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Study of Cracking of Composite Deck Bridge on I-75 over Peace River

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Observed cracking on the Peace River Bridge on Interstate 75 near Punta Gorda, Florida, caused concern about the possibility of high maintenance cost and the structural adequacy of the bridge system. The deck system consists of precast panels resting on soft fiberboard, which serve as formwork for the road surface and later aid in carrying the traffic loads. An investigation has been completed that involved testing of the Peace River Bridge, testing of the FL-776 Bridge (a nearby structure of similar construction), analytic modeling using the finite element method, and limited laboratory testing of beam specimens. The investigation indicates that although the Peace River Bridge is adequate to carry normal traffic, the shear stresses in the bridge deck are substantially higher than those of deck systems that have positive bearing at the ends of the panels. Further experimental studies are under way to determine the shear fatigue life of the bridge. The causes of cracking and separation at the ends of the panels are identified as differential shrinkage and creep due to prestress forces. Recommendations for future construction projects are made.

Rising costs of formwork, materials, and labor have greatly increased the cost of reinforced concrete bridges constructed with conventional field forming techniques. Construction techniques that reduce the amount of forming done under field conditions increase the economy of the bridge. Prefabricated prestressed girders have been in common use in bridges for approximately 30 years. Precast stay-in-place forms of concrete and steel replaced wooden forms in recent years and eventually led to the development of precast composite deck panels. Composite deck panel bridges contain precast prestressed panels that span between bridge girders and support the cast-in-place topping, eliminating most of the field formwork. Research in Florida, Pennsylvania, and Texas led to their widespread acceptance and incorporation into the American Association of State Highway and Transportation Officials (AASHTO) specifications (1). Figure 1 shows typical composite bridge panel construction, as built in Florida, prior to this research.

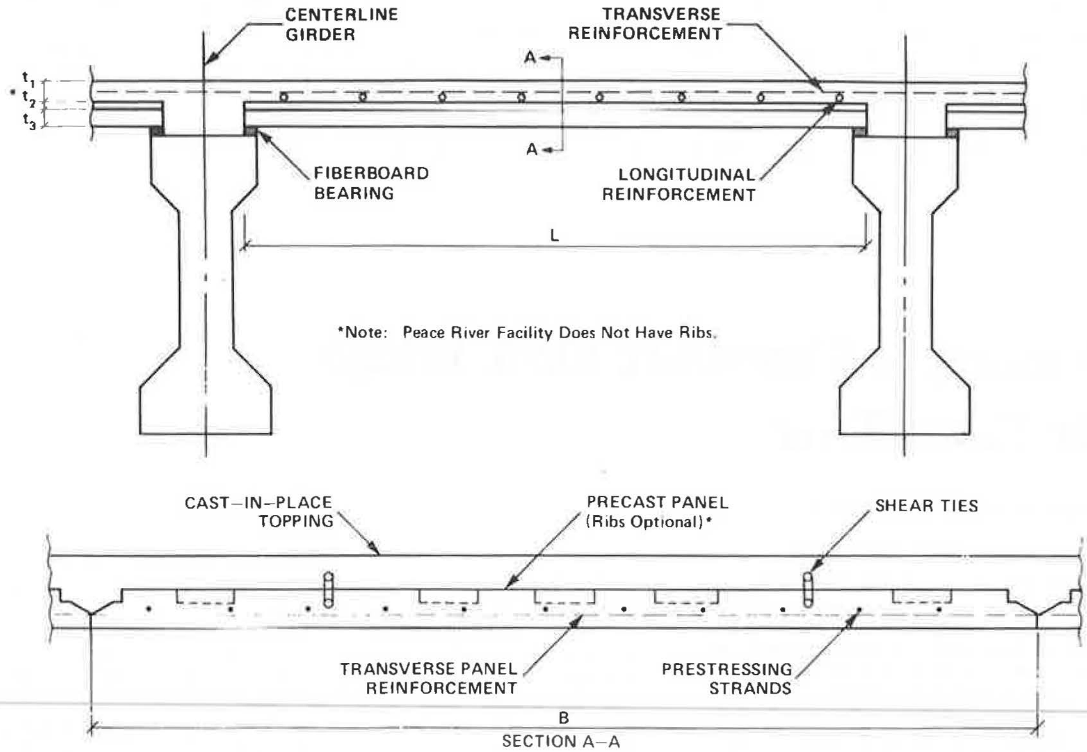
Recent research in Florida (2) and Louisiana (3) dealt with full-span form panels that span directly between piers without using prestressed girders. Additional research on deck panels was recently completed in Texas (4). Although there are significant differences in these two types of construction, they both exhibit more regular cracking patterns than bridges with reinforced concrete decks constructed by using conventional forms. The combination of (a) shrinkage due to placing a thin layer of fresh concrete on top of a deck panel that has already undergone a major portion of its shrinkage and (b) vertical joints between panels and cast-in-place concrete in regions of high stress (due to traffic) will cause cracking and the cracking will follow a regular pattern. However, extensive research and experience have shown that these systems can be safely used in bridge construction.

The Peace River Bridge on I-75 near Punta Gorda, Florida, was constructed with prestressed girders and composite deck panels. During the construction of the bridge, an unusually large number of cracks were observed in the deck. As pointed out earlier, some cracking is inherent in this type of construction, but the extensive early cracking that was observed caused concern about the possibility of excessively high maintenance costs due to deterioration of the deck with time.

Preliminary studies of the plans for the Peace River Bridge indicated one major difference from details used in other states. On the Peace River Bridge, and other bridge work in Florida, the precast panels are supported, as shown in Figure 1, by fiberboard so that the panels do not have positive bearing on the girders. One series of the Florida panel tests (5) was made without positive bearing for the panels, and satisfactory performance of the panels was observed. However, these test panels had prestressed strands that extended a short distance into the cast-in-place concrete. In addition, these laboratory test specimens were not exposed to temperature, creep, and shrinkage stresses, which aggravate the cracking near the end of the panels under field conditions.

