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USE OF PRECAST CONCRETE DECK PANELS
SUMMARY OF RESEARCH
PANEL TYPES I, II, AND III

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STATE OF FLORIDA
DEPARTMENT OF TRANSPORTATION

UNIVERSITY OF FLORIDA
CIVIL ENGINEERING

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16. Abstract

The objective of this research was to investigate the feasibility of using precast concrete deck panels in place of traditional formwork for AASHTO girder type bridges. The precast deck panels span between girders, support the weight of a topping slab, and act as a composite unit with a topping slab in order to resist live loads. This research focused specifically on a new joint details not used elsewhere. A two part testing program was established in order to investigate the structural adequacy of the new systems. The first set of tests examined the strength of the precast panel itself under construction loads, with specific attention to the support detail. The second set of tests dealt with the behavior of the total composite system under service, cyclic, and ultimate loads. All tests were designed to produce the most severe loading conditions at the panel to girder joint.

Three types of panels were investigated. Type I incorporated a notched support scheme at the panel and girder flange joint. In Type II, steel angles were used to suspend the panel from the girder flange (this type was not pursued to a full composite test). Type III, utilized a constant thickness panel supported directly on the girder flange (i.e., no bearing material). In all cases, welded wire fabric was used exclusively for the reinforcement. Results indicate that Types I and III provide viable systems for precast concrete deck panels.

A summary of the test results for all three types of panels is presented in this report. Detailed information on the full testing program for each panel type can be found in the references given at the end of this report. These references are available from the Florida Department of Transportation

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Prepared in cooperation with the State of Florida Department of Transportation and the U.S. Department of Transportation.

SUMMARY OF RESEARCH

Introduction

Currently, the Florida Department of Transportation (FDOT) uses cast-in-place (CIP) concrete decks on girder type bridges. The use of CIP concrete decks results in a relatively high cost for temporary formwork to support newly placed concrete. The purpose of this research was to investigate the feasibility of using reinforced precast concrete deck panels in place of the traditional formwork used in CIP construction. The precast panels would span between girders supporting the weight of a CIP slab and equipment during the construction phase. Once the CIP deck cured, the system would act compositely in order to resist vehicular live loads.

As a pioneer of new bridge systems, the FDOT utilized this construction technique in the 1970's. As a result of some cracking problems associated with joint details between the precast panels and girders, these systems were abandoned by the FDOT in the 1980's. This girder-panel detail within the negative moment region was the main concern in both the design and testing of several different panel systems.

Objectives

The first objective was to design a number of precast panel options in coordination with the FDOT. The precast panels were designed in accordance with the

1994 LRFD Bridge Design Specifications (LRFD) published by the American Association of State, Highway, and Transportation Officials (AASHTO). Four of the proposed panels were selected for testing.

A two-part testing program was established in order to investigate the structural adequacy of each new system. The first set of tests examined the strength of the precast panels themselves under construction loads. The second set of tests dealt with the behavior of the total composite systems under service, repeated, and ultimate loads. These tests involved the construction of full-scale composite sections in the laboratory. The tests centered on the known cracking problems within the negative moment region observed in some precast deck form systems. All of these tests were ultimately used in the overall evaluation of the new bridge deck systems.

Precast Panel Options

Four separate precast panels were originally designed and constructed. These four panels were tested as separate units simulating in-field construction conditions. All panels were reinforced with welded wire fabric. Prestressing was not used in any of the panels. The first panel, Type I, was comprised of notched ends in both the panel and the girder (Figure 1). This design contained two separate reinforcement details within the notch of the precast panel. Reference 1 provides complete details for panel Type I. The second option, Type II Option 1, used two, continuous, double angle devices as support mechanisms (Figure 2), while the third option, Type II Option 2, used four independent single angles as supports (Figure 3). A fourth option, Type III, was a simple rectangular precast panel placed directly on the girder (Figure 4). This differed from the previous design used by the FDOT that incorporated felt bearing pads between the panel and the girder. Each panel had a span of 2440 mm (8 ft) and a width of 1830 mm (6 ft). Reference 2 provides complete details for the Type II and Type III panels.

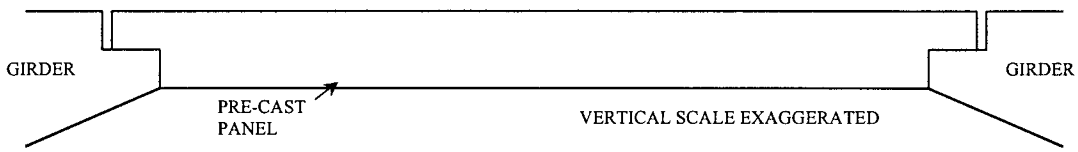

Figure 1. Type I – Notched Supports.

