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Highway IDEA Program

Testing, Evaluation, and Installation of Fiber-Reinforced Polymer Honeycomb Composite Panels in Bridge Deck Applications

Final Report for Highway IDEA Project 46

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ABSTRACT

This report documents the progress of a project to design, fabricate, and install a fiber-reinforced polymer honeycomb sandwich panel system for use as a superstructure on new or existing vehicular bridge structures; the basic premise being to develop a vehicular bridge decking system, using technologically advanced materials, that was long lasting, economical in both initial cost and long-term use, and required minimal alterations in current installation techniques by outside construction contractors. The general procedure was to determine basic engineering properties of the proposed materials, provide an optimized design through computer analysis of the deck and structure, then manufacture and install the final product. The resulting design has successfully addressed the premise. Monitoring of the structures to determine the long-term viability of the design continues. There are a number of areas where further study would prove useful.

CHAPTER 1 - INTRODUCTION AND RESEARCH APPROACH

This TRB/IDEA project was part of a larger effort initiated by Kansas DOT to re-deck two bridges in the southeastern part of the state. The KDOT project was an offshoot of the successful conclusion of a previous IDEA project, *Fiber-Reinforced Polymer Honeycomb Short Span Bridge for Rapid Replacement* (NCHRP-IDEA Project 30), involving an installation over No-Name Creek (NNC) near Russell, Kansas in late 1996. The NNC project was brought to the attention of Kansas DOT and it was thought that there were possible applications of the KSCI technology to other structures in the State.

In 1997, it was proposed to Kansas Structural Composites (KSCI) by the Kansas Department of Transportation Materials and Research Division (KDOT-M&R) to replace the existing asphalt-over-corrugated-steel superstructures of two bridges on Kansas Highway 126, located west of Pittsburg, Kansas, with sandwich panels fabricated from fiber-reinforced polymer honeycomb.

The objective of the KDOT project was to develop an adaptable fiber-reinforced polymer (FRP) decking system for replacement of existing superstructures on vehicular bridges that no longer met current highway load and lane width standards. Bridges in this category may be found throughout the United States, the estimated number being 150,000.

The basic problem in the rehabilitation of these structures is one of cost and time; the creation of a completely new bridge is an undertaking that causes great inconvenience to the general public and requires many hours of design time. Many bridges deemed deficient are still structurally sound, but have deteriorating superstructures which require replacement. The existing roadway may also have accumulated numerous layers of overlay during its lifetime, creating a condition of excessive dead load on the structure necessitating the re-rating of the bridge to lower weight limits, forcing large vehicles to detour around the area; this is a particular problem in rural areas where alternate routes may be unavailable. In addition, current highway standards may require greater lane width. The policy in Kansas is to bring any bridge under rehabilitation up to current weight and lane width standards. In the case of the two bridges involved, this required the removal of approximately 10in of asphalt overlay to eliminate excessive dead load, along with widening of the road surface from 26ft to 32ft.

It had been suggested that, though the NNC bridge had proved the viability of composite honeycomb sandwich construction, the application of this technology would not appear to be economically viable to local governments where limited budgets and low traffic volumes would preclude the expense of FRP construction. A more economical approach would be the combination of the FRPH product with conventional bridge structures. Many bridges, especially on local roads, are structurally sound, though the roadway superstructure may have deteriorated or subsequent roadway maintenance has increased the dead load to the point where it adversely affects the carrying capacity of the structure. In these cases the re-decking approach proves attractive when compared with the time and expense of building a completely new structure.

The project posed a number of interesting problems that were not applicable to the No-Name Creek project. There was a requirement to provide a means of draining precipitation from the surface which had been a minor concern on the NNC bridge. It was hoped to be able to accomplish this without the use of additional overlay to provide deck camber. Secondly, as previously mentioned, the existing roadway had been overlaid several times with asphalt. It was obvious from the number of support beams that a relatively thin deck would provide adequate strength, and the beams themselves could not be raised due to the original method of construction; therefore, an economical design would require raising the deck to the grade with a minimal amount of FRP material. Thirdly, a method had to be devised to attach the deck to the structure without modifying the stringers.

As part of the design phase, a more rigorous materials testing program was initiated. Post-analysis and testing of the NNC bridge had shown that the bridge could have been built to acceptable strength and stiffness levels with 60% of the material that was used. The base cost of FRP materials is high when compared with more conventional materials such as concrete; while there are mitigating characteristics of FRP versus conventional materials, such as lower stiffness-to-weight ratios, greater longevity, and various qualitative considerations regarding public inconvenience costs, the cost of an FRP deck is currently twice that of typical concrete construction.

It was determined that a testing program should be developed that would facilitate the design process by determining materials properties that could then be implemented into a computer-aided design program, thereby assisting in optimizing the design of the deck panels. Test samples would be fabricated, then tested at the Mechanical Engineering Department of Kansas State University (KSU). The resulting data would be applied to a finite element analysis model, and the results of the physical testing would then be compared with the analytical results to determine a baseline for future analysis of various FRPH sections.

Fabrication of the deck was to follow the same general procedures developed for the NNC bridge with some minor modification. Some pultruded FRP materials for panel edge close-outs and joints were incorporated into the design.

As with the NNC bridge, one objective was to achieve rapid installation of the decks by a work force with limited technological capacity and with a minimal amount of equipment; the goal being to offset the higher cost of the FRP material with reduced installation costs. The design attempted to incorporate details that would facilitate installation.

CHAPTER 2 - FINDINGS

INTRODUCTION

Testing for the KDOT project was performed by Kansas State University (KSU) on samples provided by Kansas Structural Composites (KSCI) to provide data to be used in the design of the two FRP composite sandwich bridge decks to be installed in Crawford County, Kansas. Data was required to determine panel stiffness, materials properties, and failure limits of the proposed design. Also, testing of the constituent laminates used in construction was performed. Additional testing was performed to determine the resistance of various thicknesses of wear surface overlay to punch-through and of the panel's resistance to crush.

The results of specimen tests were applied to a finite element analysis program to determine and appropriate core thickness, then to provide an idea of the behavior of the deck in conjunction with the existing structure.

The deck was fabricated at KSCI, then installed.

TESTING

Constituent Laminate Tests

Sample laminates were fabricated to determine the materials properties of the face laminates in the panels and test beams for use in design analysis procedures. Materials data and weights can be found in Appendix A. All samples were tested by KSU.

Analysis

Graphical results for the tests can be found in Appendix A. Not all the results were useful or believable. The UNI-L1 sample shows a very low modulus and strength, leading to the conclusion that there may have been slippage of the clamps during testing. The other two UNI-L samples exhibit more consistent behavior. The average of the two moduli is 2.1Msi, which matches that obtained in previous tests of similar laminates.

The UNI-W results pose a more interesting dilemma. The modulus for the UNI-W1 sample is obviously too great for the constituent materials involved. The maximum stress is also somewhat high given the low level of reinforcement in the tested direction. The results for UNI-W3 are somewhat too low, in particular the stress number. It might be expected that the UNI-W2 results would be 'just right'; however, the modulus would seem high for the aforementioned reason. Additional samples would have been useful. The samples provided to KSU were hand laid sheets of approximately 12in by 12in. The sheets were cut into dog-bone shapes at KSU. This has proven to be unnecessary according to most composite testing laboratories. The current practice is to cut rectangular strips, as the necking of the dog-bone in an anisotropic material can produce high stress concentrations near the clamped ends. It would have been extremely useful to have had more samples prepared from the material provided.

Beam Bending Tests

Tests and Procedures

After testing of the face laminates, various beams were constructed in an attempt to determine what contributions to stiffness were made by the core, faces and wear surface overlay.

It was deemed desirable to determine the stiffness of the panels. Therefore, beams were constructed and tested under three-point bending over 80in and 60in spans on a constant deflection test machine. Data was acquired using a dial indicator installed between the loading head of the machine and a bar resting on the beam support points. Data was taken at various load increments, then plotted and corrected to determine the effective bending stiffness of each beam. The results given below are from tests over a 60in span.

Sample Description

Samples fabricated for these tests included beams of various constructions and constituent laminates. There were five sets of beam samples made. The basic set descriptions follow; laminate schedules and sample physical parameters are given in Appendix B.

Set-01: Beams constructed of core whose web laminates were fabricated from 3oz/ft^s chopped strand mat containing approximately 35% reinforcement by weight. Faces on these beams consisted of a resin-rich 3oz chopped-strand mat layer to provide a low face modulus. It was of interest to determine (as nearly as possible) the basic properties of the core alone without the contribution of the faces to stiffness. Beams were constructed to represent both longitudinal and lateral panel sections.

Set-02: Beams were constructed of core with a reinforcement content of 40%. These beams were also constructed with full-thickness faces as proposed by the design. Only a longitudinal beam was constructed for comparison of the difference in core laminates with beams in Set-03.

Set-03: Full faces were applied to the Set-01 samples to compare the contribution to stiffness of the faces and core.

Set-04: A 1/2-inch polymer concrete wear surface was applied to the Set-02 beam to determine the contribution of the wear surface to overall panel stiffness.

CM4545: After examination of the data, an additional pair of beams (longitudinal and lateral) was fabricated along the lines of Set-02 to represent the final panel design. The core for these beams was of 4.5oz chopped strand mat at 40% reinforcement. These beams were subjected to three-point bending tests over 60in and 42in spans to determine mechanical properties. In addition, these beams were tested to failure over a 24in span to determine ultimate strength and deflection at failure in a situation that closely represented the final installed configuration. They were further subjected to crushing tests to determine the resistance of the panel to concentrated loads, both in damaged and undamaged areas.

