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IDEA Innovations Deserving Exploratory Analysis Project

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

Report of Investigation

IDEA PROJECT FINAL REPORT

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FIBER-REINFORCED POLYMER HONEYCOMB SHORT SPAN BRIDGE FOR RAPID INSTALLATION

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INNOVATIONS DESERVING EXPLORATORY ANALYSIS (IDEA) PROGRAMS
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EXECUTIVE SUMMARY

The purpose of this project was to build a short-span, composite bridge over No-Name Creek (NNC) located three miles west of Russell, Kansas on a Russell County public road. In order to achieve this purpose, a series of tasks were planned and carried out.

First, a design review team was assembled which consisted of representatives from materials suppliers, consulting engineers, academics, representatives from Russell County, and the Kansas Department of Transportation (KDOT). In total, fifteen members participated in the design review committee which was held on April 23, 1996 at the Kansas State University (KSU) Student Union in Manhattan, Kansas. The team spent two hours reviewing the KSCI database that had previously been developed for the strength and stiffness values from the laboratory testing done at KSU by Professor Hugh Walker. The review team then engaged in a wide-ranging discussion from which emerged a series of recommendations to be followed in the manufacture, testing, and installation of the NNC bridge.

The major concern was that the design was to be based upon finite element calculations such that the stresses were not to exceed 10% of the tested material properties. Modulus values were to be taken from values obtained from laboratory flexure measurements. Therefore, it was necessary to conduct experiments on the strength and modulus of a bridge section normal to the span. This was done and the lateral (load-sharing) modulus was used in the finite element analysis.

The finite element analysis (FEA) involved the modeling of a half plate representing one end of the span. The loading was AASHTO HS-25 which specifies axle loads of 52 kip (40 kip + 30% impact factor). The model was subjected to single-axle loads in each lane at the center of the span and two-axle loads straddling the centerline to determine maximum deflection and maximum shear loads, respectively. The materials properties used were determined through a series of tests performed during the analysis phase combined with data previously obtained.

The testing involved the determination of face material tensile properties and the lateral stiffness properties of the proposed cross section. This data was incorporated into the model and a final design was obtained.

The final design based on the analysis consisted of a bridge spanning 23 ft. 3 in. and 27 ft. 9 in. wide, constructed of three adjoining longitudinal sandwich panels. The panels were 22.5 in. thick composed of a 20.5 in. core with a 0.750 in. lower face and a 0.500-in. upper face covered with a 0.750-in. wear surface. Composite vehicular railings were to be attached to the outer edges of the exterior panels.

The center panel of the bridge was the first constructed. The fabrication was done by hand at the KSCI facilities in Russell. After completion, this panel was removed to a test site constructed at the Russell County landfill where a number of real-world load tests were performed to accumulate stiffness data and to proof test the panel. The panel test exceeded all expectations for stiffness and there was no evidence of any damage whatsoever even though the panel had been subjected to more than twice the design load. With this evidence in hand, the fabrication of the final two panels was completed.

The bridge was installed on November 7^{th} and 8^{th}, 1996 by the Russell County Highway Department, supervised and assisted by KSCI personnel. The entire installation required one and a half days from start to finish, demonstrating the simplicity of this type of construction. The bridge was opened to the public on November 9^{th}.

IDEA PRODUCT

INTRODUCTION

The basic concept of the project bridge was to develop a product that would not only serve to introduce fiber-reinforced polymer honeycomb (FRPH) light-weight, heavy-duty structural panels, but also to fill a specific market niche that conventional materials cannot fill. The bridge is a short-span (up to 30 ft. long) bridge built in separate sections that can be transported on a single truck and rapidly installed as a permanent replacement for damaged or destroyed conventional bridges. Typically, the bridge would be designed to meet AASHTO HS-25 specifications and would be manufactured and placed in inventory so that it would be ready to move on short notice. The bridge could be deployed in a matter of hours, transported within 500 miles in 24 hours and installed on existing piers or abutments within 4 to 8 hours.

The bridge described herein is a generic structural type; its specific dimensions are for purposes of illustration. The KSCI process is extremely versatile, and a wide range of bridge sizes is equally feasible. The only important limiting parameter is the bridge length. Bridges longer than thirty feet are not economically feasible at this time. It is planned to stock bridges in two-foot incremental standard lengths. In most cases, the fill behind the abutments can be removed to allow the placement of a longer-than-normal deck.

The rapid replacement permanent bridge is a new concept that fills a market niche that is currently open. For high volume bridges, rapid replacement will limit delays to the traveling public and represent a significant overall economic savings, even if the current cost is slightly higher than conventional structures. It is expected that within three to five years the cost of FRPH bridges will be fully competitive with conventional designs.

DESCRIPTION

General Description

The No-Name Creek (NNC) Bridge is a short-span, self-supporting structure composed of fiber-reinforced polymers (FRP). As finally constructed, the bridge measures 7.08m (23 ft. 3 in.) long and 8.45m (27 ft. 9 in.) wide. The bridge is constructed of three side-by-side panels connected by interlocking longitudinal joints. The panels are covered by a polymer concrete wear surface and rest on existing steel I-beam headers that were part of the original bridge substructure. Vehicle lateral egress is constrained by an FRP railing system. A plan view of the installation is provided in Figure 1.

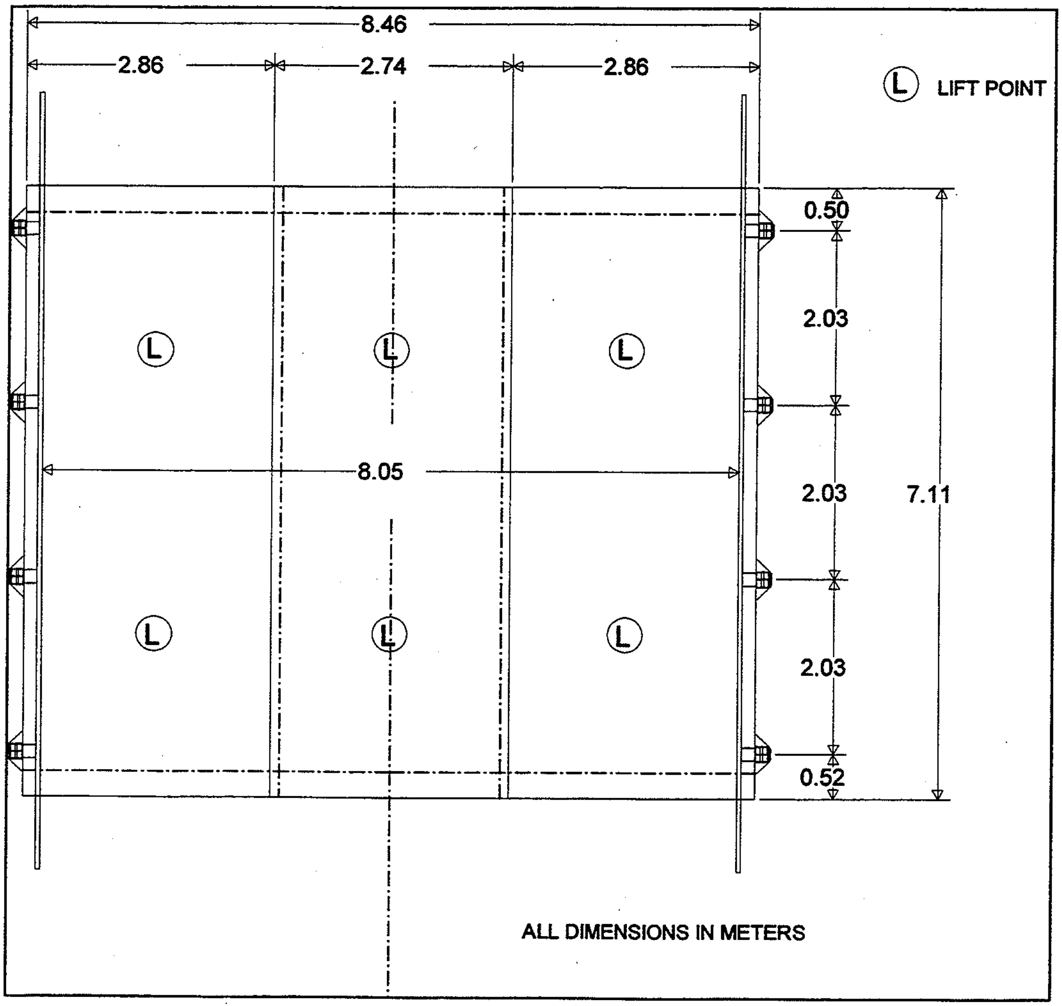

FIGURE 1. Plan View of NNC Bridge

Panel Description

The panels that make up the bridge superstructure are of a sandwich construction. FRP laminates are attached to a closed-cell FRP honeycomb-type core. Details of the fabrication process will be discussed later.

Edge Frame Description

The edge close-out frames are constructed of C-section laminates assembled in a rectangular frame. Where the panels join, one section is installed with flanges facing outwards to form a receptacle for the adjoining panel whose edge is the male counterpart. These sections provide for load sharing between the panels through shear transfer. A schematic of the joints is provided in Figure 2.