The panels are designed to act compositely with the cast-in-place concrete in resisting live loads and are assumed to act as a continuous slab with negative moment developed in the slab over the girders. The ability of the panels to transfer shear across their ends and provide continuity was questioned due to the observed cracking. Prior research concentrated heavily on demonstrating that adequate bond could be developed between the top of the panels and the cast-in-place topping. Only minimal attention was given to the bond between the end of the panels and the cast-in-place concrete over the girders. The exact mechanism of the shear transfer and the degree of continuity in this region of interfaces between various concretes with creep, shrinkage, and temperature cracks is difficult to predict with any degree of certainty. Thus, a thorough investigation of the Peace River Bridge was warranted.

Figure 1. Typical composite bridge panel construction.



FIELD TESTS

Static and dynamic tests of the Peace River Bridge were made to determine the degree of composite action and structural adequacy of the decks. Tests were performed under three conditions:

1. Present condition--Testing was performed on (a) sections with typical reinforcement used in most areas of the bridge and (b) sections with increased reinforcement.
2. Remedial improvements--The fiberboard under several panels was removed and the void so created was grouted. These panels were later retested.
3. Comparison tests--For comparative purposes, a nearby bridge on I-75 crossing FL-776 was tested. The FL-776 bridge had similar details but shorter panel spans and less extensive cracking.

LABORATORY TESTS

After a preliminary study of the field data, the major area of concern became the shear behavior of the joints at the ends of the precast panels. Thus, a series of laboratory tests was made on slab specimens constructed by using strips sawn out of panels left over from the construction of the Peace River facility. These strips or "beam" specimens were loaded cyclically to study their shear fatigue strength. The results of these beam tests (6), although only applicable in a qualitative way to actual bridge decks, indicated that testing of wider specimens was necessary to determine the shear fatigue life of the Peace River Bridge. Tests are already in progress at the University of Florida to evaluate the shear strength and behavior of wide-slab specimens with joint details similar to those on deck systems of the Peace River type.

FIELD TESTING PROCEDURES

Peace River Bridge

The Peace River Bridge consists of two parallel structures that have a number of simple girder spans between 65 and 105 ft. A Florida Department of Transportation (DOT) water tanker was used for loading the bridges, either hydraulically or as a vehicle load. A data acquisition system was stored in an instrument trailer and used to record deflection and strain measurements. Figure 2 shows the configuration used for most of the tests in which deflection measurements were made by using two wooden gage support beams attached to the center three girders. Two other displacement gages were supported on top of the deck to measure the relative slip across longitudinal cracks.

FL-776 Bridge

The FL-776 Bridge also had two parallel structures. Each FL-776 structure consisted of two simple skewed spans with seven Type IV girders. Girder spans were 108.5 ft, and slabs spanned transversely approximately 4.5 ft between girders. The test layout for the FL-776 Bridge was similar to that described above for the Peace River Bridge.

Test Loadings

Figure 2 shows the truck in position for one of the hydraulic tests with the load in the center of a panel. All figures that show transverse sections of the bridge were drawn facing north. Figure 3 shows the detailed plan location of the loadings. Figure 3a shows the positions of the hydraulic loadings. Loads A, B, C, and D refer to different positions of the wheel plate (2), whose dimensions are shown in Figure 3b. Two positions are shown for the gage line. The primary gage line was in the longitudinal center of the panel. In most cases the testing pattern was load positions A, C, B, and D with the gages located on the primary gage line. However, for a few of the tests, the test gages were moved to a secondary position 6 in north of the end of the panel and tests C and D were performed for the gages on the adjacent panel.

The hydraulic loads were applied in 8-kip increments up to a maximum load of 32 kips, which is approximately 1.5 times the AASHTO HS-20 design wheel load of 16 kips with an impact factor of 0.3. The hydraulic load was applied by jacking against the water tanker. The jack was centered over the wheel plate and located with respect to the truck axles as shown in Figure 3d.

Figure 3c shows the location of the axle loads for the static and dynamic truck tests. For each static truck test, the three axles were each separately placed over the gage lines. The symbols TA,

Figure 2. Schematic of field test arrangement for Peace River Bridge.

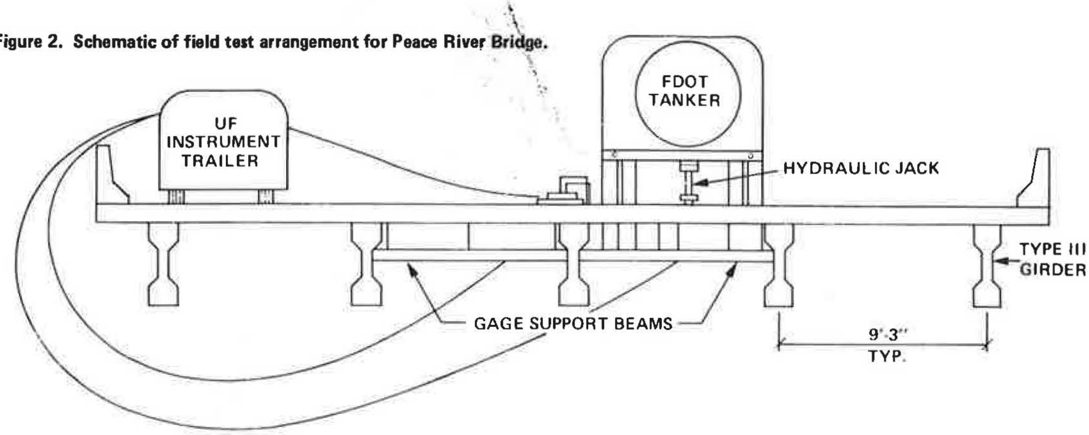


Figure 3. Plan location of loads.