All beams were fabricated with 4-inch core depth. The core was fabricated in the standard geometry which consists of triangular cells with bases of 4in and heights of 2in. All structural face laminates were of identical construction. Laminate schedules and testing matrices are included in Appendix B.

Results

Rigidity was determined by solving the classical beam deflection in three-point loading for stiffness, then inserting the values for load and corrected deflection from the data. The total beam rigidity was then divided by the beam width to obtain a value for rigidity per inch of width.

Set-01: These beams were fabricated with faces representing the bonding layer only. Loading tests of the bare core showed that deflection is mainly caused by buckling of the webs. Load bearing comparisons were difficult due to the creep buckling of the webs during loading. Stabilizing the webs with a bonding layer of chopped strand mat provides an isotropic face of limited stiffness, allowing comparison of the L- and W-direction core properties.

It can be seen from Table 1 that there is a ratio in stiffness per inch of 1.3 between the L- and W-directions. From an analysis of the geometry, it would be expected that the ratio for the core alone would be 3, given the orientation and spacing of the webs. If this ratio is assumed to be correct, calculation shows that the contributions of the face is slightly more than 65% of the overall stiffness for the L-direction beam, or 7.52×10^5 lb-in. This calculates to a core stiffness of 3.98×10^5lb-in in the L-direction and 1.33×10^5lb-in in the W-direction.

Set-02: The calculated rigidity for the Set-02 beam is given in Table 2.

Set-03: The application of full face laminates to the Set-01 beams produced the results in Table 3 below. The L-03 beams were 8 times stiffer than the L-01 beam. The W-03 beam was 6 times stiffer. This difference is attributable to the greater stiffness of the face laminate in the L-direction compared with the W-direction. From the results of the Set-01 tests, the core contributes 4.3% of the overall beam stiffness in the L-direction.

Comparison of the L-03 beam to L-02 shows a ratio of 1.15. The ratio of face thickness between the two beams was 1.12, so the difference in stiffness is most likely attributable to this fact, and the difference in core web thickness would seem to be negligible.

Set-04: The application of the .50in wear surface to the Set-02 beam gave the results shown in Table 4. This is a 12.5% increase over the stiffness of the Set-02 beam. Some of this increase can be attributed to the increase in moment of inertia of the section with the addition of the wear material.

CM4545: These beams were tested twice at the two spans of 60in and 42in. They were then loaded on a 24in span to failure. Values in the table are the average of the two tests over the 60in span.

Punch-Through Tests

Punch-through testing was done on sections of the previously tested RLX-02 beam over-laid with increasing thicknesses of polymer concrete wear material. Two diameters of punch were used and the data compared. Results are given in the tables below. Additional data is given in Appendix C.

The results are given graphically in Figure 1, and show progressively higher failure loads as concrete thickness is increased. The only aberration is in the test of the 3/4in overlay with the larger diameter punch. This discrepancy was reported to be due to the location of the punch near the edge of the panel. Higher loads were seen for the larger diameter punch, though failure occurred at lower pressures. This result is an anomaly. It would be expected that the greater load sharing provided by the core would lead to higher strength.

It could be wished that more data had been collected so that a statistically meaningful analysis of the data could be performed. The anomalies might be explained by whether or not the load was applied over a core web or over the open cell. In the first case, the web would transfer load. In the second, the deflection of the face would stress the bond line and perhaps cause a moment to be applied to the web, forcing premature buckling. The vertical shear stress at failure for the larger punch is relatively consistent for all loads (with the exception of the premature failure of the thickest face). This might be explained by the large punch more nearly matching the effective radius of the cell, and therefore having more consistent web support than the smaller sized punch. It was observed that the holes were almost exclusively cylindrical through the concrete.

Additional punch-through tests were performed at KSCI in an effort to qualitatively determine the mode of failure. The tests were performed using a 1in diameter steel punch with load applied on a hydraulic hand press. The tests were performed on another section of the RLX-02 beam, without any wear surface applied.

The load was applied in areas where there was core support and areas near the center of cells. Initial failure was recognized from acoustic emissions. The initial failure in all cases was buckling and crushing of the core webs near the upper bond line. This was deduced through the absence of any noticeable damage to the face laminate and also from observations of the visible bond areas near the edge of the panel. This failure was localized within an inch of the load area. As the load was increased, the core continued to crush, but adjacent areas of the web deformed without failure. The faces dimpled under the load and finally failed through inter-laminar de-bonding from bending stresses and vertical shear. In addition to the de-bonding, the upper lamina fractured along lines parallel to the fiber orientations. Fractures appeared to be longest in the direction of the primary face reinforcement, i.e. the principle direction of the underlying uni-directional layers. Fractures perpendicular to this were less pronounced. This was to be expected given the lack of reinforcement normal to these lines for resisting shear forces. The upper lamina also failed in vertical shear; however, the supporting laminae continued to deform under bending after the upper layer had delaminated and sheared through.

Observations of the upper layer lead to the conclusion that this layer is failing due to in-plane shearing forces. The critical damage is due to crushing failure of the core. One would expect that the inclusion of a wear surface would spread the load to a larger area of web support and also provide some stress relief to the uppermost structural lamina. The wear surface itself tends to fail in shear and produces a cylindrical hole. When this shear failure occurs, the load is then taken completely by the face laminate, and final panel failure would occur in a manner similar to that described above. The resistance to failure is most likely determined by the thickness of the overlay.

Overall, damage to the panel is highly localized. The radius of damage to the face does not exceed 1.5in, even with total shear failure around the load. The area of damage beyond the circumference of the punch is primarily inter-laminar de-bonding, with little or no fiber breakage. It can be assumed that the overall strength and stiffness of the panel is minimally affected by punch-through failure. It would be possible to repair the face in such an area with minimal effort. Repair of the core would be more difficult, but the load sharing capability of the panel construction and small damage area would make this unnecessary in most cases. The resilience of the face laminates allows large deformation without failure (as can be seen in the above KSU data) and subsequent localized loading would produce, at most, the same effect. From the observations, the core begins to fail under a relatively low load and failure of the face requires much additional effort.

Crush Tests

Crush tests were performed on the CM4545W-05 beam. The test involved pushing a 6x6 inch steel plate into the face of the 1ft by 7ft beam. Three tests were done; two in the previously damaged areas, where the beam had failed under bending, and one in an undamaged portion. The maximum loads sustained were 78.6kip, 74.6kip, and 88.8kip respectively. The loads drop rapidly after initial failure to 30kip and level off at this value as the core compresses. Pressures at initial failure were 2.2ksi, 2.1ksi, and 2.5ksi, well in excess of what would be experienced in service. The pressure after failure was 830psi. This is the load that could be sustained by damaged core.

The earlier shear damage had little effect (15%) on the crush strength. These numbers are quite large enough to support a tire load without any chance of failure.

Fatigue Tests

Fatigue testing was performed at KSU in mid-1998. A number of beams were tested with varying results. The KSU report will be found in Appendix D.

The beam specimens had been previously tested for stiffness, though not to failure. This may explain the variation in the results. The stiffness testing was generally performed until the first major audio signal was acquired. The cause of the first audible signal was difficult to pinpoint; therefore, the subsequent failure during fatigue testing may have had a number of different causes. As the failure mode was not noted in the test report, it can only be assumed that failure was due to de-bonding of the upper face from the core. This has been the normal mode of failure observed in previous experiments.

DESIGN

Design of the panels and ancillary hardware began after completion of the testing phase. Design of the deck panels themselves was a relatively straightforward process involving the application of the material properties derived from the testing to a finite element model. It was determined that a 0.375in face laminate was most economical when punch-through characteristics were considered. Various core thicknesses were analyzed using these faces to determine an appropriate panel stiffness given the support beam spacing of the existing structures. It was determined that a four inch core depth would provide a factor of safety of greater than 10 against ultimate strength. The design analysis report can be found in Appendix E.

In addition, a method of accommodating the panel camber and raising the panels to match the existing roadway was devised. A system of FRP honeycomb beams, termed 'saddles' was devised. These ran the full length of the bridge, straddling the flanges of the stringers. Neoprene rubber was attached to the bottom of the saddles to provide a wear surface between the FRP and the steel.

There was also the necessity of designing an anchoring mechanism. This consisted of a clamp that captured the flange of the steel support beams, bolted through the joints between panels. The design theoretically allowed all work to be done from the upper surface of the deck, without the necessity of installing temporary scaffolding. A detail of this system, with the saddles, is found in Appendix E.

FABRICATION

Beginning in early 1999, fabrication of the deck panels began. The procedures used were essentially those used in the manufacture of the No-Name Creek bridge done under a previous TRB/IDEA grant. KSCI's manufacturing process is a manual procedure involving open mold laminating techniques. The general procedure is to manufacture the core, then assemble the panels with a wet lay-up process. Pultruded FRP pieces are used for the edge closeouts. Final panel parameters are given in Appendix F.

The decks were manufactured as flat, single-lane panels of 8ft by 16ft. Two of these panels were then joined to produce a full-width panel of 8ft by 32ft. The panels were attached so that there was a camber of approximately 1.5° from the centerline to provide precipitation drainage.

All FRP fabrication was completed in the Summer of 1999.

Fabrication of the steel anchor brackets was completed in the Fall of 1999.

INSTALLATION

The decks were installed in late October and early November of 1999.