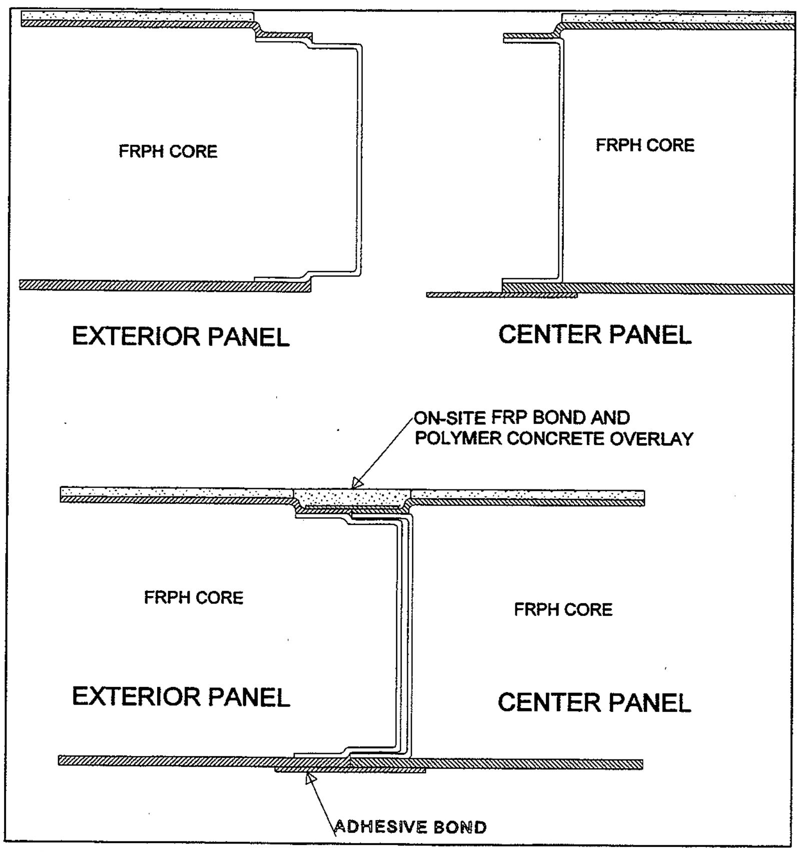

FIGURE 2. Panel Joint Detail

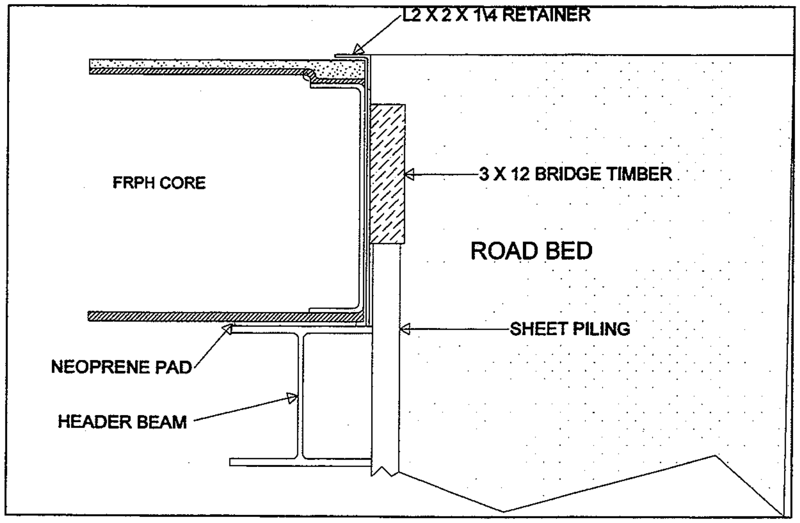

FIGURE 3. Header and Anchorage Detail

Anchorage System Description

The panels are anchored to the substructure with a full-length 2 x 2 x 1/4 steel angle clip over the upper lip of the panels. This retainer is attached by 3/8-in. steel straps welded to the header beam. This method provides constraint against uplift forces and also provides protection for the edge of the wear surface. A schematic is given in Figure 3.

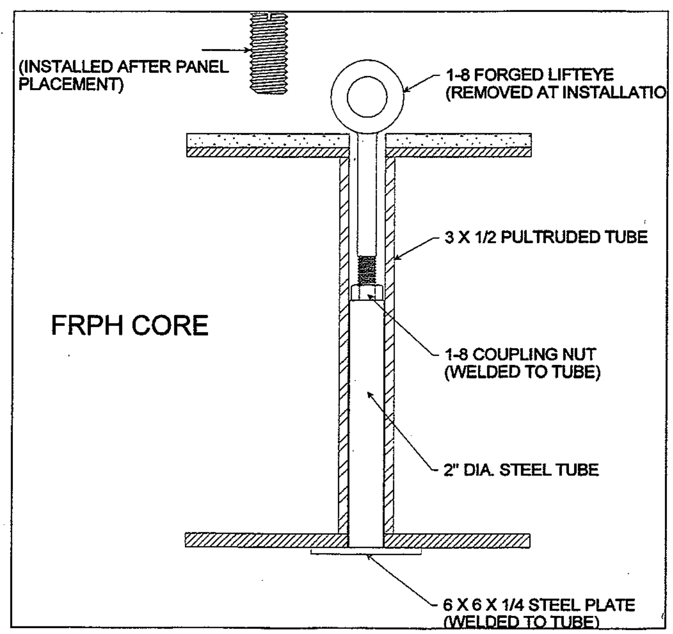

FIGURE 4. Lift System Detail

Lifting System Description

The panels were hoisted into place using the hard point shown in Figure 4. The steel tube/nut assembly was installed after each panel was assembled by drilling through the finished panel at the location of the pultruded tubes and inserting the assembly from underneath.

Railing System Description

One of the objectives of the project was to demonstrate the viability of constructing a vehicular bridge using only FRP materials. This included the installation of the FRP railing system shown in Figure 5. The post pockets were fabricated separately and installed in the edge frames before final panel assembly.

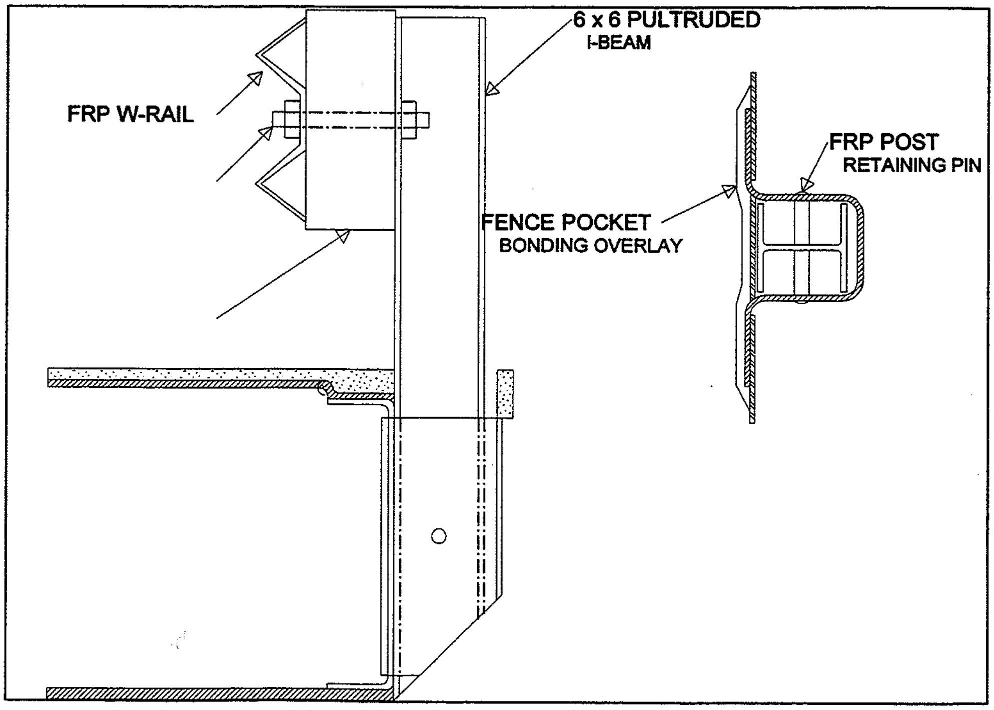

FIGURE 5. Vehicular Railing Detail

Wear Surface Description

The wear surface consists of a local gravel aggregate in a polyester resin matrix. This mixture has a density near that of conventional concrete. The specifications of the aggregate mixture used on the NNC bridge wear surfaces are generally the same as those used by the California Transportation Agency (CALTRANS). The aggregate can be generically termed as a 3/8-minus mixture, meaning that 100% of the aggregate will pass a screen with a 3/8-in. square mesh.

POTENTIAL IMPACT ON CURRENT ROAD AND BRIDGE PRACTICES

It is felt that the use of the KSCI short-span bridge's modular construction will be of great benefit in replacing and repairing bridges. Field labor and equipment costs would be considerably lower than current methods. The down time of the corresponding road, and therefore the aggravation cost to the public, would be lower than with current construction methods. The time required for a proper cure of a poured-in-place concrete bridge can be 30 days. The assembly of any type of structural component bridge, i.e., that involves steel or pultruded composite members, is also time consuming.

Maintenance costs should be lower. One current rapid installation bridge involves the overlay of asphalt or concrete on corrugated steel sheet placed over steel stringers. While this is an inexpensive and relatively quick method of construction, the life span of this type of bridge would be relatively short due to corrosion of the steel. Steel bridges need to be repainted for corrosion protection periodically. This can be a somewhat dangerous operation depending on the type of structure. Repainting of steel members, with its detrimental environmental effects and labor costs, would not be required of a composite bridge. Current design practice for a bridge containing steel members or reinforcement requires that a corrosion factor be added into the safety factor when calculating strength of the members. This practice results in a bridge that is much stronger initially than is required for the design load. A composite bridge would not require this sort of overkill and would provide a more cost-effective use of materials. Only time will tell if this product will live up to its potential benefits in this area, but, from past experience involving other products manufactured from FRP composite materials, this bridge should require no additional corrosion protection over its life as a bridge containing steel would.

