. Inspection of bonded plate joints by means of an ultrasonic scanner.

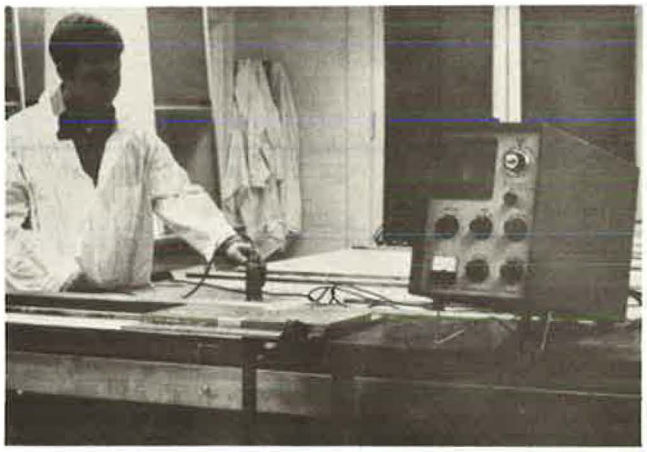
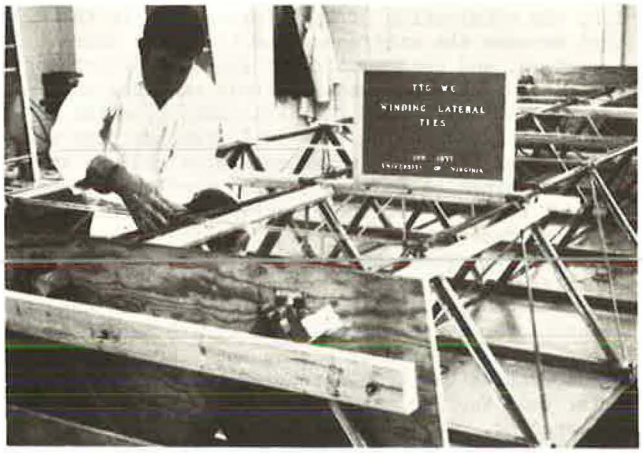


Figure 13. Connecting the lower chords of TTG-WC with continuous strands of resin-impregnated fiberglass.



throughout the bridge and movable wooden towers were built to support the structure as it was rotated.

Instrumentation

The bridge was instrumented with electrical resistance strain gages to monitor the deformation of various diagonals, chords, and plates. A total of 20 electrical strain gages (Micro Measurements CEA-06-250-UW-350) were bonded (epoxy, M-Bond AE 15) to the surfaces at selected locations to measure strains in various stranded elements and plates.

Prefabricated Concrete Seats

Concrete seats were designed and prefabricated in the laboratory to match exactly the sawtooth profile of the ends of the assembled bridge. The seats were designed to provide bearing over approximately the middle one-third length of each end stiffener with sufficient clearance around the lower chord connector to preclude any bearing on the stiffener after settlement in the supports. A 6 mm (1/4 in.) thick layer of elastomeric material was cast into the concrete surfaces along the bearing length to assure uniform pressure on the stiffener surfaces. The width of the seat was 10 cm (4 in.) to accommodate the full width of the double stiffener assembly at the ends of the girders. Attachment to an abutment in the field was provided by dowels extended from the back face of the seat which were anchored when the backwall was cast. No anchorage provision was made to hold the bridge in the seat as the dead weight of the completed structure should compensate for any uplift which might be anticipated.

Schedule for Field Studies

Originally, the erection of the pedestrian bridge was scheduled for the spring of 1977. However, certain environmental considerations at the highway rest area postponed the advertisement of bids for construction. At the time of this writing, it is anticipated that the bridge will be erected during the summer of 1978. Therefore, data from load tests and evaluations of erection procedures of the prototype structure will appear in the final report of the project to be issued upon completion of all proposed work.

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. Fiberglass 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. Joints between pultruded sections and plates were bonded 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 viscosity for the blended adhesive was specified as 2,000 cps and a tensile strength of 24.1 MPa (3,500 psi) was to be developed after curing for 14 days.

Acknowledgements

The reported research is being sponsored jointly by the Virginia Department of Highways and Transportation and the Federal Highway Administration. Administrative oversight is being provided by the Virginia Highway and Transportation Research Council. Special recognition for assistance is given to Husamettin Alper and Wayne Burton, graduate research assistants, and to John Stulting, James Mills, and Clyde Giannini, members of the technical staff of the Virginia Highway and Transportation Research Council.

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

References

1. McCormick, F. C., "Modification Studies for a Bridge Girder of Reinforced Plastics," Virginia Highway & Transportation Research Council, Report No. VHTRC 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 No. VHTRC 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 No. VHTRC 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.

Selected Bibliography

1. Baer, Eric, Editor, Engineering Design for Plastics, Reinhold Publishing Corp., New York, 1964.
2. Benjamin, B. S., Structural Design with Plastics, Van Nostrand Reinhold Co., New York, 1969.
3. "Composite Materials: Testing and Design", American Society for Testing and Materials, Philadelphia, STP 546, 1973.
4. "Composite Reliability", American Society for Testing and Materials, Philadelphia, STP 580, 1975.
5. Structural Plastics: Properties and Possibilities, American Society of Civil Engineers, New York, 1969.