Inclement weather in southeast Kansas during the spring and summer, along with heavy construction schedules for local contractors, delayed the preparatory field work for the project considerably. Earth work by the contractors commenced on September 27, 1999 and proceeded rapidly. The deck was delivered to the site at Lightning Creek on a single flatbed trailer on Tuesday, 26Oct. Installation began the next day at 8AM.

Setting of the first end panel and the first main panel required approximately 4hrs; subsequent panels required about 1hr each. By the end of the day, all the main panels were bolted down with only the final end panel to do the next morning.

On Thursday morning the contractor installed the final panel and the end anchor bar. The joints were grouted and the installation was essentially finished by noon. Overall, the first installation went very well. The laborers from the contractor learned the procedures very quickly.

On Friday morning, the KDOT people in Pittsburg asked that we modify the clamps of the anchor system as they were not satisfied with the amount of purchase of clamp brackets on the I-beam flanges. This necessitated adding material to the foot of the clamp bar, which was accomplished partially in Russell, and partially on-site during the second installation.

The second deck was installed at Limestone Creek, beginning on 3Nov and proceeded almost identically to the first. The procedure had seen some small refinement from the first installation and had become more automatic with the crew. The work progressed more rapidly, but was slowed by a high amount of tourist traffic as the project had generated some interest among the engineering school at the local university.

The installation procedures that had been worked out by the designers required little modification. The decks were installed by a crew of five using a crane to set the panels and come-a-longs to pull the panels into place. The only serious problem was with the modifications to the clamps, and this was solved in good order.

The highway was re-opened on November 24th, after completion of the approach work and installation of the guardrails. There have been no reports of problems or concerns thus far.

	Rigidity(D=EI)/in (lb-in)
CL-01	1.15×10^6
CW-01	8.81×10^5

Table 1. Stiffness Comparison of Set-01 Beams.

	Rigidity(D=EI)/in (lb-in)
CL-02	7.98×10^6

Table 2. Stiffness of Set-02 Beam.

	Rigidity(D=EI)/in (lb-in)
CL-03	9.20×10^6
CW-03	5.27×10^6

Table 3. Stiffness Comparison of Set-03 Beams.

	Rigidity(D=EI)/in (lb-in)
CL-04	8.98×10^6

Table 4. Stiffness of Set-04 Beam.

	Rigidity(D=EI)/in (lb-in)
CM4545L	8.81×10^6
CM4545W	4.78×10^6

Table 5. Stiffness Comparison of Set-05 Beams on a 60in Span.

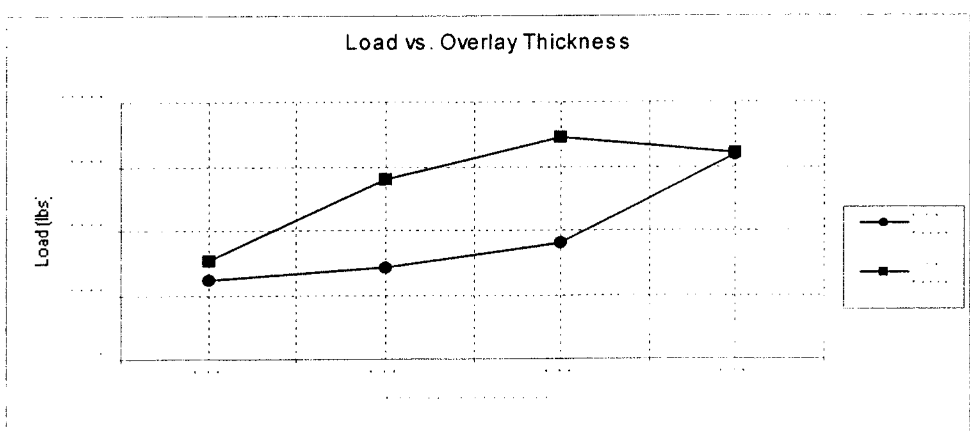

Figure 1. Punch-through; Load vs. Overlay

CHAPTER 3 - CONCLUSIONS AND SUGGESTED RESEARCH

The decking system developed has shown promise. The main objectives of the project were realized, though further work in the area of materials testing is desirable.

While some useful results were obtained, the areas of tensile properties of materials, shear behavior of FRPH sandwich construction, and a more detailed analysis of the interface area of the construction deserve more detailed investigation. There is also need to investigate the long-term behavior of such structures. While this involves monitoring of current installations over long periods, there are also existing methods of determining long-term performance through fatigue and environmental testing.

The final design of the panels and ancillary hardware proved to be the most interesting aspect of the project. The need to meet the requirements of the premises proved to be challenging and ultimately satisfying. The design of a FRPH sandwich panel to meet given structural requirements has become a relatively straight forward task. The design of such structures is still not optimal due to a lack of long-term performance data, but this structure appears to be more optimal than the No-Name Creek bridge in terms of use of material.

The ability to match the road grades at the Crawford County bridges through the use of the saddles was particularly expedient because the stringers for both bridges were encased in the concrete abutments, precluding the method of blocking the beams up to adapt to the grade. The use of saddles would probably not be economical in cases where beam shimming is possible, but proved to be expedient in Crawford county, eliminating the need to provide an over-designed deck, or the overlaying of asphalt on the deck to meet the grade, which would have defeated one of the main goals of the project: to reduce the dead load on the structure, thereby increasing live load capacity. The saddle design also provided wear protection for the deck panels by providing a carrier for the neoprene bearing pads, and also facilitated installation by providing a low-friction surface while pulling the panels into place.

The anchor system devised was an important aspect of the total design. Current bridge construction guidelines preclude the welding of brackets or drilling of support beams to provide attachment for deck panels. This is particularly important where FRP decks are concerned as conventional techniques of attachment can be difficult to apply. Some FRP deck projects have received special dispensation to allow such intrusive procedures, but the current guidelines obviously have some purpose regarding long-term safety of the structure, and the effort should be made to adapt FRP technology to their use. The KSCI clamp system is non-intrusive, and involves a minimal amount of labor and materials to implement. One goal of the clamp design also was to eliminate the need for work to be performed underneath the bridge. While this goal was not strictly reached in this project, it was demonstrated to be achievable with a little more care in application during the installation and a slightly modified design. It eliminates the need for adhesives, which are yet to be proven to have long-term viability and are environmentally undesirable.

The one consideration that needs to be addressed is that this anchorage method does not provide composite load sharing action between the deck and the structure. It is the contention of the investigators that FRP decks, because of their inherent lack of stiffness, contribute little in the way of overall structural improvement when applied to a steel or concrete support structure with a relatively high modulus; therefore, designs incorporating composite action between FRP deck and structure are unimportant. It is also felt that some current composite stiffness designs, both conventional and FRP, do not maintain their integrity in the long term. It would be well to investigate this problem further, both experimentally and analytically, to determine the actual contribution of the deck to the overall stiffness of the structure and what guidelines should be used in the application of this design technique.

Fabrication proceeded without major problems. Core production rates produced approximately enough material for $80ft^2$, or 320 bd. ft., of panel per day. This production rate needs to be increased if the cost of the panels is to be reduced. Mechanization of the entire manufacturing process would greatly reduce costs. The greatest previous quality issue was the difficulty of manufacturing panels that were consistent in thickness. A modification to the panel assembly process improved the quality of this aspect of manufacture. Finished dimensions showed much improvement over the NNC panels; measurement of the installed bridge showed length to be within 1mm of specification. While this was somewhat serendipitous, it showed that it is possible to manufacture large panels to relatively tight tolerances. The panel production rate reached one 8ft by 16ft panel per day.

Installation went more smoothly than expected. It had been an object of the project to develop a product that did not require great technical skill to construct and this goal was realized. By the time of setting the third deck panel on the first bridge, the contractor's crew had completely taken over the installation chores from the KSCI personnel. The contractors had no previous experience with construction involving FRP materials, but the design was such that most procedures were familiar to them and required little in terms of new techniques or equipment. A full report on the installation will be available from KDOT in the near future, and images of the installation are available at the KSCI web site, *www.ksci.com/crawford.html*.

The bridges will be visually monitored by local KDOT personnel. There is also a proposal to have long-term structural monitoring done by a university, though the school has not been determined as yet. This points to an area of further study that would involve the development of methods of inspection of FRP bridges in general. There will be a test performed by KDOT, tentatively set for January, 2000, to compare the flexural characteristics of the FRPH deck with the original.

There has been some national interest generated in this project by an article written by Jim Fisher in the *Kansas City Star*. The project was also featured in an installment of a Millenium-ending series on technological innovation broadcast by PBS on the *Lehrer News Hour* December 27, 1999.

Aside from the aforementioned areas of further study, it would be useful to determine the actual cost differential of FRPH decking in the price of a rehabilitation project, including ancillary construction activities.

APPENDIX A – LAMINATE TESTS

Tensile specimen laminates are described in Table A-1. The descriptions of the various reinforcements are as follows:

Bonding – 3oz/ft^2 chopped strand mat.

CM3205 – 16oz/yd^2 roving @ 0°; 16oz/yd^2 roving @ 90°; 0.5oz/ft^2 chopped strand mat.

UM1810 - 18oz/yd^2 roving @ 0°; 1.0oz/ft^2 chopped strand mat.

The strain data was corrected to provide a basis for calculating the stresses from the loads. The stress data was then subjected to a least-squares fit, then corrected to provide a basis for calculation of ultimate strength and an average modulus.