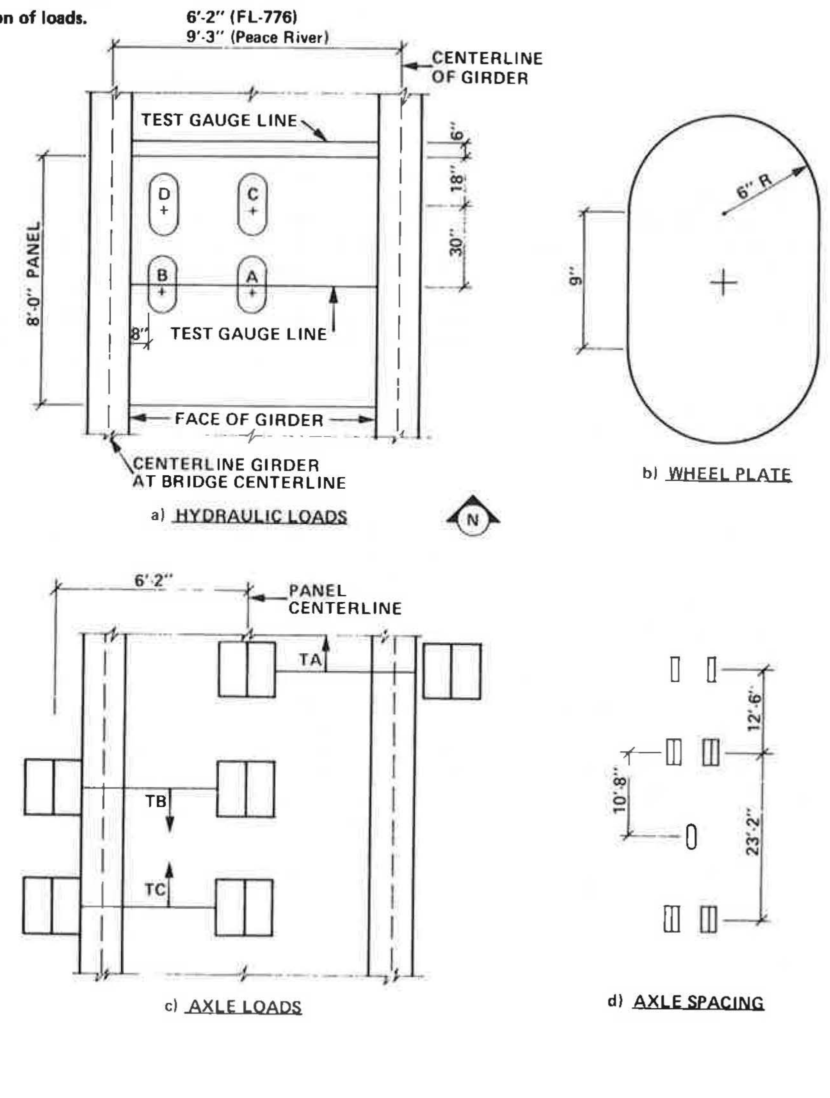
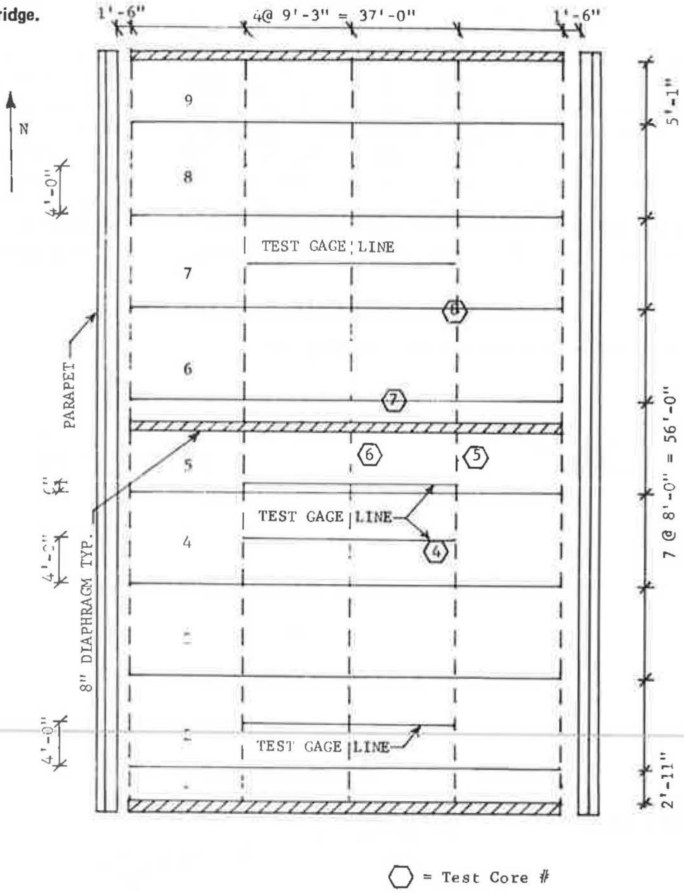


Figure 4. Location plan of gages for span 11R of Peace River Bridge.



TB, and TC refer to three different orientations of the truck wheels. The arrow shows the direction in which the truck was facing. In the dynamic tests, the transverse position of the wheels was the same as in the corresponding static tests (as closely as could be maintained) and the truck was driven over the bridge at speeds ranging from 5 to 40 mph.

Deflection Gage Locations

Figure 4 shows the plan location of the gage lines for one span of the Peace River Bridge. Deflections were measured by a series of linear variable differential transformers (LVDTs) located on the gage lines previously discussed. An LVDT is an electrical-mechanical transducer that produces an electrical output proportional to the displacement of a separate inner core. One end of a threaded rod attached to the core was connected to a threaded tube that was epoxied to a magnetic block. This block was then attached magnetically to a steel plate that had been epoxied to the bottom of the bridge slab. The plastic LVDT holder was attached to an aluminum bar that was held onto the gage support beam by a C-clamp. Figure 5 shows a section of the gages for the FL-776 Bridge as well as the Peace River Bridge. Wooden end blocks were epoxied to the sides of the prestressed girders. The wooden gage support beams were then connected to the end blocks by dowels. One end of each gage support beam had a circular hole and the other end was slotted. With this configuration, the gage support beams would not restrain the movement of the girders and should have moved as a rigid body during the testing. The gage support beams were made out of plywood and could be adjusted in length to accommodate minor variations in girder spacing.