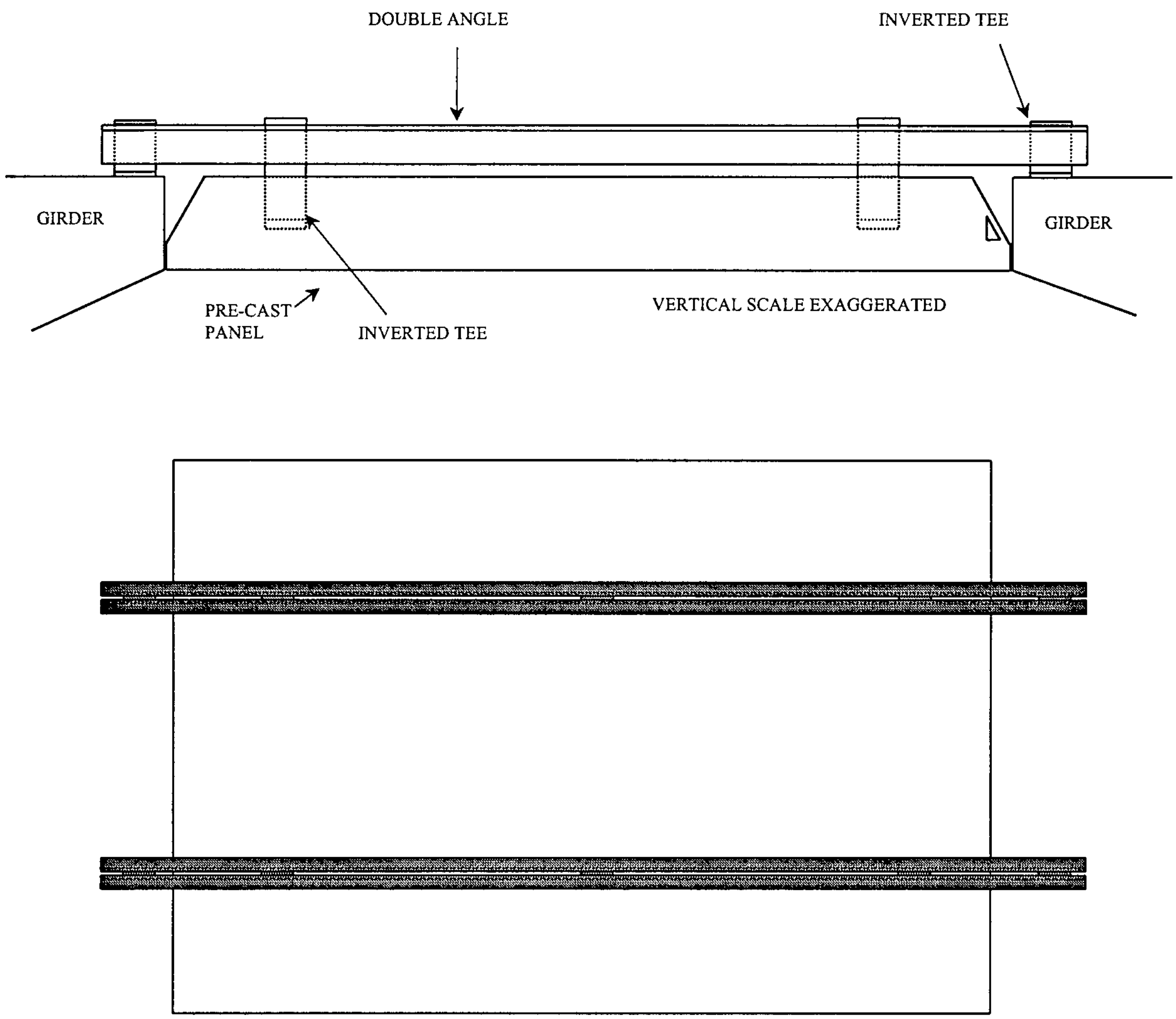

Figure 2. Type II Option 1 – Double Angle.

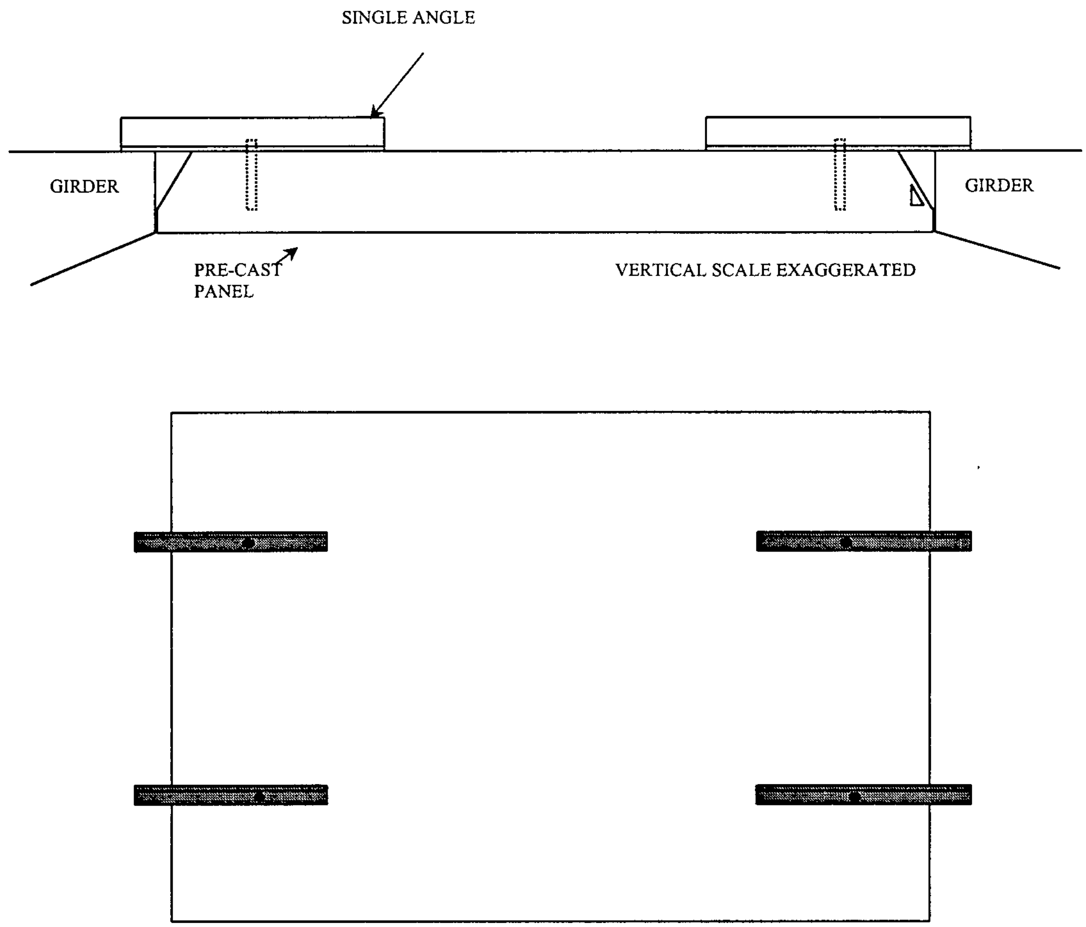

Figure 3. Type II Option 2 – Single Angle.