Data was analyzed in areas that gave a correlation coefficient of .98 or better. Not all strengths are true ultimate strengths, but can be used to designate points where the laminate has lost its useful properties. Corrected maximum stresses and moduli are given in Table A-2 below. The -L designation is for force applied in the principle fibre direction while -W designates force applied normal to the principal fibers. The plots in Figures A-1 and A-2 are of stress vs. strain for L and W specimens respectively.

Laminate Schedule		Weight (lbs)			Glass Percent	Thickness
No. Layers	Reinforcement	Glass	Resin	Lamina		
1	Bonding	0.188	0.417	0.605	31%	0.082
1	cm3205	0.253	0.243	0.496	51%	0.059
6	um1810	0.960	0.960	1.920	50%	0.229
1	cm3205	0.253	0.243	0.496	51%	0.059
	Totals	**1.65**	**1.86**	**3.52**	**47%**	**0.428**

Table A-1. Tensile Specimen Laminate Schedule; properties per ft^2

Sample	Max. Stress (ksi)	Corrected Modulus (psi)
Uni-L1	19.9	8.30E+05
Uni-L2	59.4	1.90E+06
Uni-L3	45.6	2.30E+06
Uni-W1	35.5	5.04E+06
Uni-W2	19.2	1.72E+06
Uni-W3	10.0	1.04E+06

Table A-2. Tensile Test Results of Face Laminates.

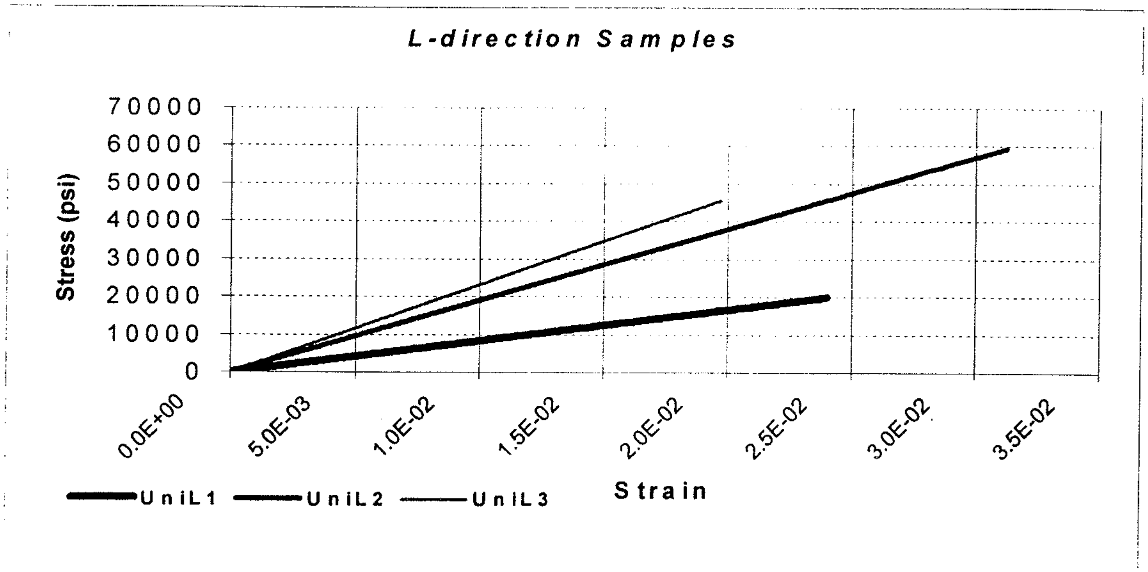

Figure A-1. Stress/Strain Curve for L Specimens.

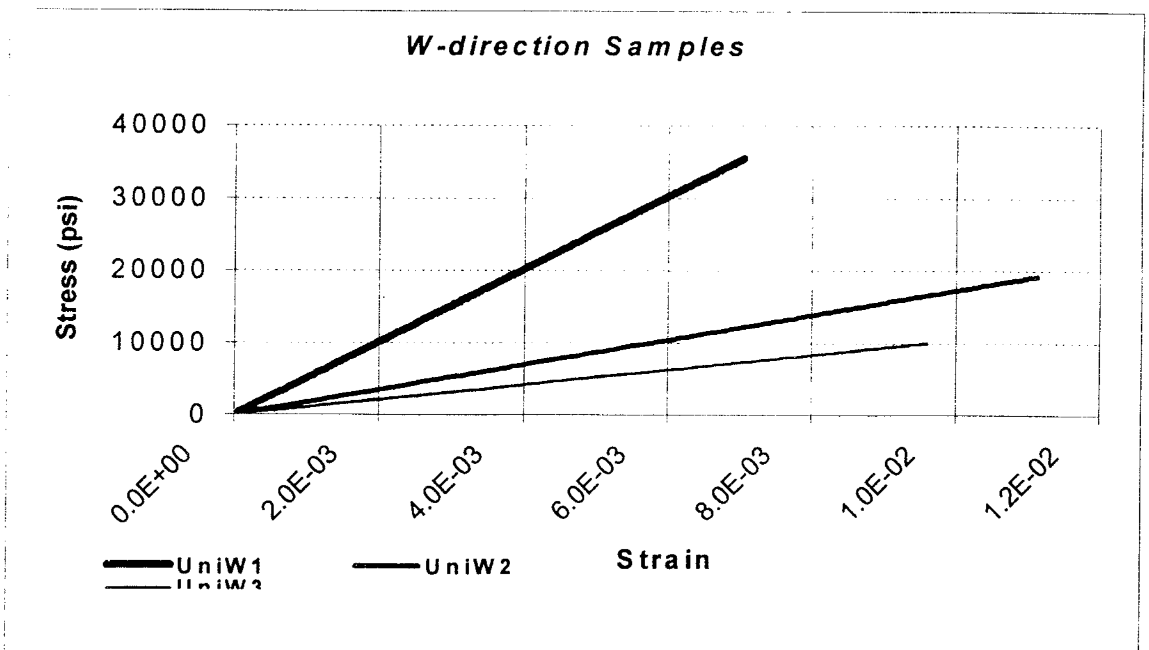

Figure A-2. Stress/Strain Curve for W Specimens.

APPENDIX B – BEAM TESTS; LAMINATE SCHEDULES AND PHYSICAL PARAMETERS

Laminate schedules and physical parameter for the various beam specimens are given in the tables below.

SET-01

	Laminate Schedule No. Layers	Laminate Schedule Description	Weight (lbs) Glass	Weight (lbs) Resin	Weight (lbs) Lamina	Glass Percent	Thickness
Bonding Face (per sq.ft.)	1	3oz csm	0.1875	0.5625	0.75	25%	0.106
Core Laminate	1	3oz csm	0.225	0.4178571	0.6428571	35%	0.071

Beam CL-01	Length (inches)	Width (inches)	Depth (inches)	Weight (pounds)	Density (lbs/ft^3)
Core	95.75	11.63	4.00	20.63	8.00
Beam	97.50	11.75	4.10	33.13	
Face	97.50	11.75		12.50	1.57

Beam CW-01	Length (inches)	Width (inches)	Depth (inches)	Weight (pounds)	Density (lbs/ft^3)
Core	91.50	11.88	4.00	20.13	8.00
Beam	93.38	12.19		31.88	
Face	93.38	12.19		11.74	1.49

Table B-1. Laminate Schedules and Beam Dimensions for Set-01 Specimens.

SET-02

Core Laminate (values per bd.ft.) No. Layers	Laminate Designation	Weight (lbs) Glass	Weight (lbs) Resin	Weight (lbs) Lamina	Glass Percent	Thickness
1	3oz csm	0.23	0.34	0.56	40%	0.060

Face Laminate (values per ft2) No. Layers	Laminate Designation	Weight (lbs) Glass	Weight (lbs) Resin	Weight (lbs) Lamina	Glass Percent	Thickness
1	Bonding	0.19	0.42	0.60	31%	0.082
1	cm3205	0.25	0.24	0.50	51%	0.059
6	um1810	0.96	0.96	1.92	50%	0.229
1	cm3205	0.25	0.24	0.50	51%	0.059
	Totals	1.65	1.86	3.52	47%	0.428

Beam CL-02	
Beam Width	10.00 In
Beam Length	97.25 In
Beam Weight	65.10 Lbs
Areal Density	9.64 Psf

Table B-2. Laminate Schedules and Beam Dimensions for Set-02 Specimen.

SET-03

Face Laminate Schedule		Weight (lbs) (values per ft^2)			Glass	
No. Layers	Description	Glass	Resin	Lamina	Percent	Thickness
1	csm150	0.09	0.14	0.23	40%	0.030
1	cm3205	0.25	0.24	0.50	51%	0.059
6	um1810	0.96	0.96	1.92	50%	0.229
1	cm3205	0.25	0.24	0.50	51%	0.059
	Totals	1.56	1.59	3.15	50%	0.376

Beam CL-03	
Areal density	10.71 lbs/ft^2
width	11.75 Inches
Beam CW-03	
areal density	10.67 lbs/ft^2
width	12.19 Inches

Table B-3. Laminate Schedules and Beam Dimensions for Set-03 Specimens.

SET-04

Wear Surface (values per ft^2)				
Type	Thickness	Resin Weight	Aggregate Weight	Total Weight
Polymer concrete	0.5	0.70	5.17	5.87

Beam Width	10.00 in
Beam Length	97.25 in
Beam Weight	100.20 lbs
Areal Density	14.83681234 psf
Wear Surface	0.562 in

Table B-4. Laminate Schedules and Beam Dimensions for Set-04 Specimen.