Gages 1-10 were supported on the gage support beams under the bridge deck. Gages 11 and 12 were supported by a wooden fixture over the girder on top of the deck. The two top gages were located approximately 2 in on either side of the longitudinal crack that ran close to the edge of the girder. The difference in deflection between gages 11 and 12 was therefore a measure of the deformation at the end of the panel.

Strain Gage Locations

Strain gages were also used in some tests and were located along the same gage line as the LVDTs. Figure 6 shows the locations of the strain gages in relation to the girders and the LVDT gages. No strain readings were made on the FL-776 Bridge.

Ground Reference Tests

Due to the large number of spans tested and their precarious locations (generally over water or high above the ground), it was felt that the deflections of the slab relative to the girder would be easier to obtain than absolute deflections measured from the ground. However, some measurements were obtained with LVDT gages attached to scaffolding supported on the ground as means of referring relative measurements to absolute deformations.

Data Acquisition System

The 3052A data acquisition system manufactured by Hewlett Packard consists of a 9825A system controller, a 3495A multiplexer, a 3497A system volt-

Figure 5. Location of LVDT gages.

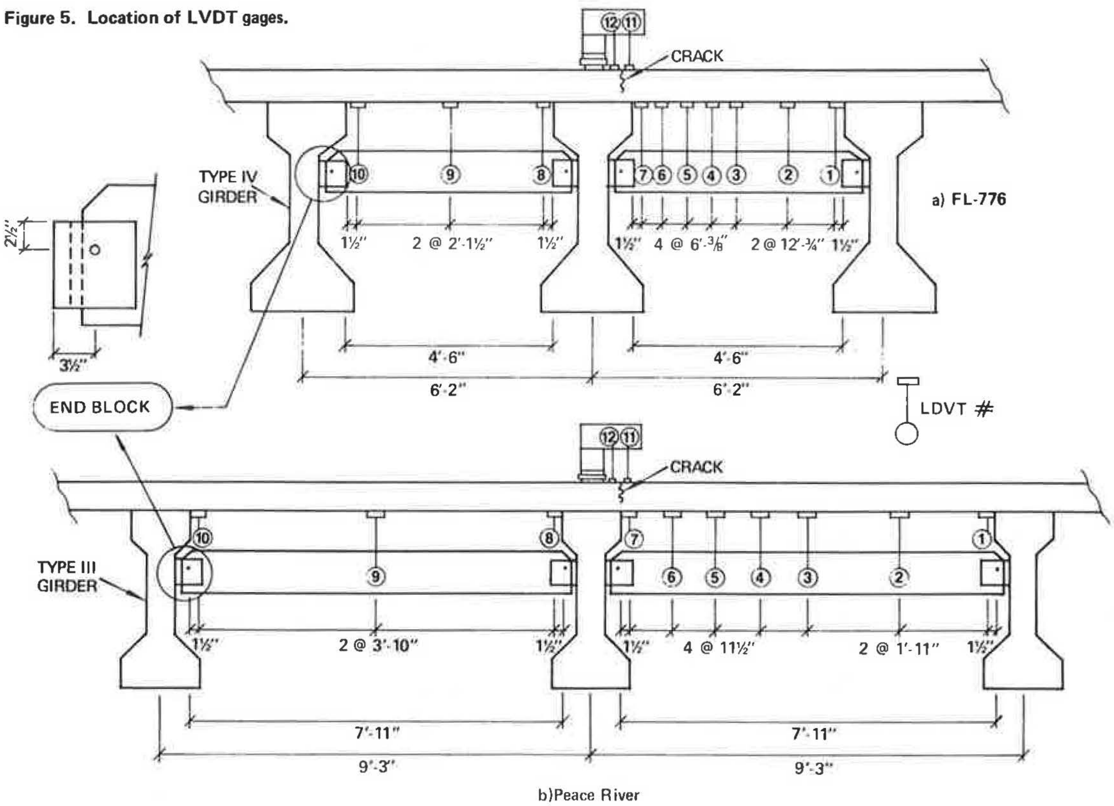
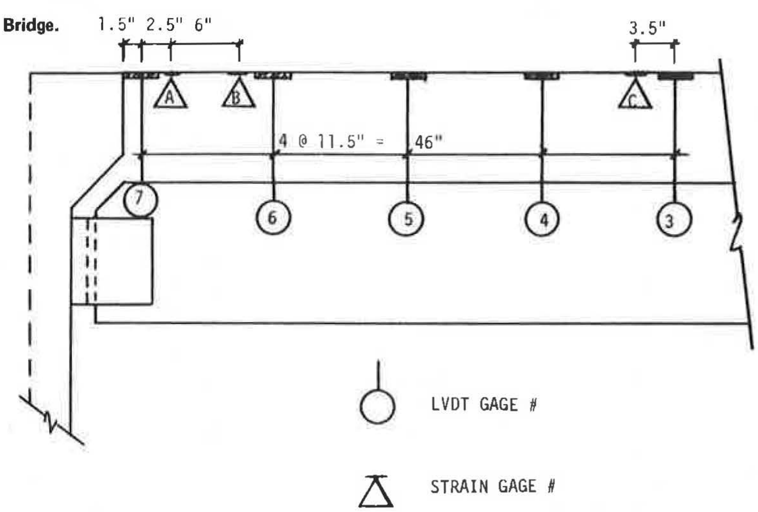


Figure 6. Location of strain gages for Peace River Bridge.