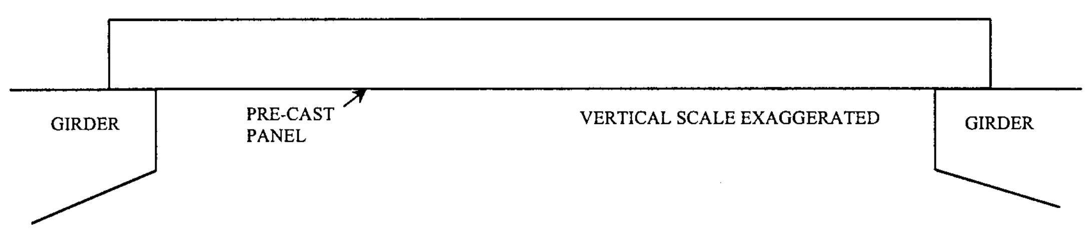

Figure 4. Type III – Rectangular Panel.

Panel Tests

Each panel was tested using the same setup in order to accurately compare the results. The complete test setup can be found in the attached volumes. Although each panel experienced favorable results, only two of the four panel options were chosen for further investigation. Both options of the Type II panel carried an increased economic factor due to the amount of materials and labor required for construction. The lack of redundancy in the single angle design was also a contributing factor in the elimination of the Type II panel. Panel Types I and III were selected for additional testing. A sample of the results for both Type I and Type III can be seen in Figures 5 and 6 respectively. Detailed results for panel Type I can be found in the Reference 1 while detailed results of panel Types II and III can be found in Reference 2.

The results were expressed in terms of typical construction loads. These loads refer to the weight of the CIP deck and a construction load of 2.4 kPa (50 psf). The load ratio, $P/P_{construction}$, depicted in Figures 5 and 6 represents the applied load during testing divided by the typical construction loads. The two panels chosen for further testing were used in the construction of two separate, full-scale, two-span continuous bridge sections.

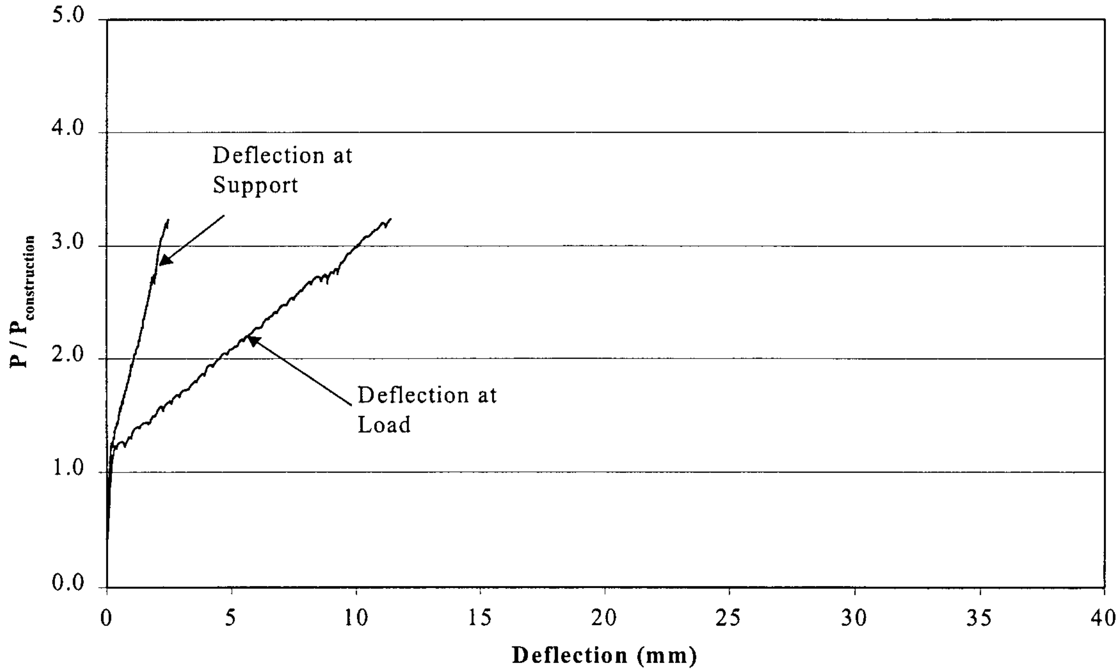

Figure 5. Panel Tests – Sample Results for Type I Panel.

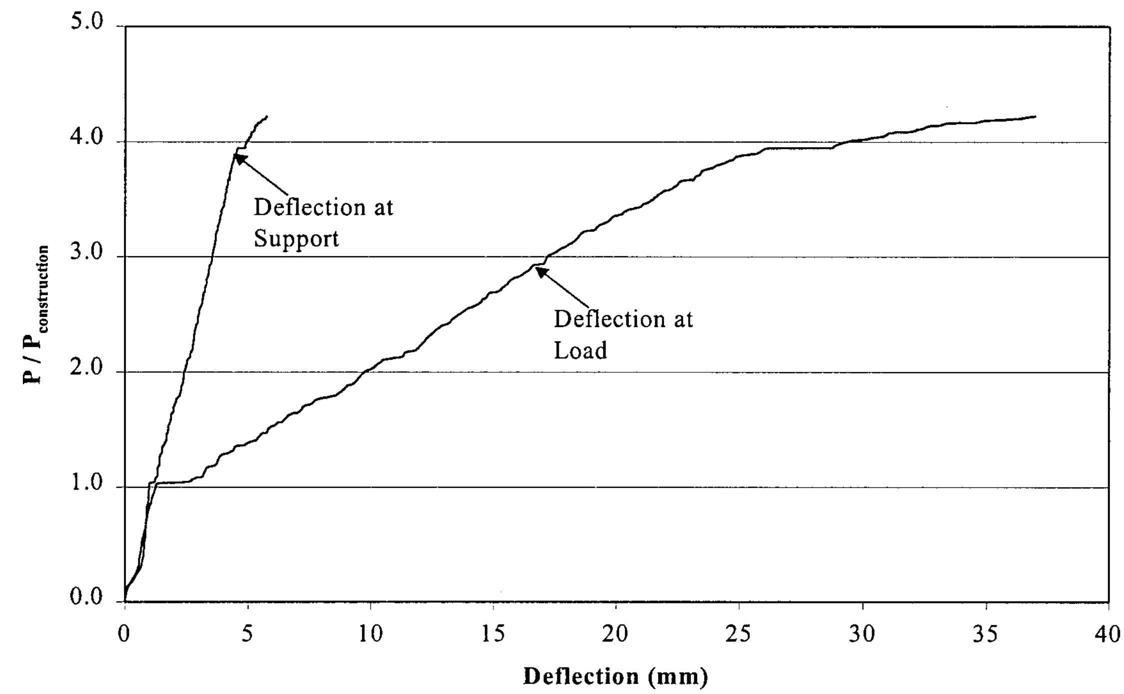

Figure 6. Panel Tests – Sample Results for Type III Panel.