CM4545

Laminate Schedule			Weight (lbs/ft^2)			Glass	
	No. Layers	Description	Glass	Resin	Lamina	Percent	Thickness
Core Laminate	1	4.5oz csm	0.28	0.42	0.70	40%	0.090
Face Laminates	1	Bonding	0.19	0.42	0.60	31%	0.082
	1	Cm3205	0.25	0.24	0.50	51%	0.059
	6	Um1810	0.96	0.96	1.92	50%	0.229
	1	Cm3205	0.25	0.24	0.50	51%	0.059
		Face Totals	1.65	1.86	3.52	47%	0.428

Beam CM4545L	
Beam Width	11.13 in
Beam Length	69.88 in
Beam Weight	52.10 lbs
Areal Density	9.65 psf
Beam CM4545W	
Beam Width	10.75 in
Beam Length	74.50 in
Beam Weight	53.80 lbs
Areal Density	9.67 psf

Table B-5. Laminate Schedules and Beam Dimensions for CM4545 Specimens.

APPENDIX C – PUNCH-THROUGH TESTS

Punch Size:	0.75 in			
Punch Area:	0.44 in^2			
Punch Perimeter:	2.36 in			
Wear Surface Overlay Thickness(in)	**0.00**	**0.25**	**0.50**	**0.75**
Total Thickness(in)	0.375	0.625	0.875	1.125
Load at Failure(lbs)	6200	7100	9000	15900
Pressure(psi)	14034	16071	20372	35990
Shear Force(lb/in)	2631	3013	3820	6748
Shear Stress(psi)	7017	4821	4365	5998

Table C-1. Punch-through Test Results from KSU; 3/4-inch punch.

Punch Size:	1.75	in		
Punch Area:	2.41	in^2		
Punch Perimeter:	5.50	in		
Overlay Thickness(in)	**0.00**	**0.25**	**0.50**	**0.75***
Total Thickness(in)	0.375	0.625	0.875	1.125
Load(lbs)	7700	14000	17200	16000
Pressure(psi)	3201	5821	7151	6652
Shear Force(lb/in)	1401	2546	3129	2910
Shear Stress(psi)	3735	4074	3575	2587
*core buckling at outer edge				

Table C-2. Punch-through Test Results from KSU; 1-3/4-inch punch.

APPENDIX D – FATIGUE TESTS

KSU REPORT

August 19, 1998

Five slightly different beams that were previously stiffness tested have been cut into 3-foot lengths and fatigue tested. The original beams, nominally 8ft long, 1ft wide, and 5in thick, were cut into the shorter lengths and fatigue tested with a three point loading on a 2ft span. The repeated loading (over 3600 cycles) was done by hand on a hydraulic testing machine. The honeycomb sandwich beams have an approximate 2in cell honeycomb core made from alternating sinusoidal and flat webs four inches tall. The thickness of the core material varied between .035 to 0.115 inches on the different beams. The faces on this honeycomb core varied from 0.4 to 0.5 inches on the different beams. Three of the beams had ½-inch polymer concrete wear surfaces applied to the top surface.

Nine of these 3-foot beams were tested. One had a premature failure on the first cycle and was ignored. Six others from four of the beams had failure data characteristics that seemed very related. These six data sets were used together to obtain a slope and intercept on a Fn (Frequency vs. cycle Number) type Fatigue curve. The other two 3-foot samples from the fifth beam seemed to be much stronger than the other 6 tests and were therefore analyzed separately.

This reported data is very preliminary, but should be a good approximation of future results. The averaged data from the six samples give a single cycle failure of 27,900 pounds and extrapolated million-cycle fatigue load of 13,400 pounds. The other two samples give a single-cycle load of 33,100 pounds and an extrapolated million-cycle fatigue load of 23,000 pounds.

An early observation that is indicated by the tests is that the fatigue type loading actually increases the single cycle strength of these beams. It is believed that this type loading actually increases the single-cycle strength of these beams. It is believed that this type of repeated loading breaks the "tight" fibers at lower loads thus not causing a catastrophic shock wave that would destroy a beam at a lesser load when that beam would be tested in true single-cycle failure. No explanation for the stronger two samples has been determined. These results are just as expected and this work should and will be continued.

APPENDIX E – DESIGN REPORT

Finite Element Analysis of Proposed KDOT-Crawford County, KS Bridge Deck Panels

Stephen Gill

May 14, 1998

INITIAL ANALYSIS OF VARIOUS PANEL THICKNESSES

Project Design Parameters

The two bridges to be re-decked in Crawford County are 45ft long by 32ft wide. The current construction is an asphalt-on-steel deck supported by 14 W21x68 I-beam stringers on 27in centers. The project proposes to replace the asphalt and steel deck with fiber-reinforced polymer sandwich panels to allow the re-rating of the bridge to HS-20 loading.

The bridges are 45ft long by 32ft wide. The proposed deck panels are to be 32ft by 9ft laid across the longitudinal stringers (perpendicular to traffic).

It is desired to determine the effects of panel thickness on load sharing between the stringers.

FEA Model

The superstructure was modeled by combining composite sandwich deck elements with a beam model of the steel stringers. From symmetry considerations, it was determined that half of one longitudinal lane could be used to determine deflections (i.e., one-quarter of the superstructure).

The sandwich deck was modeled using 660 elements in a 22W x 30L mesh. The elemental area was generally 81in^2 (9in x 9in), though there was some variation in width due to geometry constraints. The elements were ALGOR Type 16 quadrilateral sandwich elements having five degrees of freedom at each node. The deck model was simply supported at the header end of the bridge. The panel boundary conditions along the edges were such as to prevent rotation and translation in the appropriate directions to provide continuity with the phantom areas of the bridge structure.

The supported edge was constrained against translation in the Z-direction (vertical). The contribution to stiffness of the edge closeout beams was neglected. Material properties for the constituent laminae of the deck are given in Table 1.

The steel superstructure was modeled as seven individual half-beams of 30 elements each. The stringer model did not take the additional support provided by beam flange width into account. The two models were combined with the boundary conditions being taken from the deck model.

Loading

The loading on the structure was taken from <u>AASHTO LRFD Bridge Design Specifications</u>. This loading included dead load forces plus a live load combination of the AASHTO design lane plus one HS-20 truck or one tandem axle load. Due to the symmetry of the model, this represents the concurrent loading of both bridge lanes.

The dead loads applied to the bridge include the weight of the structure and that of the wearing surface. The structural load varies with the construction of the deck. The wearing surface was assumed to be a 0.75in layer of polymer concrete.

The AASHTO design lane consists of a 64psf load applied over a 10ft width and, in the case of this single span structure, extended the length of the bridge traffic lane. The HS-20 truck consists of two 32kip axles positioned to provide the maximum stress to the structure. In this case, the axles were spaced at 13.5ft straddling the lateral centerline of the bridge. The alternative tandem axle load consists of two 25kip axles spaced at 4.5ft straddling the lateral centerline of the bridge. The spacings of the axles are not in strict conformance to AASHTO standards due to geometry constraints in the model. Each axle was represented by two wheel loads distributed over 324in^2. Again, due to geometry constraints, the area of the contact patch is not in strict conformance to the standards; however, the total load is as specified. Load distribution is shown on Figure E-2.

Two load cases were analyzed. The Strength I loading conditions are defined as the basic load combination relating to the normal vehicular use of the bridge. The loads and multipliers for this case are given in Table E-2.

The Service I loading conditions are defined as the load combination relating to normal operational use of the bridge. The loads and multipliers for this case are given in Table E-3.

The load factors γ are taken from Tables 3.4.1-1 and 3.4.1-2 of the LRFD Specifications. The factor η is the geometric sum of factors considering the importance of the structure, redundancy of construction, and the ductility of the materials used.

Analysis Method

The depth of core was varied from 2in to 10in to determine the effect on beam load sharing and panel stress levels. The minimum allowable face laminate thickness is deemed to be 0.375in and was held constant for all analyses.

It was determined from early analysis that the maximum deflections - and maximum stresses - would be found under the single truck loading as opposed to the tandem axle loading. The results given are for the truck loading with the design lane load.

Results

The tables give the stresses and stringer deflections for all core depths under both load cases. The stresses given are the maximum stresses in the structure for the given loading. The deflections are taken at the lateral centerline of the bridge. The load sharing is based on the assumed linearity of deflections under loading for elastic deformations. The load sharing percentage is the deflection of each stringer versus the sum of all deflections.

It can be seen from the tables of deflections that load sharing between stringers increases with depth of core. This behavior is to be expected as the stiffness of the panel increases. Overall deflections do not vary appreciably with panel stiffness. The difference between the maximum deflections for the 2-inch and 10-inch cores is 8.2%.

While the maximum single beam deflection decreases with increased core depth, the sum of all beam deflections increases. This may suggest that the total strain energy in the bridge is increased for thicker panels. Some of this may be due to added panel weight. Proof of this would be the comparison of deflections away from the span centerline for the various thicknesses. While the major thrust of the analysis is to determine load sharing in the lateral direction, there is, of course, load spreading in the longitudinal (traffic) direction. An increase of two inches in core depth increases the areal weight of the deck panels by 1.7psf or 2400lbs for the bridge.

There is little difference in load sharing percentages between the two load cases. It is obvious that the Strength I condition is the worst case.