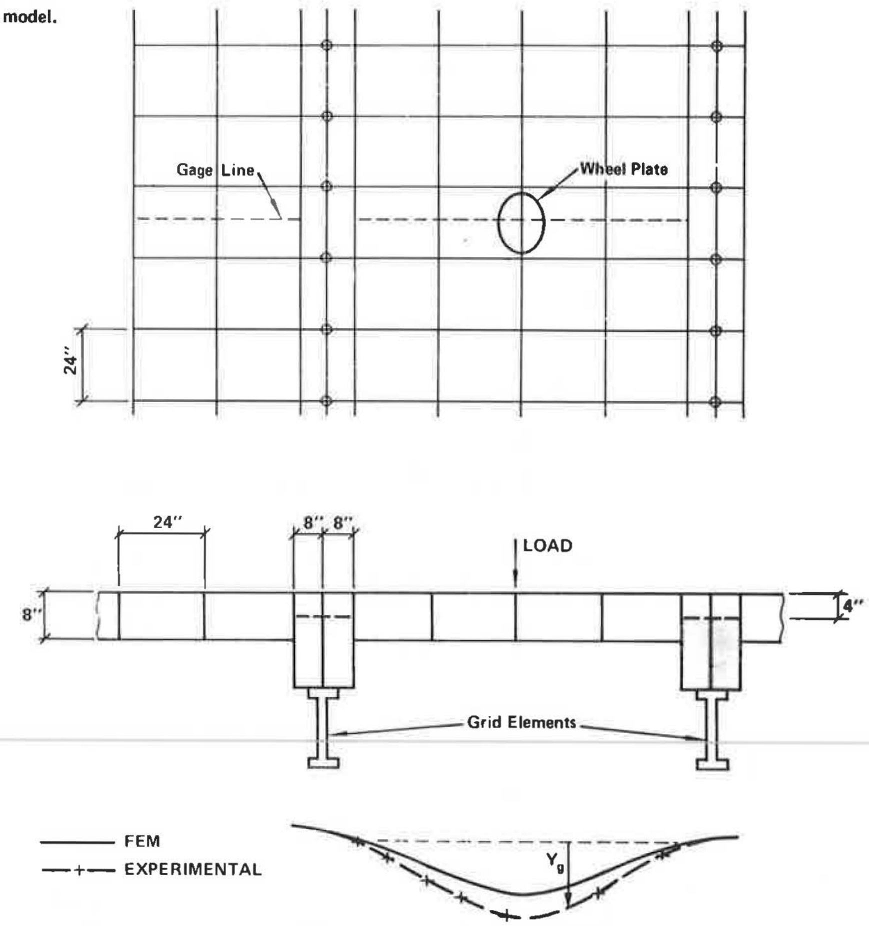


meter, and a 9871A printer-plotter. The 9825A system controller is a 23K desk-top computer that controls and services all peripherals through the HP-IB interface bus. A cassette tape unit and a small thermal printer are built into the 9825A. The 3495A multiplexer is a high-speed scanning device capable of a switching rate of 1000 channels/s. All transducer output signals are received by the 3495A, which relays them to the 3497A voltmeter. The 3497A is a 3.5-digit voltmeter that converts analog signals to digital signals. These signals are then transmitted through the HP-IB to the 9825A for storage and/or reduction. With the 9871A printer-plotter, hard copies of raw or reduced data are obtainable.

Displacements were measured by using Schaevitz LVDTs. Input to the LVDTs consisted of an excitation of ±15 V direct current and a common tied to the ground of the LVDT output signal. The output signals were passed through lowpass filters to reduce noise and alternating current spikes. Each filter consisted of a parallel circuit of two 100-MF capacitors and one 5000-ohm resistor. Each LVDT core was attached to one end of an 8-in-long stainless steel rod. Both input and output cables were approximately 50 ft in length. The input (excitation) cables were three-conductor, 22-gage telephone cables. Output cables were two-conductor, 24-gage microphone cables.

Strains were monitored with either a two-channel Hewlett-Packard 7404A strip chart oscillographic recorder or a BLH 1200B portable strain indicator

Figure 7. Finite element model.



and its accompanying switching and balancing unit (model 1225). A Wheatstone Bridge configuration was used for every active gage with temperature-compensating gages attached to a concrete cylinder maintained under similar environmental conditions as the active gages.

EVALUATION OF FIELD TESTS

Load-Deflection Plots of Hydraulic Tests

Automatic load-deflection plots of all gages for the hydraulic loading tests were made at a small scale to see whether the response to the hydraulic loads was generally linear and to spot any obvious malfunctioning gages. The majority of the load-deflection plots indicated quite linear response.

Transverse Deflection and Moment Profiles

Finite Element Model

In order to have a reference against which to compare the observed data, finite element models were made of the bridges. A given span was modeled as a collection of plate bending and grid (beamlike) elements.

Figure 7 shows a portion of the model for the Peace River Bridge. The pairs of 8-in-wide elements are used over the width of the top flange of the girders. The full model (not shown) has all five girders and a total of 832 plate bending elements. The wider elements were used to represent the slab (precast panel and cast-in-place topping) between girders. An average value of the modulus of elasticity was used for all elements (6).

When the hydraulic load was applied down onto the deck, the reaction to that load was developed by a decrease in the loads on the wheels of the water tanker. These decreases in the wheel loads represent upward loads on the finite element models and were computed by assuming that the trailer acted like a simple beam with supports at the drive wheels and the rear wheels of the trailer unit. This procedure was used because the gages were "zeroed" with the truck in position prior to the application of the hydraulic loads. Because the nearest wheel was approximately 11 ft from the hydraulic load, the upward wheel loads would have almost no effect on the deflections measured relative to the girders but would appreciably affect the absolute deflections of the finite element models.

The thickness of the elements in the vicinity of the girder was increased to account for the torsional stiffness of the girder. This also gives extremely stiff elements (little bending deformation) across the width of the girder. The sum of the moments of inertia of these thickened elements was still below that of the composite girder and slab. Therefore, grid elements were added along the centerline of the girder with moment of inertia such that the combined moment of inertia of the thickened plate and grid elements was equal to that of the composite girder and slab. Grid elements were also used for the diaphragms. If the precast panels and cast-in-place topping were acting as one integral unit, it was felt that the observed results would be in close agreement with those predicted by this finite element model.

When it became apparent that the observed deflections were larger than those from the finite element model and that the field results were indicating much smaller negative moment than that predicted by the model, a new reduced-thickness model was developed to include the effect of a discontinuity at the ends of the panels. The thickness of the plate elements in the girder region was reduced to 4 in, the thickness of the cast-in-place topping. Both the torsional stiffness and the moment of inertia of the grid elements were increased to the values for the composite girder.