Composite Sections

Each bridge section was constructed of three modified AASHTO Type V girders, two precast panels, and a CIP deck. The girders were spaced at 3500 mm (11.5 ft). The precast panels were 100 mm (4 in) thick and spanned 2440 mm (8ft). The CIP deck was 130mm (5in) thick and each section measured 1830 mm (6 ft) in width. Figure 7 shows a schematic of each option.

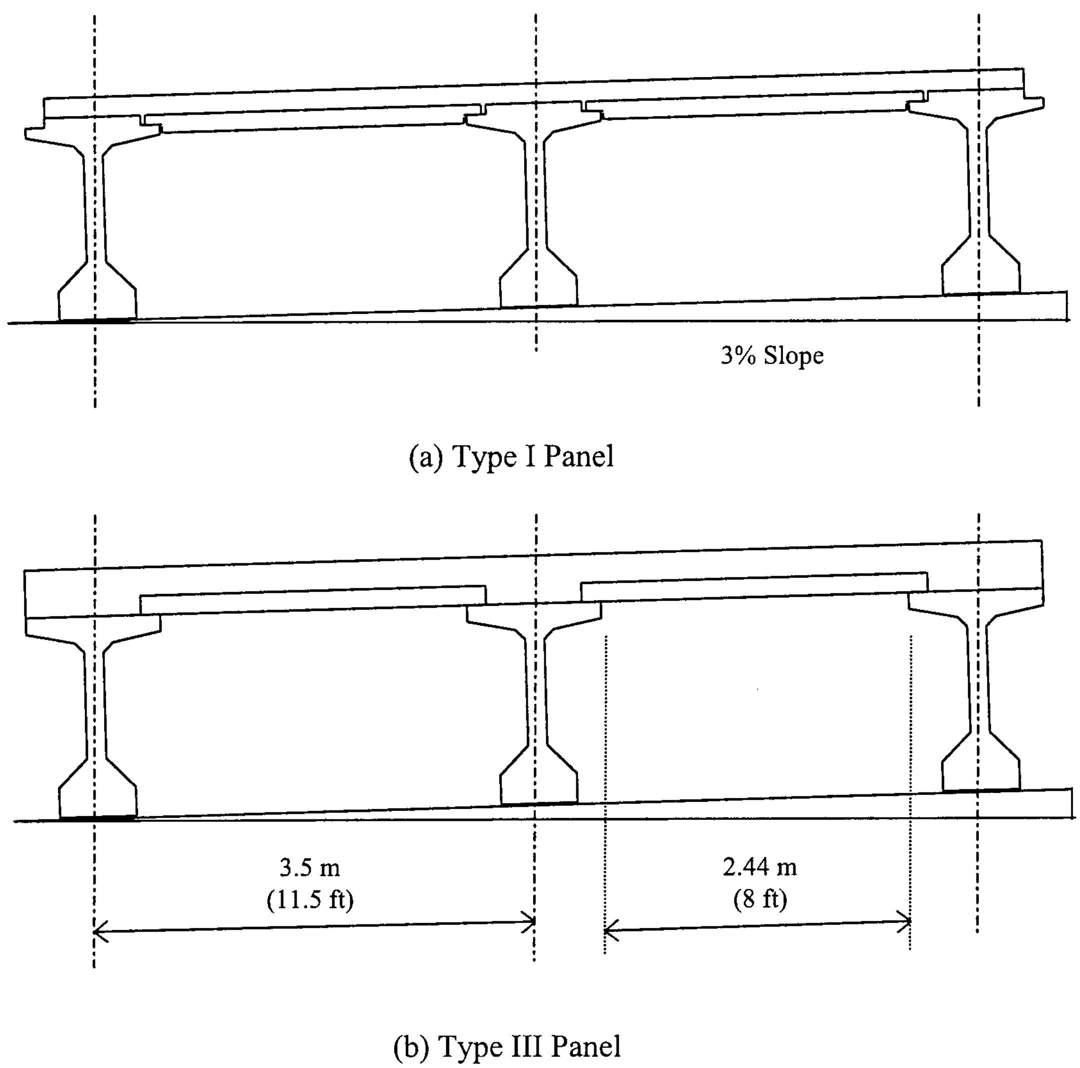

Figure 7. Schematic of Full-Scale Composite Sections.

Through the request of the FDOT, the bridge was placed on a 3% slope to simulate drainage and superelevation conditions. This was achieved by using sloped concrete blocks of increasing heights underneath each girder. Once constructed, the sections were subjected to several tests.

Full-Scale, Composite Section Tests

The bridge sections, as composite systems, were subjected to three separate tests beginning with a two-part static service load test. A second test involved repeated loading, and a third test examined the ultimate strength of each bridge deck system using a one-point and two-point loading system. Although the applied loads for each of the three tests varied, the loading configuration remained the same except for the final part of the ultimate load test where one-point loading was incorporated.

The main focus of the composite tests was the negative moment region at the interior support near the girder face. It was decided at the start to load each bridge with either a one-point or two-point loading arrangement to produce a negative moment at the interior support that would be equivalent to the moment caused by an AASHTO truck load configuration.

Numerous finite element models were developed to examine the differences between a one-point loading configuration and a two-point loading configuration. The models ranged from simple constant area models with single point supports to more complex models that modeled the actual girder supports. Depending on which model the bridge followed, the single load configuration could yield a variety of moments at the girder face. The two-point loading configuration had less variation from one model to the next within the negative moment region and was therefore used throughout the composite tests.