The stresses shown are as follows:

- σ_{11} = maximum absolute bending stress in the lateral direction
- σ_{22} = maximum absolute bending stress in the longitudinal direction
- τ_{13} = maximum absolute transverse shear stress in the lateral plane
- τ_{23} = maximum absolute transverse shear stress in the longitudinal plane
- σ_{vm} = maximum von Mises stress

Stress levels show an anomaly with increased core depth. As can be seen from Tables 6 and 7, stresses generally decrease with increased depth. However, at a core depth somewhere between six and ten inches, the σ_{22} stress begins to increase. This begins to raise the overall stress level in the panels. This high stress area is located near the center of the bridge and extends outward to the wheel loads. It is unknown whether this is due to the ratio of panel thickness to beam spacing or the ratio of beam stiffness to panel stiffness. It would seem from these results that there is an optimal panel thickness where the stress level reaches a minimum with an acceptable load sharing capability. This phenomenon may also be related to the difference in face laminate stiffness in the 1 and 2 directions and the effect this may have on stress transfer between the two directions. The longitudinal stiffness of these panels is lower than the lateral stiffness due to the structure of the laminate.

ANALYSIS OF THE FINAL DESIGN

After analysis of the data taken from test beams with various core configurations, it was determined that a four-inch core with .375in faces and .50in wear surface would be sufficient to support the specified loading. The core will have .090in webs and be of triangular, crown-to-crown construction.

The loading is to be AASHTO HS-25. The panels are supported on 14 stringers. The bridge stringers are connected with diaphrams along the lateral centerline of the bridge, with additional diaphrams between the outer three beams located at the guardrail posts (8 locations spaced at 75in beginning either side of the bridge lateral centerline). The guardrail post diaphrams are C15x33.9 steel channels. The centerline diaphrams are W10x19 beams.

The expected properties of the sandwich construction are the same as those given previously in Table E-1. The model is extended to full length to accommodate the inclusion of the diaphrams.

Results

The deflections in Tables E-8 and E-9 are taken at each stringer along the lateral centerline of the bridge.

For the 4in panels, the values in Table E-10 were calculated. The inter-beam span to deflection ratios for Table 10 are calculated by subtracting the deflection of the appropriate beam from the average deflection of the two adjacent beams. The ratios given are well above the usual failure L/d of 100. The minimum safety factor would be 12.5 under Strength I loading and 22 under Service I.

The total panel differential deflection is 1.40in for Strength I loading. The allowable deflection for a L/d of 100 is 3.48in (based on the 29ft distance between outer stringers), giving a Strength I factor of safety of 2.48 against total panel failure.

Comparison with Classic Beam Theory

As a check on the validity of the model, a comparison was made between the results of the FE analysis and the expected results using classic beam deflection equations. The classic equation assumed one beam to support the total load. The deflection resulting from this calculation was compared with the sum of the center point beam deflections from the FE analysis. The contribution of the deck to longitudinal bridge stiffness was ignored as it is only 0.2% that of the stringer beams. The load included only the design lane load and truck axle loads, ignoring the dead load of the structure. The results for the classic calculation gave a center point deflection of 7.22in. The sum of beam deflections from FE was 7.07in. The difference is 2.1%.

As an additional check of the results, consultation was conducted with the KDOT Design engineer in order to compare figures. The total dead load deflection of the structure is 0.217in based on a deck weight of 16psf. FE deflection was 0.188in, a 13% difference.

TESTING AND ANALYSIS OF THE CM4545 BEAMS

Previous testing of various core web thicknesses pointed to the use of 4.5oz material in the construction of the core. There was a desire to strengthen the core due to the high shear loads expected over the short spans between the existing beams of the bridges.

Two beams were constructed; one each in the longitudinal and lateral directions. The test results from these samples were used to determine safety factors based on the FE analysis.

Beam Construction

The beams were constructed with crown-to-crown core configuration. The web thickness was approximately 0.090in. Faces consisted of the current .375in laminate schedule. Overall beam dimensions are found in the test data spreadsheet. No wear material was applied to the surface.

Test Method

The beams were tested twice for each span of 60in and 42in to provide comparison with previous data. The tests over these spans were continued until noticeable acoustic emissions were recorded. These emissions were assumed to be the initial failure of the glue bonds within the cored panel. The beams were tested to failure over a span of 24in to approximate conditions of the installed panels and to generate data for determining ultimate failure parameters.

All tests were conducted under three-point loading. Deflections relative to the supports were measured with a dial indicator mounted between the loading head of the machine and a bar between the supports.

Analysis

Deflection

Corrected deflection values were within 1% for both tests over the 60in and 42in spans. The data was extremely linear in all cases. Effective moduli decreased with span length, due to the increasing contribution of shear deformation to overall deflection.

Stiffness

Longitudinal Beam (L)

The effective moduli for the various spans were:

Span (in)	60	42	24
Effective Modulus (psi)	2.46e6	1.96e6	1.28e6

Lateral Beam (W)

The effective moduli for the various spans were:

Span (in)	60	42	24
Effective Modulus (psi)	1.32e6	1.08e6	0.54e6

The ratio of L/W moduli for the three spans averages to 2:1. The ratios for the 60in and 42in spans was 1.8, and that of the 24in span was 2.37.

Failure occurred for both specimens over the 24in span as the L/d approached 100. This would suggest that the shear stress in the bonding layer is the critical factor in design. Previous experience has shown that this ratio of 100 is relatively consistent. However; all the previous experience has also been with the standard core geometry. The only consistent factor has been the ratio of bond length to cell area, suggesting that other geometries with greater bond density (i. e., greater perimeter/cell area ratios) might prove to have higher face bond strength.

Stress

Longitudinal Beam

Extreme fiber bending stress at failure is calculated to be 14.2ksi. Maximum plate shear stress in the core is 410psi.

Lateral Beam (W)

Extreme fiber bending stress at failure is calculated to be 5.4ksi. Maximum plate shear stress in the core is 156psi.

Failure for both beams initiated in the glue joints within the core panel. The final catastrophic failure of the panel occurs when the faces de-bond from the core due to shearing and upper face buckling forces.

COMPARISON OF TEST RESULTS WITH FE ANALYSIS

Stress factors of safety can be determined from the test results in comparison with the FE analysis.

For Strength I loading, the safety factor for lateral bridge direction bending stress is 7.9. The safety factor for longitudinal bride direction is 3.0. The lateral vertical shear stress factor is 3.7. The longitudinal shear stress factor is 4.3.

For Service I loading, the safety factor for lateral bridge direction bending stress is 13.8. The safety factor for longitudinal bride direction is 5.2. The lateral vertical shear stress factor is 6.4. The longitudinal shear stress factor is 7.5.

	Moduli (psi)				Strengths (psi)			
Layer	**E11**	**E22**	**G13**	**G23**	**S11**	**S22**	**S13**	**S23**
Wear Surface	5000000	5000000	500000	500000				
Top Face	2200000	490000	500000	500000	17800	6675		
Core (.090 web)	27000		75000	25000			1050	615
Bottom Face	2200000	1000000	500000	500000	17800	6675		

Table E-1. Deck Material Properties.

Description	**Color**	**Nominal Force (psi)**	**Load Factor(γ)**	η	$\eta\gamma$	**Total Load (psi)**
Structural Load (DC)	All	Varies	1.25	1.047	1.309	Varies
Wear Surface (DW)	All	0.065	1.50	1.047	1.570	0.102
Truck Wheel	1	65.7	1.75	1.047	1.832	120.4
Tandem Wheel	2	51.3	1.75	1.047	1.832	94.0
Design Lane	3	0.44	1.75	1.047	1.832	0.806

Table E-2. Strength I Loading

Description	**Color**	**Nominal Force (psi)**	**Load Factor(γ)**	η	$\eta\gamma$	**Total Load (psi)**
Structural Load (DC)	All	Varies	1.00	1.047	1.047	Varies
Wear Surface (DW)	All	0.065	1.00	1.047	1.047	0.102
Truck Wheel	1	65.7	1.00	1.047	1.047	68.9
Tandem Wheel	2	51.3	1.00	1.047	1.047	53.7
Design Lane	3	0.44	1.00	1.047	1.047	0.461

Table E-3. Service I Loading.

	Stringer Position						
2-inch Core	**Inner**	**2**	**3**	**4**	**5**	**6**	**Outer**
Deflection (in)	2.89	2.94	2.83	2.62	2.22	1.56	0.83
% of Total	18.2	18.5	17.8	16.5	14.0	9.8	5.2
4-inch Core							
Deflection (in)	2.92	2.88	2.75	2.53	2.20	1.76	1.29
% of Total	17.9	17.6	16.8	15.5	13.5	10.8	7.9
6-inch Core							
Deflection (in)	2.84	2.80	2.68	2.51	2.27	1.97	1.66
% of Total	17.0	16.7	16.0	15.0	13.6	11.8	9.9
10-inch Core							
Deflection (in)	2.68	2.66	2.59	2.50	2.37	2.22	2.08
% of Total	15.7	15.5	15.1	14.6	13.8	13.0	12.2

Table E-4. Deflections and Load Sharing Under Strength I Loading.

	Stringer Position						
2-inch Core	**Inner**	**2**	**3**	**4**	**5**	**6**	**Outer**
Deflection (in)	1.683	1.714	1.657	1.543	1.311	.922	.497
% of Total	18.0	18.4	17.8	16.5	14.1	9.9	5.3
4-inch Core							
Deflection (in)	1.731	1.712	1.634	1.509	1.319	1.060	.784
% of Total	17.8	17.6	16.8	15.5	13.5	10.9	8.0
6-inch Core							
Deflection (in)	1.715	1.690	1.622	1.519	1.375	1.195	1.006
% of Total	16.9	16.7	16.0	15.0	13.6	11.8	9.9
10-inch Core							
Deflection (in)	1.677	1.660	1.621	1.563	1.487	1.398	1.310
% of Total	15.6	15.5	15.1	14.6	13.9	13.0	12.2

Table E-5. Deflections and Load Sharing Under Service I Loading.