This reduced-thickness (thin element) model clearly does not represent the exact conditions in the deck if bond is lost at the end of the panel. It should not be expected that this thin element model could be used to study the detailed behavior of the joint. It does, however, give deflections and moments that are very close to those observed in the field. This close correspondence is used as substantial evidence that the moments are quite small at the edges of the girder.

Reduction of Field Deflections

Most of the field deflection measurements were made from gages supported on the girders. Thus, these deflections are the deflections of the slab relative to the girder. To make the visual comparison of the theoretical and experimental deflections easier, the theoretical girder deflections were added to the field deflections. Figure 7 shows how this was done. The gage support beam is shown as a dashed line. The deflections from the gage support beam (Y_g) are added to the deflections of the gage support beam at the proper point along the axis. The solid line represents the deflected shape of the slab as predicted by the finite element solution, and the + signs and the dashed line represent the deflected shape observed in the field.

Computation of Experimental Moments

The bending moments in the slab were computed by two procedures outlined by Callis, Fagundo, and Hays (6). First, the deflections along transverse gage lines were numerically differentiated by using the finite difference technique. Second, the measured strains were used to compute the bending moments by assuming linearly elastic response and the flexural stress formula to be valid.

Comparisons of Finite Element Model and Experimental Data

Figure 8 shows the type of behavior exhibited by most of the hydraulic tests of the Peace River Bridge. The analysis made by using the full-continuity model indicates both smaller deflections and positive moments than that based on the field data. Figure 9 shows the same field data as Figure 8 but in comparison with the thin element model. Clearly, the correlation between the model and the experimental data is quite good. This type of behavior indicates that the panels are approaching a simply supported condition.

Figure 10 shows results for a test that had strain gage data as well as deflection data. The correlation between the computed moments based on displacement data and those based on strain gage data is quite good.

Tests for the FL-776 Bridge (6) showed larger negative moments near the face of the girder, which indicated that this bridge was behaving more like a continuous deck.

Comparison of static truck tests generally showed better agreement between the continuous solution and the experimental than did the hydraulic tests. This was probably due to the fact that in the truck test the double wheels spread out the load more than did the single-wheel plate used for the hydraulic tests.

Joint Deformations

Gages 11 and 12 (Figure 5) were mounted on top of the deck directly above the girder and approximately 2 in on either side of the longitudinal crack near the end of a panel. The difference in deflection (Δ) between gages 11 and 12 is a good measure of the deformation at the ends of the panel. Because A and B tests generally gave about equal magnitudes of Δ ($\approx$0.0012-in slip for a 32-kip load), the deformation must be a combination of shearing and flexural deformations. The flexural deformations in this short 4-in length are probably due to the hinging action at the ends of the panels.

The panels on the Peace River Bridge that were retested after being grouted showed smaller joint deformations than in the original tests. The A tests that were repeated showed a 28 percent reduction, and the B tests that were repeated showed a 42 percent reduction. A reasonable conclusion would be that the grouting eliminated essentially all of the joint shearing deformation and had little effect on the flexural deformation. The joint deformations were smaller for spans 31 and 32, which had the extra reinforcement in the cast-in-place topping. Span 31, which had the extra transverse reinforcement, exhibited less joint deformation for the B load, whereas span 32 with the extra longitudinal steel showed a decrease for the A and A + 30 loads. Both of these reductions in joint deformations are encouraging.

CREEP AND SHRINKAGE STUDIES

PCA Method

Precast, prestressed girders are sometimes used to make continuous bridges. The continuity is achieved by supporting simple-span girders on piers, adding continuity steel at the piers, and pouring the deck concrete compositely with the girders. The system then acts continuously to support live load. The major portion of the continuity steel is for negative moment at the piers. However, some positive moment steel is added for live load on adjacent spans and the effects of creep and shrinkage. A Portland Cement Association (PCA) publication (7) has been used to design this positive moment reinforcement.

Calculations based on the PCA method (6) indicated that initially high tension stresses would develop on top of the deck due to shrinkage and would be sufficient to cause cracking either by themselves or in conjunction with load stresses; then, with time, creep in the panels due to prestress would cause a separation of the end of the panel from the cast-in-place concrete.

Core Specimens

Several cores were taken from the Peace River Bridge for the purpose of examining the interface between the ends of the prestressed panels and the cast-in-place topping. Prior to these coring operations, epoxy grout was pumped into the cracks around the coring locations by a contractor selected by the Florida DOT. This was done in an attempt to prevent the coring operation from possibly increasing the panel-topping interface separation. After the cores had been examined and photographed, two cores were sawn apart along the transverse panel-topping interface so that the effectiveness of the grouting operation could be examined. Neither of these two cores showed much grout penetration into the interface region. However, several of the other cores showed that the grout reached the interface and in a few instances completely filled up the crack. The separation at the ends of the panel was approximately the same for the cores with epoxy penetration as for those without penetration. In addition, cores taken at the ends of the panels from a laboratory

Figure 8. Deflection and moment profiles from test S6.