The only maintenance probable on the nnc bridge would be replacement of the polymer concrete overlay due to wear. Current polymer concrete application procedures have been in use for a number of years. Replacement or repair of this surface would be easier, and certainly faster, than repair of a concrete surface.

Polymer concrete does not exhibit the spalling and cracking problems associated with conventional concrete because of the elastic nature of the resin binder. Another advantage of the overall design is the compatibility of the materials involved. The resins used in the bridge structure and the wear surface are similar in chemistry and they show good adhesive properties;

CONCEPT AND INNOVATION

CONCEPT

Overview

Currently, FRP composites are used in two major types of applications: 1) aerospace and 2) light, non-critical structures. However, neither of these applications serves as a useful model for infrastructure applications such as bridges. Aerospace composites have reached a high degree of refinement through careful modeling analysis, testing, and relatively long periods of use. However, this high degree of perfection and sophistication comes at great economic cost. With current costs, it is simply not possible for aerospace composites to compete economically with steel, concrete, or wood as infrastructure material.

Light structural FRP composites are useful as architectural materials in decorative, non-critical structural applications because of a combination of high cost and less-than-satisfactory physical performance.

Based on an analysis of all the information relative to composites, KSCI personnel came to see a ray of hope for FRP infrastructure applications if the major advantages of composites, namely their generally excellent physical properties vs. weight, could be used to offset their disadvantages of low stiffness and high materials and manufacturing costs.

An initial study of the applications of FRP composites came to several conclusions. These are:

- The barriers to adoption of FRPs by the engineering community were their low stiffness, which greatly reduced their resistance to deflection, and their low strengths vs. conventional materials.

- Design of competitive, affordable structures were not thought to be possible by potential users.

- Environmental deterioration was thought to be an unsolved and unsolvable problem.

- FRPH must offer clear specific advantages to departments of transportation and county highway departments if it is to be accepted in the marketplace.

therefore, a superior bond between the two surfaces and the wear surface can be applied in a relatively thin layer as opposed to an asphalt overlay. Repair of a concrete surface involves the removal of the entire wear surface and replacement with a new overlay. Surface preparation would also be minimal and road closure would only be required for a number of hours rather than days.

- FRP must offer the traveling public obvious advantages of fewer traffic delays and disruptions.

- Higher materials costs can only be offset by lower field labor expenses.

- FRP could not succeed by mimicking the geometries of steel and concrete.
- The use of FRP panels as bridge decks for new and replacement work was seen as the area of application most likely to become cost effective since the forming costs of conventional materials were high with respect to the amount of material used.

- Rapid replacement of failed or destroyed bridges that could be placed on existing sub-structures in a few hours was seen as an important advantage.

- FRP would not be accepted until a complete composite bridge was fabricated, tested, and installed.

Based on these thoughts and observations, KSCI personnel decided on the following development strategy as the most likely route to the acceptance of FRP materials for bridges and other applications.

- Given the low modulus of FRP materials, the geometric advantages of sandwich construction must be utilized.

- Given the low density of FRP materials, large panels could be factory-made and shipped to installation sites by road.
- Short-span, light-weight bridges of various lengths could be kept in inventory and deployed when needed.

TABLE 1. Center Panel Laminates and Properties

PANEL PARAMETERS

Core Type:	standard	
Length:	279	in.
Width:	104	in.
Depth:	20.5	in.
Web:	0.090	in.

CORE

Laminate: 4.5oz/ft^2 CSM
Resin: AOC 7RCP

Number of Flats:	52	
Number of Flutes:	52	
% Reinforcement:	40.0	%
Core Density:	0.844	lb./bd. ft.
Total Core Weight:	3485.32	lbs.

Wear Surface

	Weight (lbs.)	Thickness (in.)
Polycon	1888.94	0.750

Face 1

Glass Percentage	44.90%			
Laminate Schedule	No. Layers	Description	Lamina Weight (lbs.)	Thickness (in.)
	1	cm 3205	100.01	0.059
	10	μm1810	644.80	0.381
	1	cm3205	100.01	0.059
	1	bonding	226.69	0.158
		Totals	1071.51	0.657

Face 2

Glass Percentage	50.17%			
Laminate Schedule	No. Layers	Description	Lamina Weight (lbs.)	Thickness (in.)
	1	cm 3205	100.01	0.059
	15	μm1810	967.20	0.572
	1	cm3205	100.01	0.059
		Totals	1167.22	0.689

Panel Weight (less closeout)	7612.9	lbs.
Weight/Area	37.7	lb/ ft.^2
(less wear surface)	28.4	lb/ ft.^2

TABLE 2. Exterior Panel Laminates and Properties

PANEL PARAMETERS

Core Type:	standard	
Length:	279	in.
Width:	113	in.
Depth:	20.5	in.
Web:	0.090	in.

CORE

Laminate: 4.5oz/ft^2 CSM
Resin: AOC 7RCP

Number of Flats:	5	
Number of Flutes:	5	
% Reinforcement:	40.0	%
Core Density:	0.844	lb./bd. ft.
Total Core Weight:	3786.9	lbs.

Wear Surface		Weight (lbs.)	Thickness (in.)
	Polycon	2052.41	0.750

Face 1

Glass Percentage 44.81%

Laminate Schedule	No. Layers	Description	Lamina Weight (lbs.)	Thickness (in.)
	1	cm 3205	108.66	0.059
	10	μm1810	700.60	0.381
	1	cm3205	108.66	0.059
	1	bonding	246.30	0.158
		Totals	1164.23	0.657

Face 2

Glass Percentage 50.17%

Laminate Schedule	No. Layers	Description	Lamina Weight (lbs.)	Thickness (in.)
	1	cm 3205	108.66	0.059
	15	μm1810	1050.90	0.572
	1	cm3205	108.66	0.059
		Totals	1268.23	0.689

Panel Weight (less closeout)	8271.8	lbs.
Weight/Area	37.7	lb/ ft.^2
(less wear surface)	28.4	lb/ ft.^2

- Complete bridges, including railings and wear surfaces, could be manufactured.
- Any manufacturing process must allow future automation and mechanization to lower costs.
- It must be demonstrated that a short-span bridge could be designed, manufactured, and proof-tested, then transported and installed in a matter of hours.
- The bridge must be designed to meet current highway standards.
- As determined by finite element analysis, the stresses induced in the structure must not exceed 10% of the tested tensile, shear and compression values.
- Direct manufacturing and materials costs must not greatly exceed those of conventional materials.
- Panels would be produced using two load-bearing surfaces with a honeycomb core.
- Prototype fabrication would be done using manual methods of contact molding.

A series of deflection tests of various specimen sizes and thickness would be performed to develop an engineering database.

Historical Background of the Technology

The use of alternate layers of flat and corrugated paper to produce a honeycomb core is an old and well established art. From this beginning, several workers have labored to produce FRP products of this type. Most notable is the pioneering work of Mr. Bernard P. Kunz who, starting approximately 35 years ago, produced a series of over 200 prototypes. This work was based on a corrugation having a 4-in. wavelength and a 2-in. amplitude. Mr. Kunz has widely reported this work and has obtained a number of patents related to it. All patents except two have expired and the patents remaining in force relate to an open-cell core and its manufacturing process. All other aspects of the FRPH technology known to the authors are in the public domain.

None of Mr. Kunz's prototypes involved the production of panels suitable for vehicular bridges. The largest structures that were fabricated were two low-profile domes of approximately 140 ft. in diameter covering a trickle bed filter at a waste water treatment plant. These domes were successfully installed but were dismantled after fourteen years when they became structurally unstable. The most probable cause of this premature failure was inelastic deformation (creep). Stress levels in the domes were thought to be excessive and creep made the shells subject to snap-through and collapse. No service data was acquired during the lifetime of these structures and the failures were not fully analyzed, but the described sequence of events is thought probable.

Adaptation of FRPH for Light-Weight, Heavy-Duty Applications

In 1994, after it was learned of the impending failure and then removal of the aforementioned domes, KSCI personnel spent considerable time evaluating the potential of these materials in infrastructure applications, specifically bridges. The scientific and technical literature was surveyed in an attempt to understand the failure mechanism of FRPH, the consequences of failure, and most importantly, choosing design criteria that would limit the likelihood of failure to an acceptable level of risk by using conservative design practices and material property values.

Because of the dome experience and the lack of any engineering data on FRPH materials, a two-year study was begun to fabricate FRPH specimens of those sizes and types thought to be adequate for heavy-duty structural panels. The purpose of the study was to test these samples and determine basic flexural properties and strengths that could be used in the design process.

A study was made of the larger and heavier panels regarding the feasibility of their manufacture. Also, methods of joining such panels such that loads could be successfully transferred were studied.

Development of the Engineering Database

Over 100 FRPH panels were fabricated and tested. First, panels 4 ft. × 1 ft. × 2 in. were fabricated with different core web types and web thickness. These panels were tested and then a series of beams 8 ft. long were produced in thickness ranging from 2 to 12 in. and with core webs from 0.040 in. to 0.125 in. in thickness. Face laminates on these beams ranged from 0.250 in. to 0.750 in. in thickness and were made with fiberglass fabrics having a variety of weights and orientations.

The largest sample produced was a beam corresponding to a section of bridge deck. This piece was 22 ft. long in the direction of span and 1 ft. wide. Total panel thickness was 14 in. Using the flexural rigidity values from previous experiments and elementary beam

deflection equations, the empirical deflection-to-load curve was found to be within a few percent of the calculated curve. The beam was not tested to failure, but acoustic evidence suggested that failure was imminent at a span to deflection (*L/d*) ratio of approximately 50. A design to an *L/d* of 250 would give the beam a safety factor of five.