	Stress (psi)				
	σ_{11}	σ_{22}	τ_{13}	τ_{23}	σ_{vm}
2-inch Core	2852	2442	248	78	3065
4-inch Core	2150	1789	118	39	2153
6-inch Core	1870	1720	95	28	1850
10-inch Core	1721	2390	56	17	2110

Table E-6. Maximum Stresses Under Strength I Loading.

	Stress (psi)				
	σ_{11}	σ_{22}	τ_{13}	τ_{23}	σ_{vm}
2-inch Core	1631	1402	142	45	1751
4-inch Core	1227	1039	67	22	1235
6-inch Core	1076	1025	54	15	1074
10-inch Core	1007	1466	32	9.6	1300

Table E-7. Maximum Stresses Under Service I Loading.

	Stringer Position						
2-inch Core	**Inner**	**2**	**3**	**4**	**5**	**6**	**Outer**
Deflection (in)	2.640	2.606	2.474	2.247	1.895	1.424	.933
% of Total	18.6	18.3	17.4	15.8	13.3	10.0	6.6
4-inch Core							
Deflection (in)	2.622	2.575	2.448	2.246	1.958	1.589	1.224
% of Total	17.9	17.6	16.7	15.3	13.4	10.8	8.3
6-inch Core							
Deflection (in)	2.569	2.525	2.419	2.255	2.033	1.765	1.489
% of Total	17.1	16.8	16.1	15.0	13.6	11.7	9.9
10-inch Core							
Deflection (in)	2.475	2.443	2.379	2.284	2.161	2.019	1.875
% of Total	15.8	15.6	15.2	14.6	13.8	12.9	12.0

Table E-8. Deflections and Load Sharing Under Strength I Loading (revised model).

	Stringer Position						
2-inch Core	**Inner**	**2**	**3**	**4**	**5**	**6**	**Outer**
Deflection (in)	1.545	1.526	1.451	1.321	1.121	.853	.573
% of Total	18.4	18.2	17.3	15.7	13.4	10.2	6.8
4-inch Core							
Deflection (in)	1.553	1.526	1.454	1.339	1.175	.971	.758
% of Total	17.7	17.4	16.6	15.3	13.4	11.1	8.6
6-inch Core							
Deflection (in)	1.540	1.515	1.455	1.362	1.236	1.085	.929
% of Total	16.9	16.6	16.0	14.9	13.5	11.8	10.2
10-inch Core							
Deflection (in)	1.517	1.501	1.465	1.412	1.343	1.264	1.184
% of Total	15.7	15.5	15.1	14.6	13.9	13.0	12.2

Table E-9. Deflections and Load Sharing Under Service I Loading (revised model).

Strength I	**Inner**	**2**	**3**	**4**	**5**	**6**	**Outer**
Deflection	-2.622	-2.575	-2.448	-2.246	-1.958	-1.598	-1.224
% of Total	17.9%	17.6%	16.7%	15.3%	13.4%	10.8%	8.3%
Diff. Defl.		-0.050	-0.127	-0.202	-0.288	-0.369	-0.365
L/d (54in)		1350	1440	1256	1333	7714	
Service I	**Inner**	**2**	**3**	**4**	**5**	**6**	**Outer**
Deflection	-1.553	-1.526	-1.454	-1.339	-1.175	-0.971	-0.758
% of Total	17.7%	17.4%	16.6%	15.3%	13.4%	11.1%	8.6%
Diff. Defl.		-0.027	-0.072	-0.115	-0.164	-0.204	-0.213
L/d (54in)		2400	2512	2204	2700	12000	

Table E-10. Deflections and Load Sharing Under Service I Loading (revised model).

Stress	**Lateral Bending**	**Longitudinal Bending**	**Lateral Vertical Shear**	**Longitudinal Vertical Shear**	**von Mises**
Strength I	1800	-1807	111	36.5	2102
Service I	1027	-1048	63.7	20.9	1205

Table E-11. Maximum Stresses for 4-inch Panels (revised model).

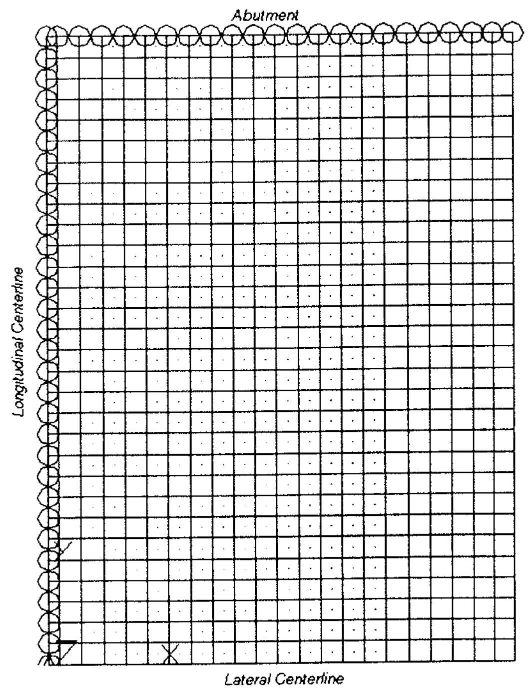

Figure E-1. Element Map of 1/4-Plate Deck Model.

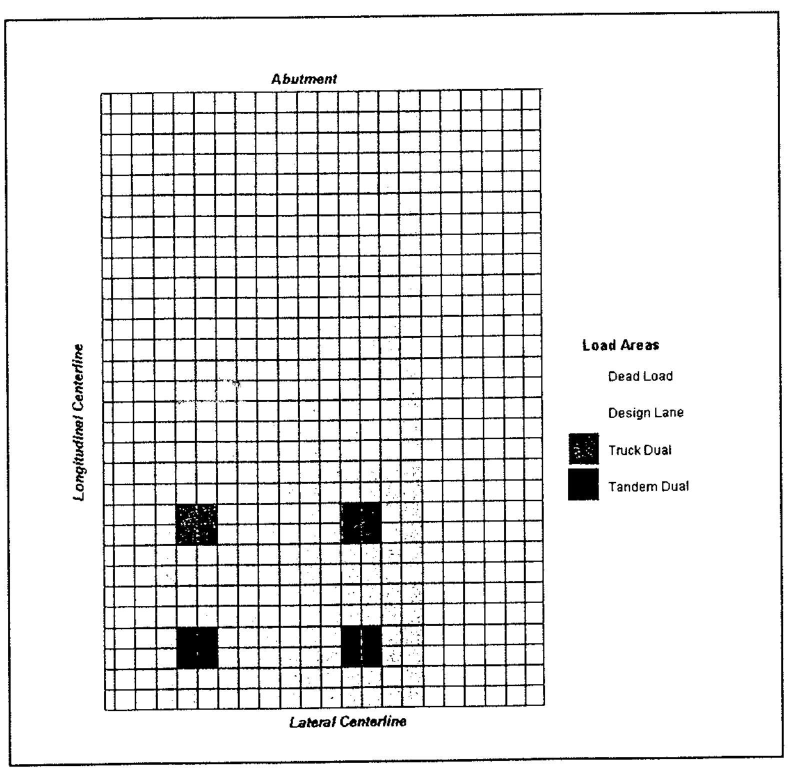

Figure E-2. Load Distribution.