The main load was applied at the center of the interior girder (Figure 8). This load was distributed to the center of each span using a spreader beam. This smaller load on each span was then distributed again using smaller spreader beams. Two steel plates were used to apply the load directly to the bridge deck. A layer of fiberboard was placed between the bridge deck and the steel plates to minimize concentrated bearing stresses.

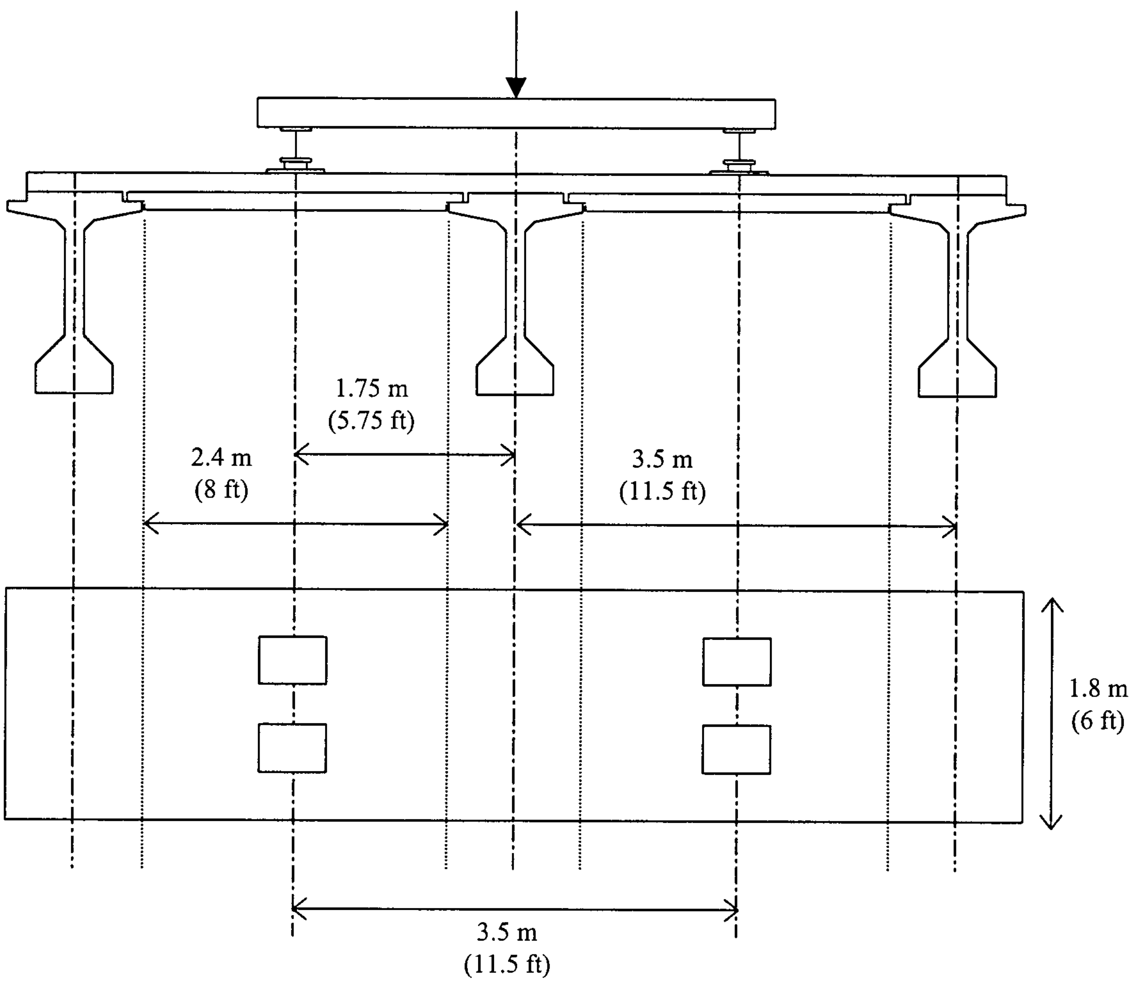

Figure 8. Load Configuration.

Service Load Tests

The service load test actually consisted of two tests. The first test, or pre-service load test, loaded the bridge to an estimated 0.50 of the service load in order to obtain strain data while the concrete was still in the linear range. The pre-service load test data was used to determine which finite element models best represented the bridge behavior. From the finite element models, the loads needed to produce a negative service moment at the face of the interior girder supports were determined. The bridges were then loaded to 100% the service load based on the load-strain data obtained in the first test. The strain data for each span of the bridge was then plotted against a ratio of load to service load (Figures 9 and 10).