The results of these tests gave KSCI sufficient confidence that the design and fabrication of a fiber-reinforced polymer honeycomb bridge was feasible. A solid empirical foundation had been established, but this project was needed to demonstrate the validity of the concept.

INNOVATIONS

Fabrication of Honeycomb Composite Materials Suitable for Short-Span Bridges

This project produced the first FRPH structural panels for heavy-duty load bearing applications. Their fabrication is a major advancement in the use of composites for infrastructure. These panels were produced by manual methods, but the manufacturing process is not so complex that it could not be automated and mechanized in many areas to allow these products to be cost-competitive with conventional materials.

The honeycomb bridge panel concept is structurally efficient and this project demonstrates the innovative use of old technology in a completely new application. Others are attempting to produce FRP composite components for infrastructure applications using different production processes, but these are not as efficient in terms of cost or labor as FRPH.

Design Methods for FRPH Structures

Methods of designing FRPH structures using both simplified and advanced calculations have been developed. The design procedures are, of course, based upon existing civil engineering methods and represent an innovative extension of these methods. More detailed studies involving finite element modeling of the core structure will help to optimize the design of honeycomb panels to improve performance and reduce costs.

Proof Testing versus Engineering Analysis

An important innovative step resulting from this IDEA project is the use of proof testing as a part of the qualification process for FRPH structural panels used in critical load-bearing applications. KSCI decided that since FRPH panels were light enough to be handled easily, the panels could be proof tested with loads in excess of the design loads in order to assure that sudden catastrophic failure will not occur. Proof testing does not account for long term failure modes, but it does represent an innovative approach to ensuring product quality.

INVESTIGATION

Prior to final design and manufacture of the NNC bridge, a design review team was assembled in order to determine the engineering requirements and specifications, and an associated test program to assure that the NNC composite bridge would be safe and fulfill the requirements of a short-span bridge structure.

Based on the recommendations and insights provided by the committee, a testing and analysis program was developed. Other smaller scale meetings were held with various members to address specific issues such as installation. The actual fabrication process was determined by KSCI personnel and the bridge installation was performed by the Russell County Highway Department and KSCI.

CONCLUSIONS AND RECOMMENDATIONS

Critical Design Factors

A design review committee agreed that, while other elements of the NNC bridge were important, the two most important design factors were:

- That the bridge possess sufficient structural strength to support the loads for which it is designed with some reasonable safety factor, and
- That the bridge deck possess a surface that would allow vehicle control under normal driving conditions and circumstances.

General Recommendations

It was agreed that the basic bridge design should be conservative since the use of FRP sandwich structures on this scale had not been previously attempted.

The basic specifications for bridge design are set forth in the AASHTO Standard Specification for Bridge Design. While this standard does not apply directly to bridges built on county roads or provide standards for design with FRP materials, it was hoped that future application of this technology would include state highways and that the bridge should meet the AASHTO

requirements insofar as possible. If a bridge could be designed that would exceed these requirements, it would provide a degree of confidence in the engineering community at large. It was decided that the AASHTO HS-20 (modified) loading, as currently used by the Kansas DOT and Russell County, would be used as a basic criteria.

It was further decided that, as there was already a body of knowledge and experience in the use of polymer concrete materials on existing roadways, a standard specification for this material would be utilized for the wear surface on the NNC bridge.

Recommendations for Testing and Design

A large number of bending tests had been performed on FRP sandwich beams with thickness ranging from 2 in. to 12 in. under previous grants. These tests had focused on generating a database of properties of various sections in the longitudinal material direction. There was, however, a need to determine the basic properties of this construction in the lateral direction of the plate.

It was suggested that a full- or half-scale panel be constructed and tested to determine these properties. Given the economic constraints of the project, it was decided that a beam representative of the lateral construction of the panel be tested and that the resulting data be combined with previously acquired data to provide a basis for computer analysis.

A series of finite element analyses using the beam deflection data would be performed on various panel geometries to determine a viable design for the NNC bridge.

Following this analysis, a full-scale bridge panel would be produced and subjected to a proof test to verify the results. In this way, a design procedure could be developed for future implementation.

Additional Recommendations

It was recommended that a long-term monitoring system be constructed under the bridge. The purpose of this structure would primarily serve for mounting instrumentation to measure long-term inelastic deformation, and load deflection. Additionally, it would serve as a safety system to limit bridge movement in the event of catastrophic structural failure.

ADDITIONAL LABORATORY TESTING

Based on the recommendations of the review committee, a beam representing a lateral section of the proposed bridge panel was fabricated by KSCI. The primary considerations for testing of this beam were to determine the bending and shear stiffness properties of the proposed section. It was determined that the beam should be subjected to three-and four-point flexural loadings per ASTM C393-62, Standard Test Method for Flexural Properties of Flat Sandwich Constructions.

In addition, tensile tests were performed on the face laminates of the beam. These samples were prepared from excess face material when the beam was fabricated.

The bending tests were performed by Professor Hugh S. Walker at Kansas State University (KSU) with KSCI personnel in attendance. The tensile tests on the face laminate were performed by John Held at KSU.

Description of the Test Beam

The beam dimensions and a laminate schedule are included in the Appendix. The weight and thickness of the face laminates were greater than originally designed. The glass weight is the same as the original design, but the resin percentages are greater. This is due to variations in the uni-directional fabric used in the construction. The fabrication of the bridge utilized fabrics more closely resembling the original design. It was hoped that the results of the test would provide properties in a range that could be used as a basis for the final design.

No wear surface was applied to the beam so that the properties of the structural panel could be determined. The wear surface was deemed to be sacrificial over the life span of the bridge.

As the beam was to represent a panel section normal to the flow of traffic and the faces were composed primarily of uni-directional fibers, the fabric was laid transverse to the major axis of the beam. As the core is not isotropic, it too was assembled in a transverse direction.

The actual fabrication of the beam did not follow the procedure identical to that used to fabricate the actual bridge panels. The original plan for fabrication of the panels called for the upper-face laminate to be laid out and the frame and core to be impressed into this wet layer. This laminate would be allowed to cure and the panel would then to be revolved about its longitudinal axis and impressed into the wet laminate of the bottom face. The core was to be assembled as a full-size panel.

This method of fabrication quickly proved to be impractical without the availability of large material

handling equipment. Also, the roof of the building in which the fabrication would occur was not high enough to rotate the panel. The beam, however, being relatively light in weight and small in size, was constructed following this original procedure.

Testing

Flexure Testing of the Beam

The beam was supported on a span of 84 in. for both the three- and four-point tests. The loading during the four-point tests was applied at the quarter points of the span (±21 in. from center). The loads given in the data are total loads on the beam.

Three series of loads were applied under each loading condition. The first series in each case was used to settle the beam and the equipment; the data is included here only for completeness.

Results of the Flexure Test

The equations used to calculate the properties are modified forms of the ASTM C393 equations. All tests were performed over the same span and the same load was used in both equations, leaving only the deflections as variables. The resulting equations are:

$$D = Pa^3/[128*(2w_2 - w_1)]$$

$$G = 3Pac/[b*(h + c)2*(11w_1 - 16w_2)]$$

where:

D = flexural rigidity (lb- in.^{2})
G = apparent shear modulus (psi)
P = load (lbs.)
a = support span (in.)
w_1 = deflection under center load (in.)
w_2 = deflection under four-point load (in.)
c = core depth (in.)
b = beam width (in.)
h = beam depth (in.).

Calculations using test data resulted in the following values for D, G, and the apparent face modulus, E. The center point deflections on the lower face for Series 3 were used. The results are given in Tables 3 and 4.

TABLE 3. Stiffness Properties of Lateral Test Beam

	D (lb- in.^{2})	E (psi)	G (psi)
20000	$9.4*10^8$	$4.3*10^5$	$5.1*10^4$
25000	$9.9*10^8$	$4.5*10^5$	$3.7*10^4$
30000	$10.0*10^8$	$4.8*10^5$	$3.0*10^4$

Calculations using linearized data for the same loads gave the following results:

TABLE 4. Linear Properties of Lateral Test Beam

D (lb- in.^{2})	E (psi)	G (psi)
$11.0*10^8$	$5.5*10^5$	$2.5*10^4$

Observations on the Results of the Beam Tests

Shear modulus was somewhat higher than expected. The expected value was in the 10 ksi to 15 ksi range. It is possible that the faces contribute substantially to the shear stiffness of the beam due to their thickness.

The elastic modulus for the faces was lower than expected. There are two factors that would contribute to this result. First, the reinforcement content is low in the direction of the beam axis. There are no uni-directional fibers in this direction and a minimal amount of chopped strand mat. Second, the solution of the two deflection equations allows a coupling between the shear and flexure variables. A partial decoupling may be accomplished by obtaining tensile modulus values for the faces and using these values to cross-check the apparent face modulus.

Nevertheless, the values generated by this test remain valid for design purposes given that the beam tested was an accurate representation of the bridge cross-section

and they therefore represent valid macroscopic properties of this particular composite construction. It is apparent from the load/deflection calculations in the tabulated data that, overall, the beam becomes stiffer with greater deflection. This phenomenon is most likely due to increased load sharing by the glass fibers in the faces and take-up in the bonds between core sections and between the core and the faces due to shear deformation.

It is interesting to note from the calculations that shear stiffness decreases substantially (-40%) versus load but flexural stiffness increases slightly (+6%). The decrease in shear values may be due to deformation of the core flat webs under the higher loads. An interesting comparison would be between the current crown-to-flat core construction and crown-to-crown assembly.

The beam was not tested to failure; therefore, the maximum stiffness value remains unknown, but at the 42-kip load applied under 4-point bending (series 3) the beam begins to show a decrease in overall stiffness. There was also a noticeable increase in the number of acoustic emissions in this load range though no evidence was found of any structural damage. Testing was stopped at this point.