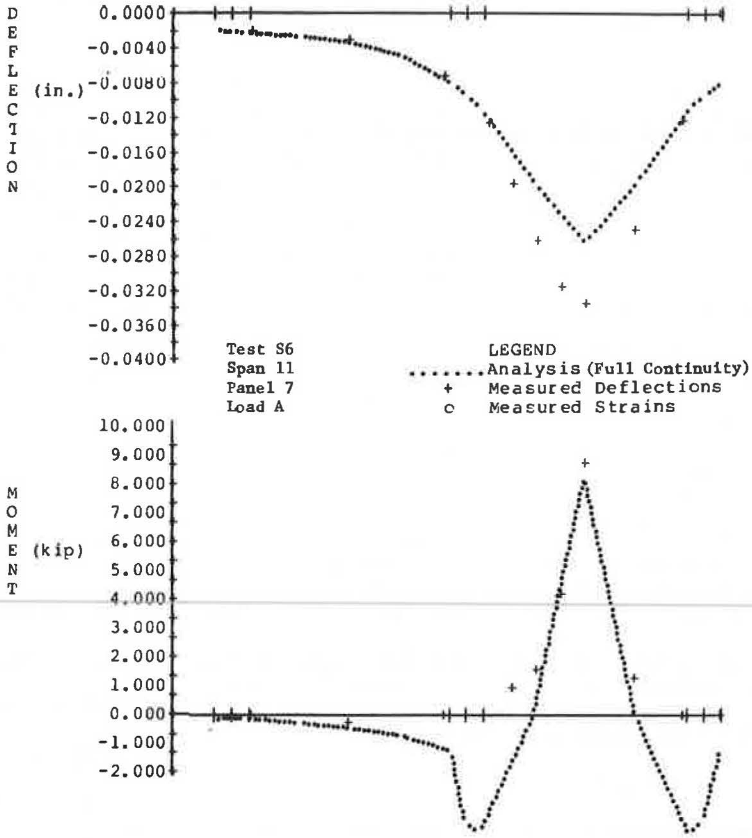


Figure 9. Deflection and moment profiles from test S6 (thin element).

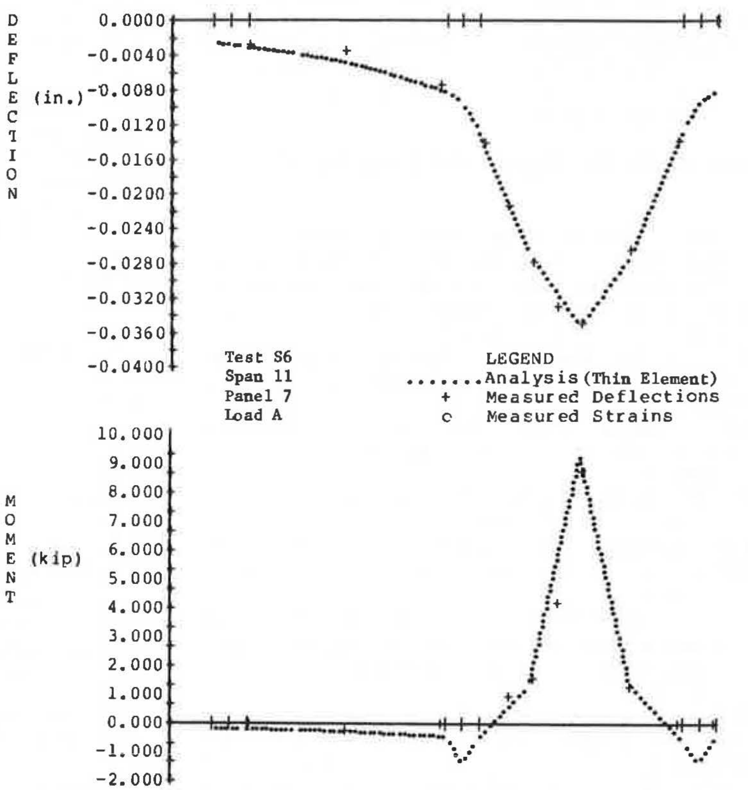
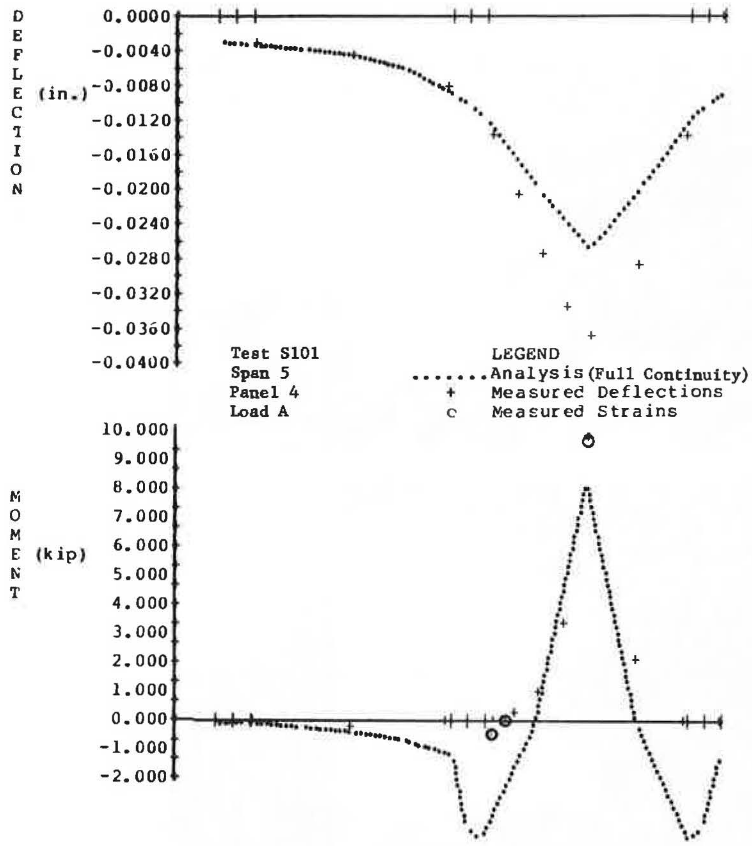


Figure 10. Deflection and moment profiles from test S101.



specimen (6) did not exhibit any separation in spite of the coring operation, which was similar to the field coring procedure except that no epoxy grouting was done. Thus, it is felt that the coring operation does not significantly affect the separation of the panel-topping interface. It should also be noted that the bottom portions of the cores completely separated once the topping was sawed off, which indicated very little bond on the end of the panels.

The photograph at the top of Figure 11 shows core 1. The separation at the end of the panel is about 0.01 in. The lower photograph shows core 3, which had essentially full penetration by the epoxy grout. The thickness of the epoxy appears to be about 0.01 in. The thicknesses of the topping of cores 1 and 3 are only 3.5 and 3.25 in, respectively. However, all of the other cores taken from the Peace River Bridge had at least a 4-in topping. It should also be noted that both cores 1 and 3 had hairline cracks extending about 1 in up the panel-topping interface. The fact that these cracks passed through both paste and aggregate indicated that the crack occurred after the topping had cured.