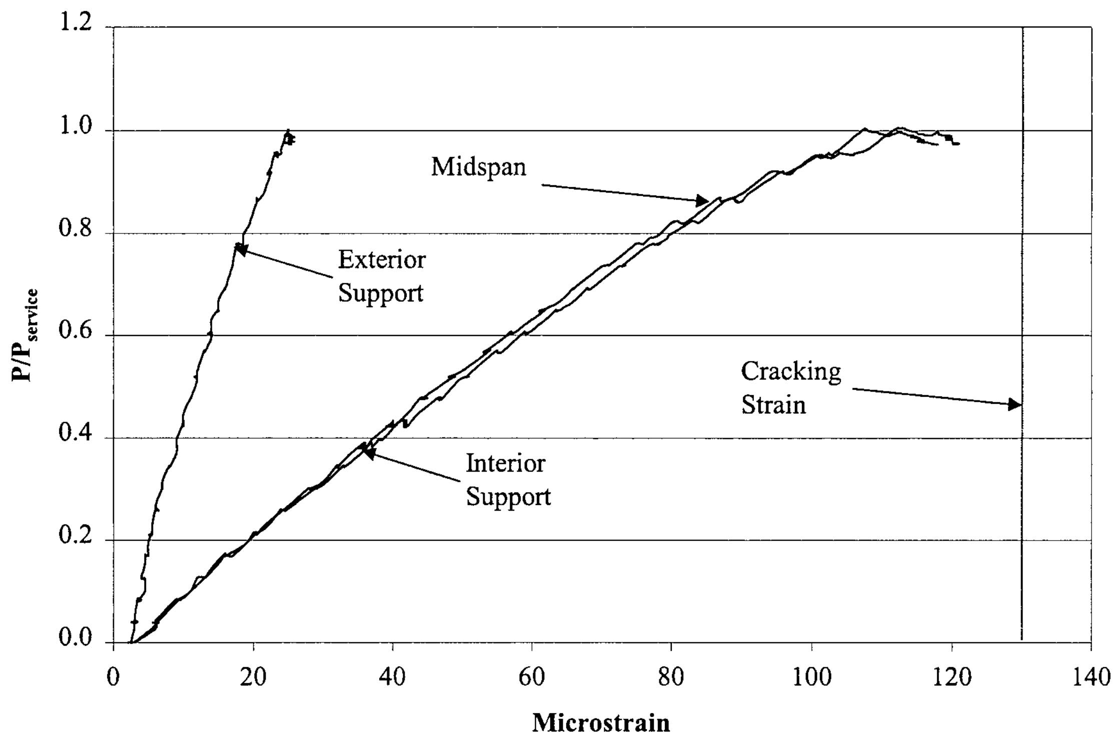

Figure 9. Service Load Test – Average Tensile Strains for Type I.

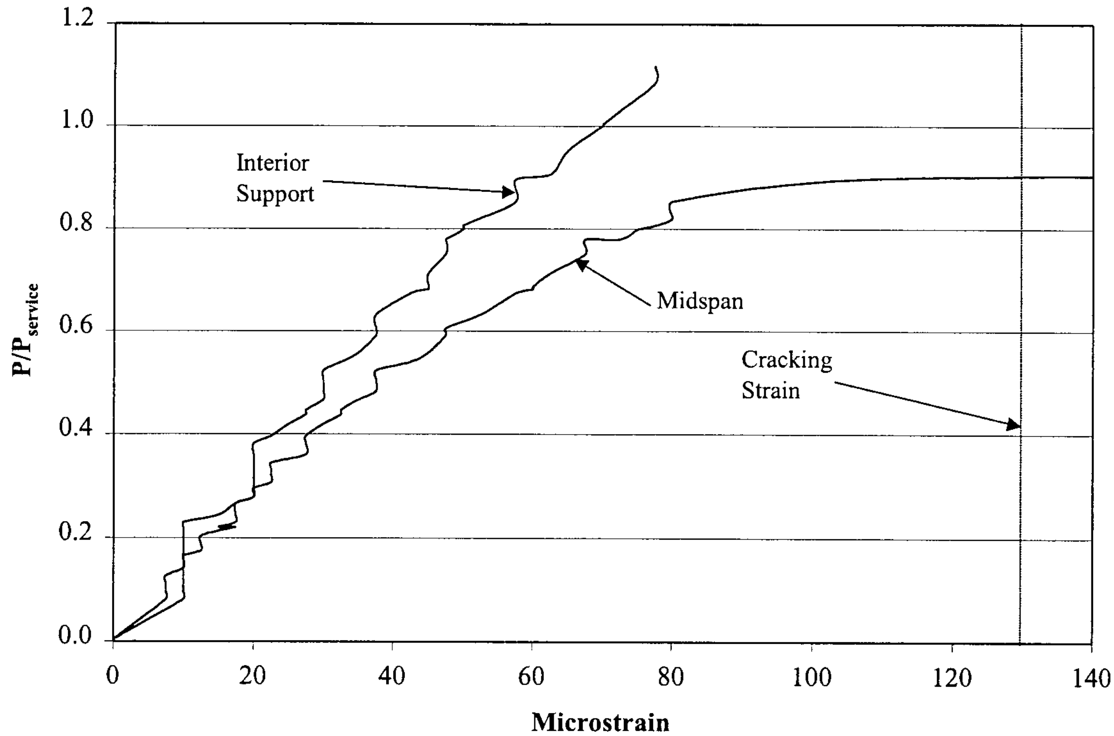

Figure 10. Service Load Test – Average Tensile Strains for Type III.

Both panel types faired well during the service load test. The Type I bridge system exhibited no cracking during the test. The Type III bridge system revealed a small crack underneath the load point that was represented in the strain gage readings but not in the deflection readings. The bridges appeared to act similarly during the service phase of the composite testing.

<u>Repeated Load Tests</u>

As mentioned in earlier, cracking at the support was a major problem in some earlier panel systems. These earlier panels experienced cracking after they had been in service for a number of years. Repeated load tests were performed to investigate whether these new composite systems would be affected in the same manner.

The load used for the repeated load test was based on a one-truck loading scheme. It was decided that the two-truck configuration was unrealistic considering the load was going to be applied for two million cycles.

The bridges were periodically examined for cracking and when discovered, cracks were mapped and logged. Cracks began to form on the underside of each bridge at the midspan near 250,000 cycles. These cracks initiated from the edge of each bridge and propagated toward the center of the cross-section as the cycles continued. While no cracks formed at the girder panel interface on top of the bridge in the Type I composite bridge system, the Type III bridge deck did exhibit some cracking similar to that experienced at the midspan of the bridge. The cracks did not exceed 0.1 mm and closed once the loading was complete. The top portion of the bridges only exhibited minor cracking during the entire two million cycles.

The deflection readings taken prior to testing during the service load tests were compared to the results taken after the repeated load test during the ultimate load test. The two sets of data were plotted on the same graph to determine whether there was any reduction in stiffness (Figures 11 & 12). The graphs show a small reduction in stiffness for the Type I bridge system and no reduction for the Type III due to cracking on the bridge. After the completion of the repeated test, only the ultimate load tests remained.