The data indicates that a residual deformation remains after relaxation. It was observed that this deformation was recovered after approximately 5 min. This is probably due to elastic creep of the faces during bending due to the high resin content in the axial direction. This does not appear to be a permanent condition.

A maximum load of 30 kip was applied during the three-point tests and 42.5 kip during the four-point test. These loads are in excess of the design wheel loads under the AASHTO loading specification and were applied to a 12-in.-wide section. Given that the level of acoustic emissions did not begin until these loads were reached and that no structural damage to the beam was observed, a high degree of confidence should be felt regarding the viability of the design.

Tensile Testing of the Face Laminates

The object of this series of tests was to determine the tensile properties of the laminates used as faces for the No-Name Creek Bridge panels. Samples were taken from the lateral test sample NNC.II.A and subjected to tensile testing to determine tensile modulus and tensile strength in both the *L* and *W* beam directions. These results were then applied as material properties in the FEA model.

Samples were tested in both the *L* and *W* directions. The *L* direction is the uni-directional fiber direction and was normal to the beam axis (as the beam represented a lateral section of the bridge panel). Yield stress was taken at 1% strain. Young's modulus was taken from the portion of the curve before yield.

Results of the Tensile Tests

The results are given in Table 5. Face 1 samples represent the upper face of the beam and Face 2 samples represent the lower face.

TABLE 5. Results of Face Laminate Tensile Tests

Sample	Maximum Stress	Thickness	Yield Stress	Young's Modulus
Face 1 *L*	33850	.493	17450	$2.19*10^6$
Face 1 *W*	10950	.518	8942	$1.04*10^6$
Face 2 *L*	31177	.941	17816	$1.80*10^6$
Face 2 *W*	6673	.813	—	$0.49*10^6$

Observations on the Results of the Tensile Tests

Graphical data for Face 1 in the *L*-direction showed a two slope curve with a knee at approximately the 1% strain level. The *W*-direction test on this laminate did not exhibit this phenomenon. Face 1 strain was measured by jaw displacement. Face 2 strain was measured by extensiometer. The knee was also apparent in the data from an eight-layer sample of A0108 and .75oz CSM. Mr. Held attributes this to progressive uni-directional fiber breakage at loads beyond yield.

Moduli for Face 1 are higher than for Face 2. This can be explained by the higher percentage of reinforcement in Face 1. The *W* modulus for Face 2 is particularly low. This shows the importance of maintaining a high reinforcement content during manufacturing. The low

glass contents in the faces of the beam sample can be attributed to the lower density of the fabric used. While the glass weight of two layers of the A0108 fabric combined with .75oz CSM was the same as new C1708, the combined layers did not lay as compactly. This accounts for the increased thickness of the test beam faces versus the original bridge design. It is thought that the increased moment of inertia provided by this increased thickness will compensate for the loss of stiffness due to the lower modulus.

ANALYSIS AND DESIGN

With the data and results from the materials testing described above, it was possible to begin an analysis process that would result in the final design of the NNC bridge. It was determined that the most cost-effective design scenario would involve the use of finite element analysis (FEA). The advantage of finite element modeling is that it allows rapid and accurate analysis of a large number of proposed designs without the need for extensive live model testing. This is especially true for composite structures such as the No-Name Creek bridge due to the non-traditional nature of its materials and construction.

A finite element model was constructed to the dimensions of the bridge and was used to evaluate changes in both the geometry of construction and the constituent materials in order to optimize costs and then measure the results against current standards for bridge construction. The FEA model, analysis procedure, and results are described in the following paragraphs.

Finite Element Model Used in the Analysis

FEA Software

The model was generated using ALGOR finite element software and its attendant computer drafting module Super Draw II, along with the composite materials extender.

The laminate schedule is defined in the composite decoder by defining material properties of the constituent laminate, including the core, and then assembling them into the proposed structure. Properties for each lamina can be stored in a library for use on future projects.

Elements

The base element used in the model is ALGOR's type 16 sandwich composite element with three- or four-node construction. These are an isoparametric thick-plate element with five degrees of freedom (three translation and two rotation) based on Mindlin theory.

Geometry

The basis for the model is a full width of the half-span. A half-span was deemed adequate due to the symmetric nature of AASHTO loading requirements. AASHTO also requires that a span be loaded simultaneously in all traffic lanes, therefore a full-width model was necessary which incorporated all three slabs.

The proposed bridge is currently 333 in. wide and 279 in. long. Center-to-center distance of the support headers is 267 in. AASHTO lane widths are generally 144 in. wide, leaving 22.5 in. on the outer edge of each panel for curb and guardrail.

The origin was placed on the longitudinal centerline at the supported end of the panel. The XY-plane contained the neutral surface of the panel. The X-axis was in the direction of traffic flow. The positive Z-direction was upwards.

Mesh

The element mesh was composed of 1475 elements constructed from 1564 nodes. The elements are generally rectangular and approximately 5.5 in. x 5.5 in.. Elements along longitudinal edges ($y = \pm 166.5$) and the longitudinal centerline ($y = 0$) are 3 in. x 5.5 in. in order to accommodate inclusion of edge beam properties. The mesh density under the wheel loads was doubled by triangulating the rectangular elements in those vicinities.

Loading

The AASHTO HS-25 (modified) base load for the rear axles is 40 kip. The impact factor based on the span of the NNC bridge is 30%; therefore, the total load applied by the rear axles is 52 kip. All wheel loads were applied as uniform pressures of 100psi acting downward on the wheel elements. Wheel loads were distributed over areas of 12 in. x 18 in and spaced at 6 ft. The minimum axle spacing for the AASHTO truck is 14 ft. These values were used in the analysis. The dead load is calculated in the processor from material density data and is applied to all elements of the panel. This feature can be eliminated, if desired.

Two axle placement scenarios were required to satisfy the AASHTO design criteria. In the first scenario, wheel loads representing one axle in each traffic lane were placed on the lateral centerline of the bridge.

Inside wheel centers were approximately 48 in. from the bridge centerline. Outside wheels were approximately 120 in. from the center. This loading produces the greatest deflection and tensile-stress levels in the faces.

The second scenario requires the placement of two axles in each lane to produce the highest level of vertical shear stresses in the core. These axles were spaced 14 ft. apart and straddled the lateral centerline of the bridge.

Boundary Conditions

The bridge panels were deemed to be simply supported on both ends; therefore, translation was constrained in all directions at x = 0. Rotation about the y-axis was constrained along the lateral centerline of the bridge at x = 139.5. These conditions were appropriate for the current loading on the lateral centerline, but would not, of course, be valid for loads which do not have symmetry about this line.

Outline of Analysis

Following is the basic outline of the analysis procedure followed in the design process.

I. Design panels for *L/d* of 500. No edge or joint stiffening.

II. Analysis with no edge or joint stiffening
  A. Both lanes loaded
    1. Two axles spanning centerline
    2. Single axles on centerline

III. Analysis with rigid edges and joints
  A. Both lanes loaded
    1. Two axles spanning centerline
    2. .Single axles on centerline

IV. Analysis with elastic edges and joints based on current C-section closeouts
  A. Both lanes loaded
    1. Two axles spanning centerline
    2. Single axles on centerline

V. Wheel loads centered at 2 ft. from longitudinal centerline
  A. Both lanes loaded
    1. Two axles spanning centerline
    2. Single axles on centerline
  B. Single lane load
    1. Two axles spanning centerline
    2. Single axles on centerline

VI. Panel with wear surface applied
  A. Both lanes loaded
    1. Two axles spanning centerline
    2. Single axles on centerline

VII. Thinner top face and thicker bottom face to determine the affect on safety factor

VIII. Change failure criteria to ultimate strength values of the uni-directional layers for the longitudinal direction and the CSM layers for the lateral direction. This would be first-ply failure of a strength layer rather than failure of a fill layer.

IX. Core properties and strengths from KSU testing of May, 1996
  A. Face thickness .375/.625
    1. Two axle
    2. Single axle
  B. Face thickness .500/.625
    1. Two axle
    2. Single axle
  C. Face thickness .500/.750
    1. Two axle
    2. Single axle

The initial analysis (*I*) was to determine a base line for further study. It was assumed that a span to deflection ratio (*L/d*) of 500 for a panel without wear surface would be sufficient for initial calculations. Phases II through IV were used to determine the effect on displacements of the edge closeouts and joints. In Phase V, the wheel loads were moved closer to the centerline to provide worst-case loading. In Phase VI, the wear surface was applied to determine its effect on *L/d.* Phase VII began the optimization based on variations of face thickness. Phases I through VII used the highest ultimate strength values of the laminates in each direction to determine failure. For Phase VIII, this was changed to values that would define a first ply failure rather than an ultimate laminate failure. Phase IX repeated the analysis using values obtained from the testing done at KSU.

Results of FEA

Following is a summary of observations on the analysis. The definitions of the stress designations, failure criteria, and material stiffness properties are given in the Appendix.

Phases I Through IV

The initial panel geometry derived in Phase I pointed to a core depth of 20 in. with face thickness of 0.500 in. The optimum geometry pointed to a core depth of 23 in.

with faces of 0.200 in. but it was felt that a top face of this thickness would be prone to buckling failure and it was decided to use the 0.500 in. value given the lack of data regarding buckling behavior.

Observations to this point are summarized as follows:

- S11 decreases with increasing edge stiffness (beams are taking more of the load).

- S22 increases with increasing edge stiffness (deflection resistance shifting to y-direction).

- S12 shows minimal change, maximum at corners S13 increases with increasing edge stiffness S23 constant.

- Von Mises stress decreases with increasing edge stiffness.