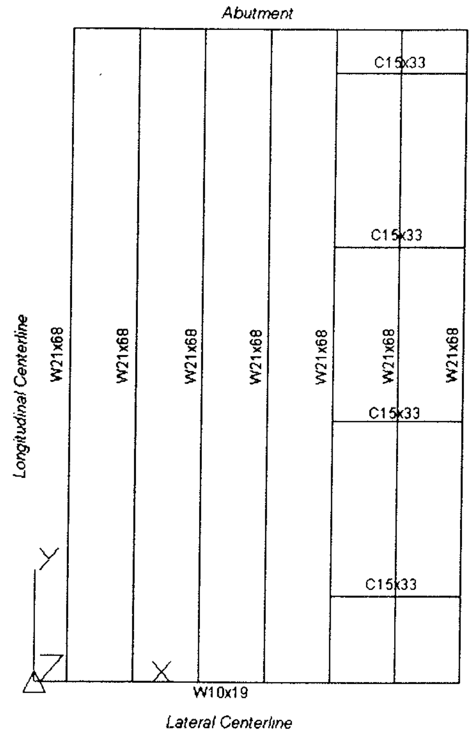

Figure E-3. Steel Frame Model of Deck Support Structure

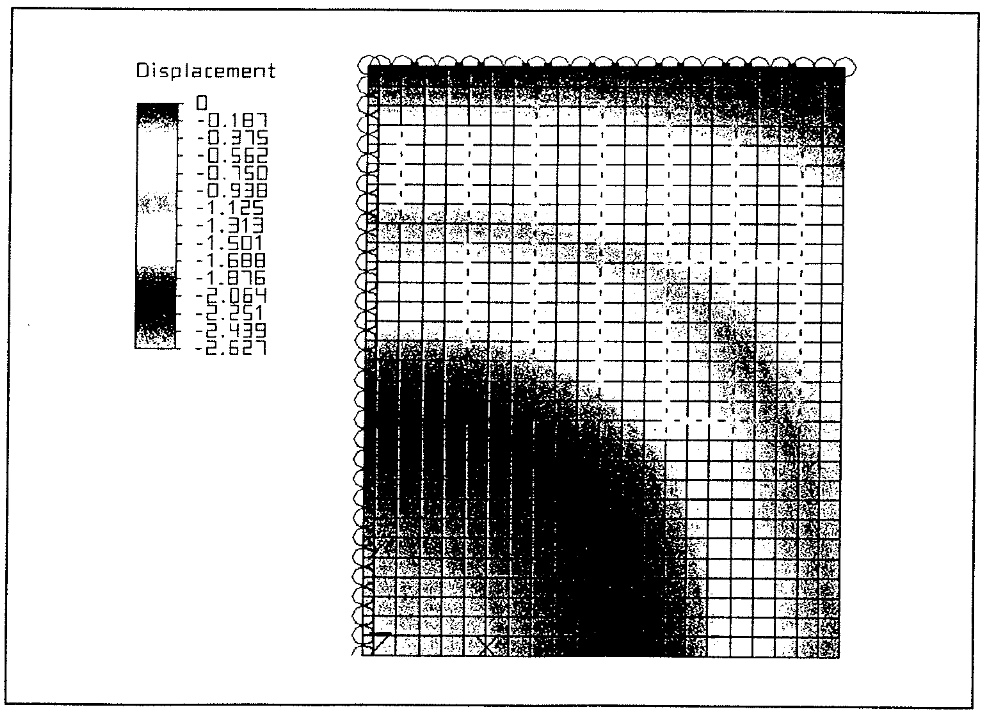

Figure E-4. Strength I Deflection Map.

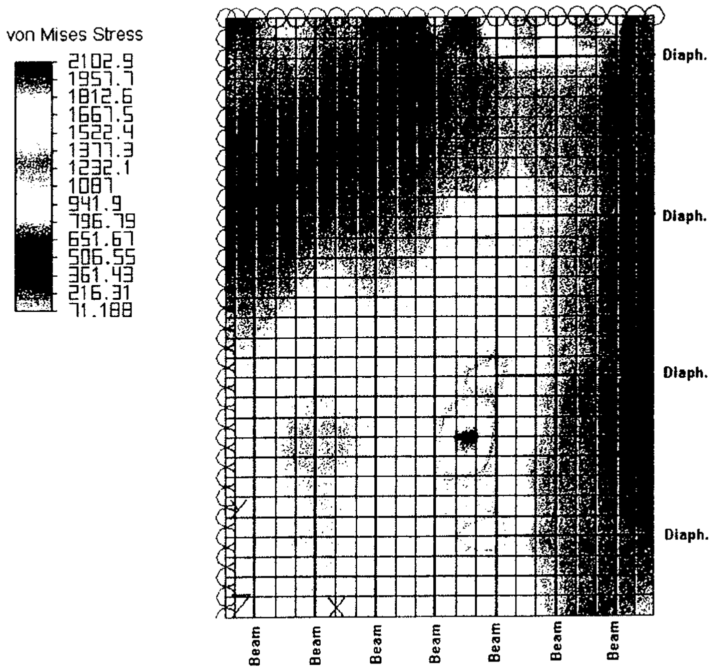

Figure E-5. Strength I von Mises Stress.

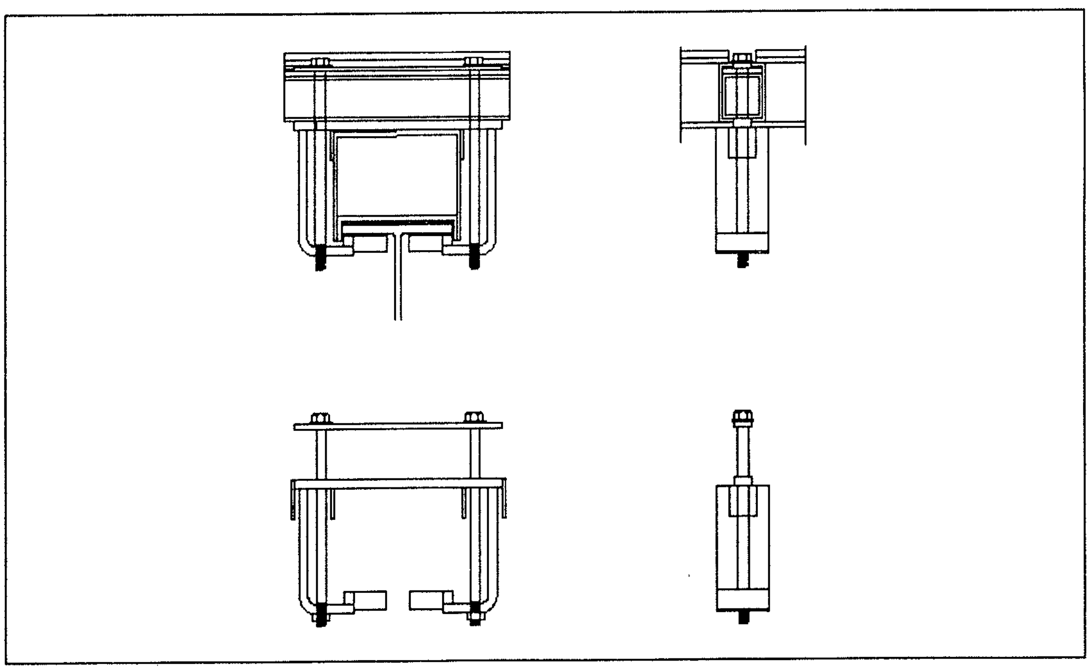

Figure E-6. Anchor Clamp and Saddle Installation.

APPENDIX F – FINAL PANEL PARAMETERS

KDOT/Crawford County Deck Panel Materials; Materials for (1) 8ft x 16ft Panel

Panel Dimensions

Length:	192 in
Width:	96 in
Area:	128 ft^2
Total Thickness:	5.48 in
Thickness less wear:	4.98 in

Core

Core Type:	Crown
Depth:	4.125 in
Resin:	Polyester
Board Feet:	528 bd. ft.

Laminate Description	Weight (lbs.) Glass	Resin	Lamina	Glass Percent	Web Thickness
Mat450	178.0	267.0	445.0	40%	0.090
Per bd. ft.	0.338	0.506	0.844		
Per ft^2	1.392	2.088	3.480		

Wear Surface

Type	Thickness	Resin Weight	Aggregate Weight	Total Weight
DCPD polymer concrete	0.500	110.9	628.3	739.2

Upper Face

Resin: Polyester

Laminate Schedule No. Layers	Description	Weight (lbs.) Glass	Resin	Lamina	Glass Percent	Thickness
1	bonding	24.00	53.42	77.42	31%	0.082
1	cm3205	32.40	31.13	63.53	51%	0.059
6	um1810	122.88	122.88	245.76	50%	0.229
1	cm3205	32.40	31.13	63.53	51%	0.059
	Totals	**211.7**	**238.6**	**450.2**	**47%**	**0.428**

Lower Face

Resin: Polyester

Laminate Schedule No. Layers	Description	Weight (lbs.) Glass	Resin	Lamina	Glass Percent	Thickness
1	bonding	24.00	53.42	77.42	31%	0.082
1	cm3205	32.40	31.13	63.53	51%	0.059
6	um1810	122.88	122.88	245.76	50%	0.229
1	cm3205	32.40	31.13	63.53	51%	0.059
	Totals	**211.7**	**238.6**	**450.2**	**47%**	**0.428**

Total Panel Weight	**2085 lbs.**
Weight/Area	**16.29 lb/ft^2**
(less wear surface)	**10.52 lb/ft^2**

Constituent Laminates

Core Web Laminate:

Weight	4.5 oz/ft^2
Reinforcement Percent	40 %
Resin	polyester
Laminate Thickness	0.0898103 in
Laminate Weight	0.0048828 Lb/in^2

Bonding Layer	3.0 oz. Mat	
Weight	3.0 oz/ft^2	Chopped strand mat
Reinforcement Percent	31 %	
Resin	Polyester	
Laminate Thickness	0.0819 in	
Laminate Weight	0.0042 Lb/in^2	

Brunswick Technologies; um1810 18oz unidirectional/1oz csm

Weight	2.56 Oz/ft^2
Reinforcement Percent	50 %
Resin	Polyester
Laminate Thickness	0.0381 in
Laminate Weight	0.0022 lb/in^2

Brunswick Technologies; cm3205 16oz@0°/16oz@90° bidirectional/.5oz csm

Reinforcement Weight	4.05 Oz/ft^2
Reinforcement Percent	51 %
Resin	Polyester
Laminate Thickness	0.0586 in
Laminate Weight	0.0034 lb/in^2

Polymer Concrete

Aggregate Density:	150 lb/ft^3
Aggregate Percent:	85 %
Resin	DCPD
Mixture Density	0.0802 lb/in^3

Polymer Concrete Wear Surface Aggregate Mix

Lincoln County, KS Quartzite

1/4 minus Aggregate

Screen Number	Hole Size	Aggregate Weight (lbs)	Percent Retained
4	0.1875	0.2	1.8%
16	0.0469	4.0	38.4%
30	0.0232	3.0	29.1%
50	0.0117	2.6	25.2%
100	0.0059	0.5	5.0%
<100		0.0	0.4%
	Total	10.4	100.0%