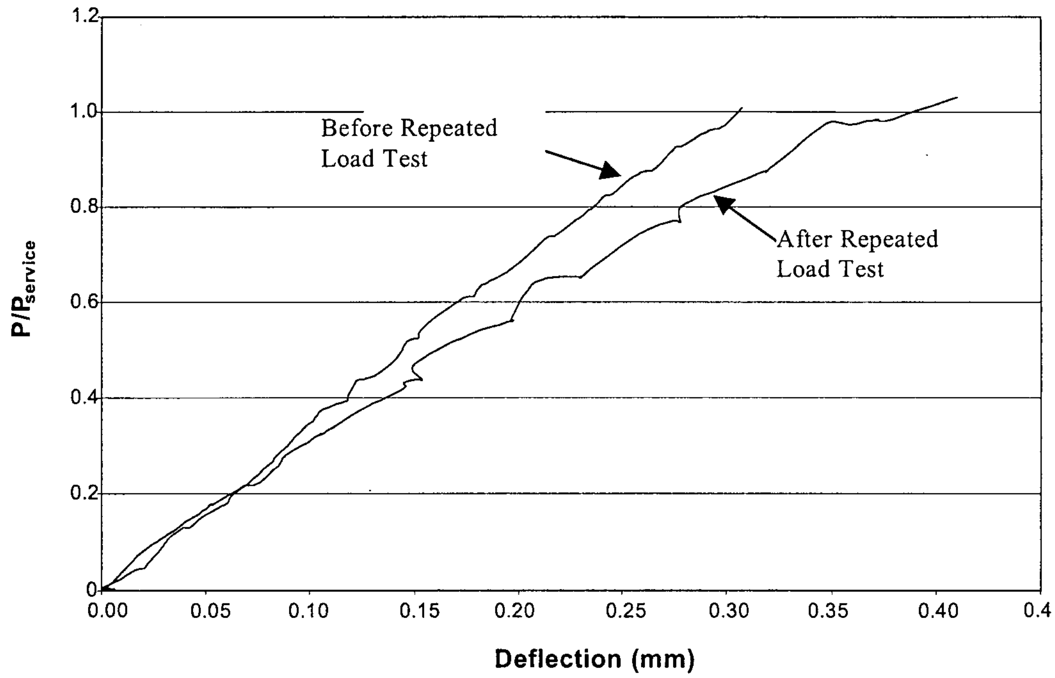

Figure 11. Repeated Load Test – Stiffness Comparison for Type I.

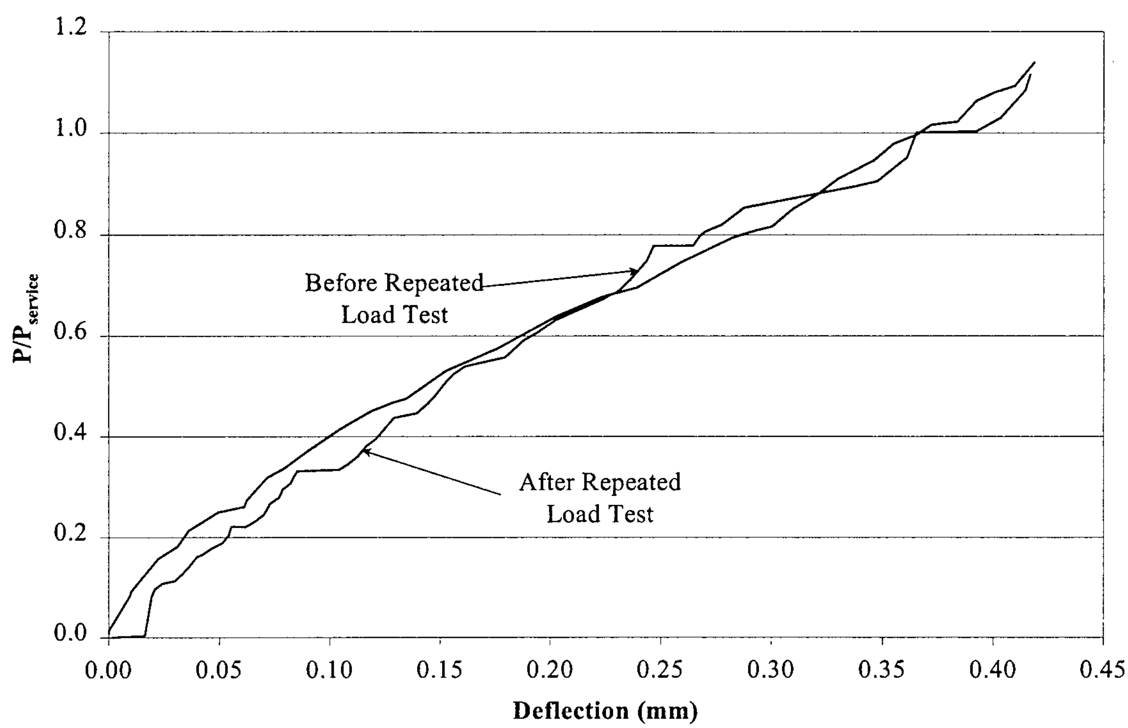

Figure 12. Repeated Load Test – Stiffness Comparison for Type III.

Ultimate Load Tests

Yield line and one-way shear analyses were performed prior to testing in an attempt to predict the ultimate failure loads. The bridges were loaded to over 4 times the service load without reaching failure. The crack widths were measured, logged, and are listed in the attached volumes. The loading on each bridge was halted when the capacity of the loading ram was reached. Deflection data can be viewed in Figures 13 & 14.