- Tsai-Wu criteria show a factor of safety of 4 to 5 for the bottom face and greater than 10 for the top face. The difference is due to the way the criteria treats axial stresses. Axial stresses on the top face are negative and therefore subtract from the final value.
- The safety factors are based on the lowest ultimate strengths derived from computer analysis of the proposed laminate. Physical properties of the laminates are also based on computer analysis. Shear properties for the core are based on the testing done at UCSD.

- Shear stresses in the core are plate values as are the strengths used to determine the factor of safety. The stresses are very low; therefore, core failure should not be a problem.

- This initial analysis was made on a panel without a wear surface. The addition of polymer concrete should increase stiffness dramatically.

Phases V and VI

The worst case for the current design occurs during V.A.2 loading. The factor of safety for the bottom face is approximately 3:1. It must be remembered that the factor of safety is based on the lowest strength obtained for the given laminates. In this case, the transverse tensile strength of the unidirectional layers is only 2077 psi and the stress generated by this loading is 580 psi. The CSM layers have a strength in this direction of 17 ksi. Longitudinal tensile strength of the unidirectional layers is 100 ksi vs. 17 ksi for the CSM layers. Therefore, the factor of safety depends on the definition of laminate failure. Changing the parameters would greatly improve the safety factor.

The addition of the polymer concrete wear surface for the VI.A.2 analysis improves the *L/d* ratio by 35% to 800 vs. 500 for the V.A.2 loading. The tensile stress in the face is reduced by 8.5%, with a comparable increase in safety.

Comparison of VII to V and VI

Moving material from the top face to the bottom face improves stress values in the lower face by 15% (VII.A.2 vs. VI.A.2) and improves deflections by 8.7% (VII.A.1 vs. VI.A.1)

Moving material from the top face to the bottom transfers stresses by comparable percentages between the two faces. Safety factors are more closely aligned (VII.B.2 vs. V.A.2).

There is a minimal increase in deflections (2%). The *L/d* ratio continues to be greater than 500 (VII.B vs. V.A).

Phase IX

IX.A Deflections increase by 33% vs VII.A.2. Longitudinal stresses increase marginally (< 5%).

IX.B No change in deflections vs IX.A. Longitudinal stresses decrease marginally (5%).

IX.C Decrease in deflections vs IX.B. Longitudinal stresses decrease by 14% for bottom skin.

Factors of safety for core shear failure are now in the range of 2.5 - 3.0 given the failures obtained by KSU.

Final Design

It was determined from the analysis that a 20-in. core depth with a 0.500-in. top face and 0.750-in. lower face would meet or exceed the design criteria. The final fabrication utilized a core depth of 20.5 in. This was due to constraints imposed by the core-fabricating equipment.

FABRICATION OF THE CENTER BRIDGE PANEL

General Fabrication Process

The fabrication for nearly all components of the bridge was accomplished through hand lay-up techniques. In general, the mold tooling was fabricated from readily

available materials and presented no obstacles to production. The core was fabricated on existing corrugated molds and the faces were laid up and the panels assembled on a 3m x 7.3m (10 ft. x 24 ft.) platen constructed of steel tubing and particle board. While this would not be acceptable for the production of Class A fiberglass parts, it was more than adequate for the fabrication of large-scale structural panels. Details of the fabrication processes for the major bridge components are detailed in the following sections.

While the hand lay-up method was adequate for this prototype bridge, an efficient manufacturing operation would require more sophisticated methods than those employed here. It is felt that a panel of the size described here could be assembled in one day by a well-trained crew of four. However, core output would require the use of automated equipment if it were to be done in a cost-effective manner. The railing could be produced by a pultrusion process much more efficiently than the hand lay-up methods used on this prototype. The success of this project demonstrates that sophisticated machinery would be desirable, but is not required, to produce quality parts.

Core Fabrication

The fabrication procedure used to produce the core elements is essentially a contact-molding process on a Mylar film release sheet. The honeycomb core is composed of a flat FRP sheet bonded to a corrugated FRP sheet.

The flat sheets are laid up on a Mylar film on a flat surface and Alpha/Owens-Corning 7RCP polyester resin is manually applied to chopped strand mat reinforcement using common paint rollers. Grooved metal rollers are used to remove air bubbles from the laminate. The corrugated sheets are wet in a similar fashion and then formed to a corrugated steel mold to produce the flutes. The pre-cured flat sheet is then placed on top of the wet corrugation sheet to produce a bond as the corrugation laminate cures. The core laminates consist of three layers of 1-1/2 oz. Vetrotex/Certainteed M127 chopped strand mat and 40% polyester resin by weight, producing a core web thickness of .090 in.

After cure, the combined corrugation/flat assemblies are trimmed to the proper width. This determines the core thickness of the sandwich panel. The completed strips are then trimmed to length and sanded along the ridge of the corrugations and correspondingly on the flat side to produce a consistent bond when the strips are assembled. Adhesive resin is applied to the fluted side of the strip before it is mated to the flat of an adjacent strip. A number of strips are bonded in this manner and then clamped until the adhesive cures. This assembly of strips is called a core board or log. In the case of the No-Name Creek bridge, six strips were assembled to form each board giving approximately 12-in. width.

The board is then laid into a wet laminate composed of chopped strand mat and an elastomer-modified vinyl ester resin (Alpha/Owens-Corning VE16). This laminate forms the basis of the top face of the finished bridge panel and provides a uniform, resilient bond between the upper face and the core.

Edge Frame Fabrication

All panel edges were enclosed by molded FRP structural sections composed of five layers of Brunswick Technologies CM-3205 Non-woven Bi-ply Combo Mat (see Constituent Laminate section). This produced a nominal laminate thickness of .312 in. These parts were produced in a manner similar to that previously described for the core layup. After werout but prior to gel, the carrier film and laminate are formed on a mold to create the desired shape. The closeout sections are produced as C-channels to provide either flat exterior surfaces or male/female interlocking joints for field installation.

The closeout sections for each panel are then assembled as a rectangular frame using FRP corner brackets bonded with Plexus adhesive.

Face Laminate Fabrication

A Mylar release film is placed on the lay-up platen. A steel framework corresponding to the dimensions of the bottom face is attached to the plated, forming a berm to contain the face layup. Layers of fiberglass fabric are laid within the berm and wet with resin until the desired face thickness is obtained. The previously assembled edge closeout framework is lowered onto the wet face lay-up. The core logs (with pre-skin on top) are then lowered into the frame and onto the wet face laminate. A vacuum is applied to the entire assembly to pull the core and edge frame into the wet face. After the face has gelled, the vacuum is removed.

The top face layup is fabricated on the pre-skin and framework. The face laminates are composed of Brunswick Technologies CM-3205 Non-woven Bi-ply Combo Mat and UM-1810 Unidirectional Combo Mat.

Application of the Wear Surface

After exotherm of the top face, a second berm is built around the perimeter of this face and the premixed polymer concrete wear surface of indigenous aggregate and Alpha/Owens Corning 78.609 DCPD resin is applied using the berm as a mold. The polymer concrete is creeded and fine sand is applied to the surface to eliminate resin rich areas. The concrete is allowed to cure before removal of the berm.

PROOF TESTING OF THE CENTER PANEL

After fabrication of the center panel was completed, it was hauled to the Russell County Landfill where a test bed had been constructed for the proof test. The test bed was constructed to reproduce the substructure of the bridge. Five I-beam stringers were installed between the header beams below the level of the bridge for the installation of measurement devices. Neoprene rubber bearing strips were laid on the header beams and the panel was set in place. The sheet piling was cut back to the road level and the ramps were back-filled to the sheet piling. This operation required approximately three hours.

The following morning dawned bright and clear. A crowd of fifty media and industry representatives, along with other interested or curious parties, was assembled to witness this historic event. Dr. Hugh Walker and John Held from Kansas State University's Mechanical Engineering Department had previously installed strain gauges in numerous locations in the panel; the gauges were attached to a data acquisition system. As a backup, mechanical dial indicators were installed in various locations beneath the bridge.

The landfill was chosen as a test site because the County keeps numerous mounds of aggregates and road fills at the location. There is also a scale house on site. The procedure for the test was to load dump trucks with fill dirt, and then measure the rear axle load. The trucks were driven onto the panel and positioned with the rear axles straddling the mid-point of the panel.

Deflection easurements were taken. The data obtained from the strain gauges was difficult to analyze immediately due to some interference with calibration of the instruments. However, dial-indicator readings were presented to the crowd. The strain-gauge readings were cleaned up at KSU and the results are presented graphically in Figures 6 and 7. One end of the panel was not settled under the lighter loads. It was noticed after placement that there was a longitudinal twist to the panel as it rested on the headers. The panel settled under the heavier loads as can be seen from the symmetry of the curve at higher loads.

It is evident from the higher end curves of Figure 7 that a certain amount of anticlastic panel deformation comes into play. This would be compensated for, to some extent, by load sharing between panels when the entire bridge is assembled. It is also obvious that there is a differential from side to side on the panel. This may be accounted for by the torsional twist showing up as uneven settling.

The maximum center point deflection was measured at .442 in. under the 60580 lb. load. This load represents a 100% overload of the bridge. The design loading is 3750 lb. per foot of width or 30 kip on the 8 ft. panel width. Extrapolating the deflection at this load gives an *L/d* of 255 in./.218 in. or 1165, well in excess of that desired. It was expected that an *L/d* of 800 would be sufficient to maintain safe stress levels.

FABRICATION OF THE REMAINING PANELS AND PIECES

The remaining panels were fabricated in the manner of the center panel as described above. In addition, the exterior panels required installation of the vehicular railing. The railing and attendant pieces are described below.