SUMMARY

An investigation of the Peace River Bridge was conducted in response to reports of extensive cracking in the deck of the facility. This investigation involved testing of the Peace River Bridge and a nearby structure, analytic modeling using the finite element method, and limited laboratory testing of "beam" specimens constructed with panels similar to those used in the Peace River Bridge. Close cooperation between the University of Florida and Florida DOT personnel was maintained throughout the investigation.

The investigation clearly indicates that the behavior of the Peace River Bridge deck is more like that of simple spans than that of continuous ones. This behavior is indicated by comparisons of the field tests and predictions made by using finite element models.

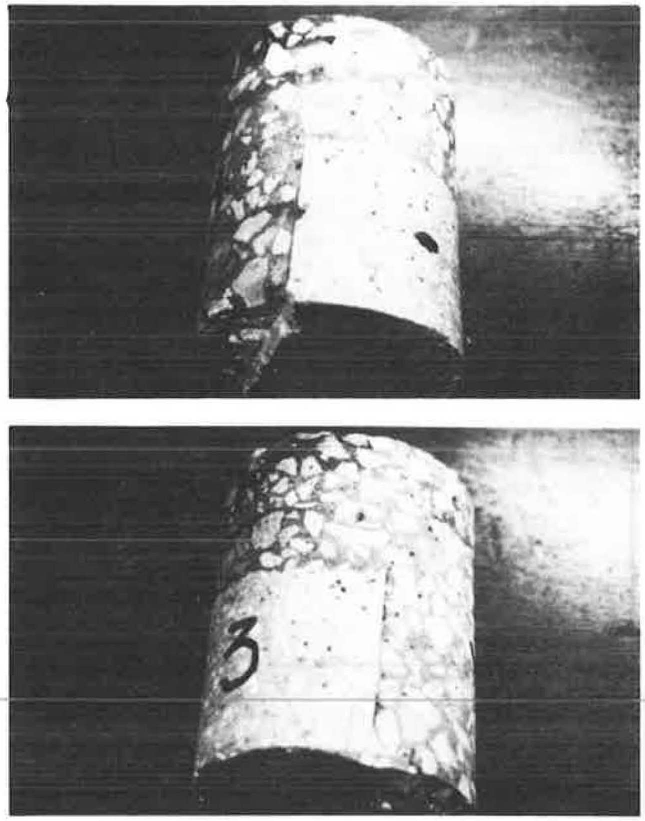
The lack of continuity causes positive flexural stresses near midpanel that are higher than for conventional decks. Analyses indicate, however, that these stresses are within allowable limits. Consequently, this effect of the loss of continuity is not serious. Unfortunately, the loss of bond at the ends of the panels and the corresponding separation of the ends of the panels from the cast-in-place concrete over the girders mean that the shear (in the deck at the face of the girder) must be carried essentially by the cast-in-place topping. This separation has been confirmed by cores taken from the Peace River Bridge and a comparison of the joint deformations measured in the field with the joint deformations of a laboratory specimen (6) that had an artificial bond breaker inserted at the end of the panel.

Creep and shrinkage studies indicate that the most probable cause of the separation of the ends of the panels and the cast-in-place concrete is creep of the panels under the action of the prestress.

The decks that had more transverse and longitudinal steel in the topping concrete than the normal decks exhibited both smaller overall deflections and smaller joint deformations than the decks that had regular reinforcement. This reduction could indicate that the separation of the ends of the panels from the cast-in-place concrete has been diminished and consequently the shear fatigue life would be improved. It is not felt, however, that extra reinforcement is a satisfactory substitute for positive bearing at the ends of the panels.

It appears, based on the relatively few tests of the FL-776 bridge, that this bridge deck slab was

Figure 11. Peace River cores: (top) core 1 and (bottom) core 3.



acting more like a continuous system than the Peace River Bridge deck slab. Because the amount of separation at the ends of the panels is a function of the panel length, the more continuous behavior of the FL-776 bridge is at least partly due to its shorter panel length. Shorter panels are likely to have less prestressing and thus less creep than longer ones. It is very difficult to predict whether this higher degree of continuity will be maintained for the FL-776 facility, particularly since under some load conditions positive moments will be developed at the ends of the panels. However, it appears that panel bridges with deck spans shorter than that of the Peace River Bridge will probably have longer fatigue lives.

CONCLUSIONS

The following conclusions have been made based on the research presented in this paper:

1. The two decks in their present cracked condition are structurally adequate to carry normal traffic. In spite of the simple action of the decks, flexural stresses are not excessive.
2. The shear stresses in the Peace River Bridge are substantially higher than those of conventional bridge decks or panel bridges with positive bearing at the ends of the panels. Because of this, the fatigue life of the Peace River Bridge deck is substantially less than that of conventional bridge decks or panel bridges with positive bearing at the ends of the panels. However, studies are under way that involve extensive field coring of bridges built with details similar to the Peace River facility and laboratory testing to destruction of wide deck specimens. Preliminary studies of this work indicate that, although panel bridges built without positive bearing may exhibit increased cracking and some spalling with time, the punching shear strength is sufficient to prevent structural shear fatigue failures under normal traffic loads.
3. The observed cracking on the top of the deck is probably primarily due to volume changes brought about by differential shrinkage between the panels and the cast-in-place topping. However, temperature changes and live load stresses certainly increase the tensile stresses and the degree of cracking.
4. Adding extra transverse or longitudinal steel is not felt to be sufficient to ensure adequate fatigue life of panel bridges.
5. Removing the fiberboard and replacing it with mortar would greatly increase the fatigue life expectancy of the Peace River Bridge. Whether this action is economically justifiable depends on further studies of the shear fatigue behavior of the bridge under way at the University of Florida.
6. Future panel construction projects should include a detail that provides positive bearing for the panels. Strand extensions may also be useful.

ACKNOWLEDGMENT

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The opinions, findings, and conclusions presented are ours and not necessarily those of the Florida DOT or the Federal Highway Administration.

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