Vehicular Railing Fabrication

Before assembly of the frames for the two exterior panels, the outer closeouts were cut out to accept pre-fabricated pockets for the guardrail posts. These pockets were fabricated of eight layers of CM-3205 to give a nominal thickness of .500 in. The pocket flanges were bonded to the inside of the closeout with the post socket protruding to the outside. The flanges are overlaid to seal the interior of the panel and provide additional bond strength for the pocket and closeout.

The vehicular railing laminate was constructed of +-90° roving stitched to .5oz/ $ft.^2$ chopped strand mat. A gelcoat resin was used as the matrix to provide color to the laminate. The laminate was first laid up on a Mylar sheet, then transferred to a mold. The mold was a section of conventional galvanized steel W-rail. The railings were trimmed and finished before being installed on the bridge panels.

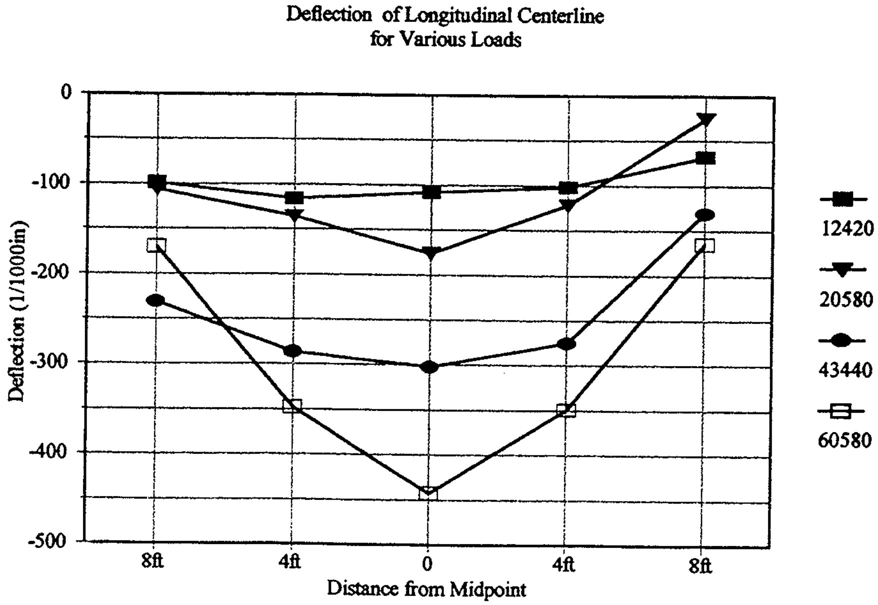

FIGURE 6. Panel Longitudinal Deflection

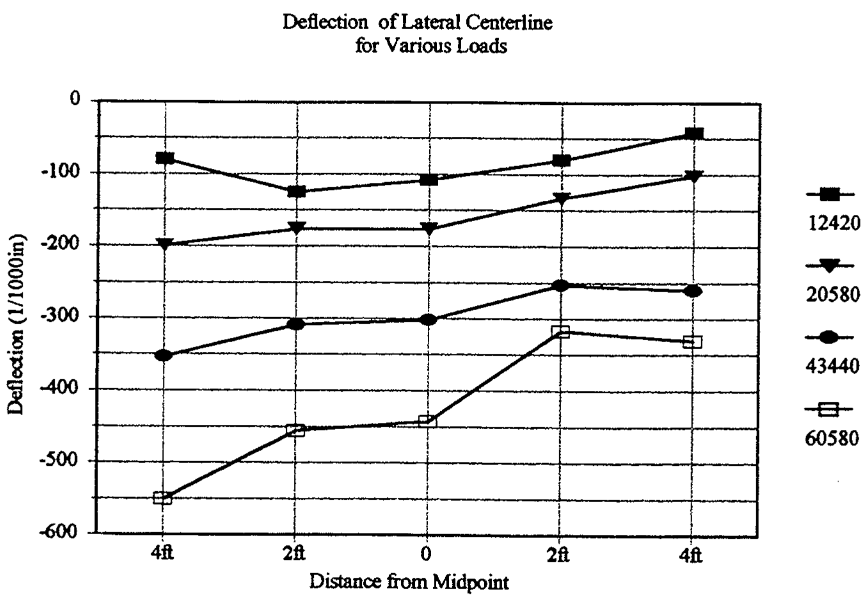

FIGURE 7. Panel Lateral Deflection

DEPLOYMENT AND INSTALLATION

Factory Preparations for Deployment

One of the concepts that was to be proven by the No-Name Creek Bridge project was KSCI's ability to pre-fabricate a just-in-time panel. Consequently, as many elements of the bridge as possible were assembled at the factory to limit the amount of field work required

at the time of installation. Both of the exterior panels were factory-fitted with guard rails. The guardrail posts were inserted into the sockets of the edge close-outs and retained with one-inch solid pultruded dowels through the walls of each socket and the web of the post. The dowels were then secured with VE16 vinyl ester resin. The posts, the synthetic wood standoff blocks, and FRP W-rail were drilled to accept one-inch FRP threaded studs which were secured with FRP nuts. This procedure eliminated the need to install railings at the site.

All three panels were loaded on a single-flat bed semi-trailer using an oil-field variety boom truck. The panels were stacked such that the center panel would be the first removed and, therefore, the first installed at the site.

Field Installation

At the site, the trucks were positioned behind the crane. A 1/4-inch neoprene rubber pad was laid on the pier headers prior to placement of the panels. The lift eyes were installed in the center panel and the panel swung into place. Placement of this first panel determined the centerline of the structure.

As this was the first attempt at installation, the two exterior panels were also placed on the headers to check the fit of the panels and the joints, but were not yet bonded to the center section. It was found that one of the exterior panels had been built with a slightly extended lower face. This panel was removed and the face trimmed to the proper dimension. Meanwhile, the other exterior panel was positioned to leave approximately 18 inches of gap at the joint in preparation for bonding to the center section.

A primary bond was achieved by applying a wet laminate of 4-1/2 oz/ft.^{2} of M127 chopped strand mat and VE16 vinyl ester resin to the lap joint flange on the bottom of the center section. The panel was then lifted and the joint was pulled together. To avoid scraping the wet laminate from the lap joint flange, the panel was suspended to hang with a five degree list. Chains were strung between the lift eyes of the center panel and the exterior panel. The panel was pulled into place with chain tensioners until the joint was firm; then the panel was lowered onto the header.

The upper side of the joint was overlaid with alternating layers of CSM and stitched roving to produce a laminate thickness of approximately 0.500 in. After this laminate had cured, the joint was filled with polymer concrete to match the level of the wear surface.

The Russell County Road and Bridge Department back-filled the roadway to the level of the bridge and attached the ends of the railings to posts sunk in the roadway berm. The first vehicle to set tires on the bridge was a road grader driven by a Russell County employee.

The entire installation was accomplished in approximately ten hours spread over a two-day period. The operation began at 1:00pm on november 7, 1996. That afternoon, the center panel was set and the exterior panels were checked for fit. The one ill-fitting exterior panel was trimmed on the morning of the 8th while the other panel was set. Both lap joints had been laminated by noon. The joints were filled that afternoon. The concrete cured more slowly than had been anticipated due to cold weather. The concrete was allowed to cure overnight and the road was opened on the morning of the ninth. It is felt that experienced workers could have installed the bridge in less than eight hours.

POST-INSTALLATION TESTING

The Mechanical Engineering Department of Kansas State University was tapped to test and monitor the bridge for the next two years.

Field Deflection Measurements of the No Name Creek FRP Bridge

The No-Name Creek FRP bridge in Russell County, Kansas was tested on November 19, 1996. The test consisted of parking the 40 kip rear axles of fully loaded dump trucks on the bridge and measuring the deflection at mid-span. The two-lane, 27-ft.-wide bridge has a clear span of 21 ft. and 3 in. and is constructed of three fiberglass sandwich panels measuring 23 ft. 3 in. long and 9 ft. wide. Deflection data was taken at five points along the mid-span with mechanical dial indicators. The weather was sunny. The temperature at the 2:00PM test was 50°F.

The dial indicators were placed on the mid-span at the north and south edges, at the centerline, and at the joint panels. The dial indicators were zeroed when installed. Zero-load deflection readings were recorded just prior to the parking of the first truck. The first truck, with a weight of 41,900 lbs. on the rear tandem axles, was parked facing east in the south lane with the tandem axles centered over the mid span. The outside edge of the tires was 50 in. from the south edge of the bridge. Deflection measurements were recorded.

The second truck, with a tandem axle load of 42,580 lbs., was parked in the north lane facing west. Deflections were recorded with both trucks on the

bridge. The first truck was then removed from the bridge and deflection data for the second truck alone was recorded. The second truck was then removed and zero-load deflections were again recorded.

The deflections reported here from north to south along the mid-span have been corrected for the initial non-zero readings of the dial indicators. As the bridge's upper surface warms in the sun, the span bows upward. During the time between noon and 1:30PM the bridge moved upward a much as .090 in. at mid-span.

The larger deflections on the north edge can be partially explained by the placement of the second truck closer to the edge of the bridge. This truck also had a slightly higher load. The south panel does seem to be slightly stiffer. This is not an indication of a problem.

The maximum deflection of .181 in. with an applied load of 85 kip yields a very respectable span/deflection ratio of 1450.

TABLE 6. Bridge Deflections from Post Installation Test

Total Load	Lane Applied	N Edge	North Joint	Center	South Joint	S Edge
0		0.000	0.000	0.000	0.000	0.000
41900	S	-0.015	0.111	0.090	0.120	0.103
84480	N-S	0.145	0.162	0.181	0.143	0.109
42580	N	0.137	0.057	0.099	0.056	0.007
0		-0.011	0.004	0.011	0.002	-0.003

Long-Term Monitoring

The long-term deflection data show no trend at this time in the test program. There is a difference between the data taken on 11/19/96 and the two most recent tests on 2/20/97 and 3/13/97, but this can be explained by differences in weather and temperature on the days that data were taken. The first data set was taken on a sunny, 50° day, and the later two sets were taken on overcast, 40° days. The final set was taken on a sunny 70° day. The middle two sets of data are within .006 in. of each other.

When comparing the earliest and latest data sets with the middle sets, the bridge had dropped .050 in. (.068 in. measurement difference -.015 in. of rubber pad settling) at the mid span. It is believed that the explanation for the difference can be found in the weather at the times of the test. In the past, it has been noticed that the bridge bows upward when the surface is warmed by the sun. This is due to differential thermal expansion between the upper and lower faces of the panels.

The measurement locations are equally spaced along the bridge centerline with locations numbered from east to west. The measurements are inches from the test jig beam to the bottom of the bridge panels as measured with an inside micrometer.

TABLE 7. Long Term Deflection Data

	Test Date			
Location	4/3/97	3/13/97	2/20/97	11/19/96
1	6.559	5.512	6.515	6.552
2	6.473	6.555	6.552	6.609
3	6.560	6.456	6.462	6.530
4	6.650	6.380	6.383	6.451
5	6.585	6.499	6.502	6.548

RESULTS VS. GOALS

The stiffness of the actual panel exceeded that of the design by 56%. This is discouraging from a design standpoint in that it represents a great waste of material.

There are a number of reasons for the discrepancy. Firstly, the model was based on a span of 279 in. vs. a free span of 255 in. for the actual bridge. Secondly, the actual construction of the bridge used a core depth of 20.5 in. vs. the design depth of 20.0 in. The

incorporation of these two differences in the design would produce a 27% decrease in deflection. Thirdly, the narrow width of the tested panel meant that the actual wheel loads were placed on the stiffer edges rather than having a more typical placement near the center of the panel which may account for some of the discrepancy seen at the center point. The stiffness contribution of the edge closeouts was an estimate that may not have been adequate. It is heartening, however, that the panel was stiffer rather than softer. The original design was conservative in the interest of safety and from lack of knowledge of the performance of the composite construction.

A subsequent reformulation of the model was tested against data from a load test performed on the installed bridge and the deflections agreed to within 1%. This may be a serendipitous result given the complexity of the problem, but it bodes well for future design work. The complexity of the problem is not only in the use of composite materials themselves, but in the composite nature of the panel construction. Composite materials are highly anisotropic and a large number of properties must be determined for each laminate in a construction if it is to be successfully modeled. The construction itself is more complex than even reinforced concrete in that it is not homogenous in any section and is, therefore, itself highly anisotropic even if constructed with materials having isotropic properties. KSCI continues to acquire data and knowledge to improve its design capabilities.

Fabrication of the panel was not as efficient as was originally hoped. Major problems were variability of climactic conditions and a lack of trained workers. However, climactic conditions can be overcome in a properly designed plant and skilled workers can be trained. Many improvements can be made to the manufacturing process even though the methods used were manual. The original fabrication plan was modified several times because of a lack of proper equipment. There were also some problems beyond the control of KSCI. However, the project was accomplished with a minimum of actual manufacturing difficulties once the problems had been solved. Given the fact that this was a prototype construction of a panel of a size not previously constructed, the fabrication went very smoothly. Efficiencies improved with each subsequent panel and it is felt that this bridge could be constructed in one-half to one-third of the time required for the prototype even using the current manual methods.

The goal of installing the bridge rapidly was met beyond the expectations of those involved. The complete installation was accomplished in under two days and, except for minor difficulties with the size of one panel and the final sealing of the joints, was finished without difficulty. This proves the viability of the rapid installation goal.

PLANS FOR IMPLEMENTATION

OVERVIEW

The implementation plan for a dramatic new product, such as heavy-duty FRPH panels for bridge and bridge deck applications, must involve an effort of several years. The market will grow slowly at first for two major reasons: first, because of the novelty of FRP materials to the civil engineering community. FRP bridges are a concept sale, that is, the user must be convinced that the idea is sound prior to considering the pros and cons of a specific product. Concept sales are the most difficult as only a few potential users will take what they perceive to be a significant risk. This is particularly true of the civil engineering community, given that public safety is the major consideration of the profession. Secondly, the initial cost of FRP bridges is ten to fifteen percent higher than conventional structures. Tight budgets will prevent many potential customers from considering FRP bridges as a viable alternative, even if they are sold on the concept.

Offsetting these real and perceived disadvantages are the demonstrable advantages of rapid installation and delivery. Unfortunately, most bridge engineers think of emergency replacement bridges as short-lived, temporary structures. The concept of a rapidly constructed, permanent bridge is a contradiction in terms to most engineers. However, civil engineers are fully aware of increasing public impatience with time-consuming bridge and road repair.

Based on these factors, it is clear that all types of information channels must be used to reach the public at large, but also to reach the civil engineers who are favorably disposed to innovation. These engineers are difficult to identify; therefore, the attempt must be made to educate the engineering community as a whole so that the identification process becomes self-induced.

Implementation also requires a considerable effort to be made in cost reduction. As costs approach those of conventional products, sales volume should increase.

In the early stages of implementation, specific market niches must be identified and projects begun to demonstrate the applicability of the product to these areas. These projects must be completed successfully if acceptance is to be realized.

The final aspect of the implementation plan involves working with the composites industry, particularly materials suppliers, in order to gain their support. This would certainly enhance KSCI's efforts in the implementation of its products. These efforts have already begun.

MARKETING PLAN

The original KSCI marketing plan became obsolete after the company won the Counterpoise Grand Design Award sponsored by Owens-Corning and presented by the Composites Institute.

Furthermore, large number of publications have published articles about KSCI and the bridge. KSCI plans to release a public media campaign to begin in June after the bridge has been in place for six months and more results are obtained from Kansas State University's continuing monitoring program.

KSCI has responded to nine inquiries from various state Departments of Transportation seeking further information. At least three of these departments have mentioned specific projects where they feel KSCI's product might be applicable. Three states have asked KSCI to give presentations at their annual bridge conferences. Recent inquiries regarding presentations have also come from local or regional groups of the American Society of Civil Engineers. The crusade to educate the engineering community at large appears to be having some success and will continue.

In summary, the company's marketing plan will initially involve presentations to professional groups combined with a broad-based public relations effort focusing on the ease of installation and rapid deployment of the short-span composite bridge in emergency situations.

ADVANCING THE MANUFACTURING PROCESS

Lowering manufacturing costs is the key to increasing future sales in areas other than emergency bridge replacement. To this end, KSCI has begun to install machinery at its Russell, Kansas facility to produce honeycomb core by an automated process. This one step alone, involving expenditures of $400,000, will reduce production costs by approximately 25%. This process to increase capacity and reduce costs will continue over the next three years.

Current production volume is adequate to produce one bridge per month, which should be adequate for prototype projects at an acceptable cost.

CONCLUSIONS

The major and most significant conclusion to be reached regarding the application of FRPH panels as rapid replacement bridges is that technical feasibility and utility have been established by this project. Through a program that involved a combination of materials testing, analysis, proof testing, and manufacture, it has been shown that FRPH has passed the initial qualification phase as a structural material for bridges.

Two issues remain to be fully resolved: the life span of the structure, and the economics of FRPH bridges and decks. The life expectancy of the bridge can only be determined through continued monitoring. The present design utilizes as little as 5% of the ultimate strength of the FRP materials involved. This may be a waste of material, but given that the long-term effects of cyclic fatigue and inelastic deformation (creep) are not known, the current design may be more appropriate than an initial glance might suggest. Future testing of this concept will determine if this is the case.

The issue of economic viability has not been fully established by this project, but the results, while not definitive, are encouraging. The conservative design for deflection resulted in using thirty to forty percent more material than that required for a bridge with more experience behind it. Also, the fabrication costs of this bridge are atypical because of tooling costs and a steep learning curve associated with any prototype product. The manual methods used to construct the bridge are particularly time consuming in many areas and will benefit greatly from any automated or mechanical evolution in the future. Optimization of both materials use and manufacturing efficiency can now begin.

While other FRP production methods may become competitive, none has shown the initial promise of FRPH at such an early stage of development.

KSCI is currently working with KDOT's research department to develop an FRPH sandwich panel to replace the deteriorating steel grate decks on three truss bridges in southeastern Kansas. Each bridge is approximately 45 ft. long and 32 ft. wide. These projects will be completed in 1997 or 1998.

INVESTIGATORS

The principal investigator for the NNC Bridge project is Dr. Jerry D. Plunkett, KSCI President and CEO. Dr. Plunkett holds a doctorate from MIT (1960) granted by the Metallurgy Department with an option in Ceramics and a minor in Industrial Management. He has over forty years experience in the R&D field. From 1975 to 1984, he served as Managing Director of the Montana Energy R&D Institute, which he built from one employee to 450 people. From 1984 to 1990, he served as Vice Chancellor for Research and Technology at the University of Denver. His major contribution has been in contributing to and, in many cases, leading projects that have resulted in the introduction of approximately fifty products to the marketplace. He has also played a major role in developing 20 industrial processes for producing various materials.

Mr. Hoback, co-principal investigator, is the Russell County Road and Bridge Superintendent. He is a hands-on manager and visits all his projects on a daily basis. His experience of over 20 years as a supervisor in the construction of roads and bridges gives him the expertise and capability to deal with innovative projects such as this one. Mr. Hoback has worked over 20 years in the construction of highways, bridges, buildings, and refineries in Kansas, Oklahoma, Texas, New Mexico, and California.