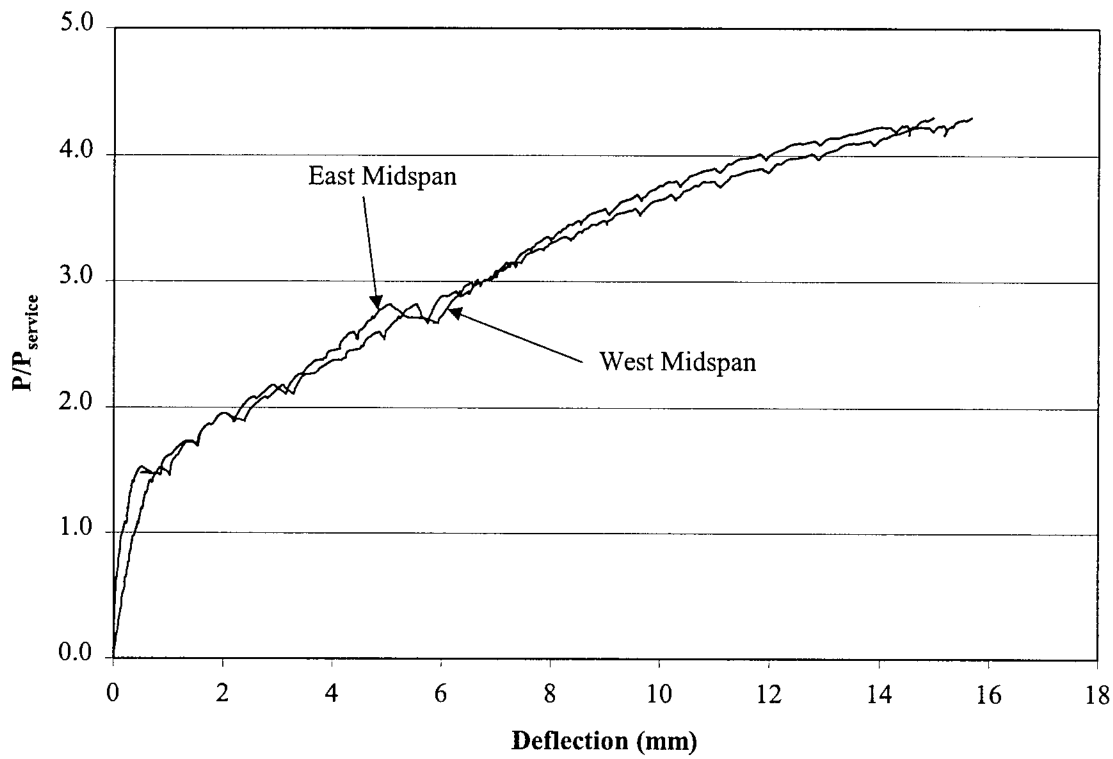

Figure 13. Two-point Ultimate Load Test – Deflection Data at Midspan for Type I.

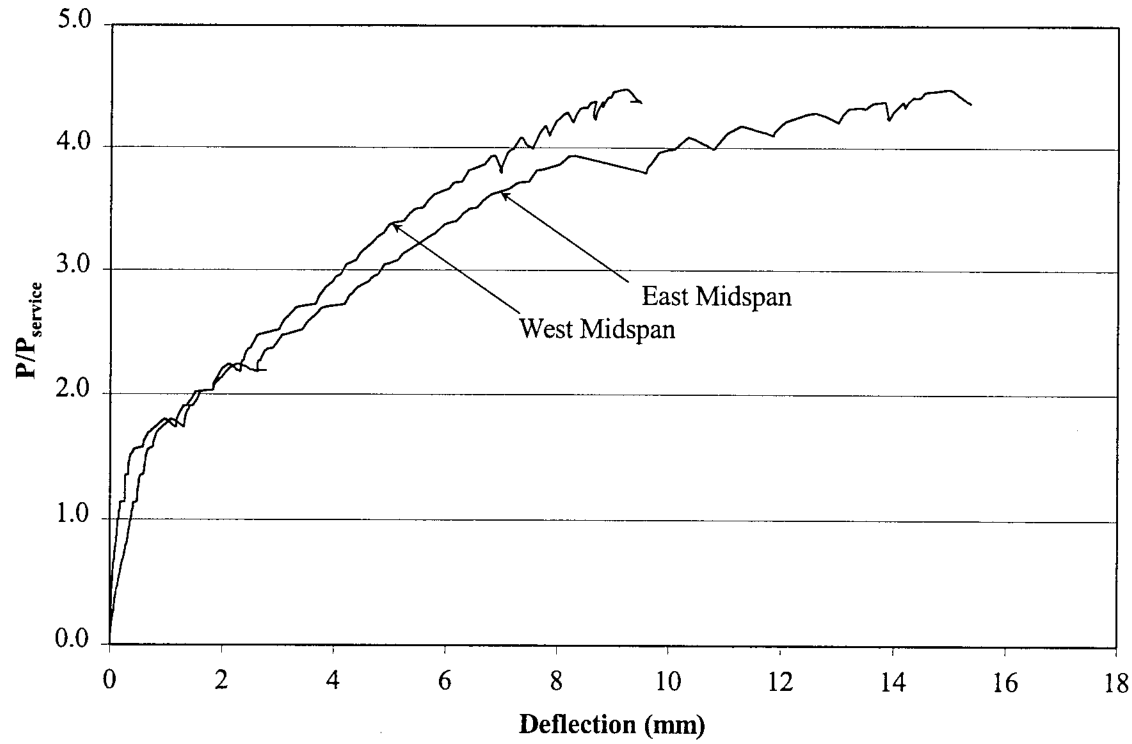

Figure 14. Two-point Ultimate Load Test – Deflection Data at Midspan for Type III.

The deck systems remained as composite units throughout the test. There was no evidence of delamination between the two separate layers of concrete. Cracks traveled through each layer as if they were a solid unit. The applied loads induced torsion at the outside girder supports and in turn developed flexure in the web of the girders. The outside girders cracked between 1.5 and 2.0 times the service loads. The cracks first started at the intersection of the web and the flange. Additional cracks formed at the middle portion of the web at higher loads. Additional pictures and data can be found in the attached volumes.

The loading setup was rearranged in an attempt to load the bridge to failure. A one-point loading scheme was selected in order to attain twice the load previously attained using the two-point loading scheme. Using the one-point loading scheme, both bridges were loaded to failure. Due to large deflections, the LVDT needed to be lowered each time the permanent midspan deflection reached the maximum length of the LVDT. Once the maximum length was reached, the bridge was unloaded, the LVDT was lowered, and the bridge was reloaded. Figures 15 and 16 show the relationship between the load ratio, $P/P_{service}$, and the deflection at the midspan of the loaded span. As in previous graphs, the load ratio represents the relationship between the applied load and the equivalent service level load for one span. The final load level was 6.2 times the service-level load for the Type I bridge system and 6.6 times the service-level load for the Type III bridge system.

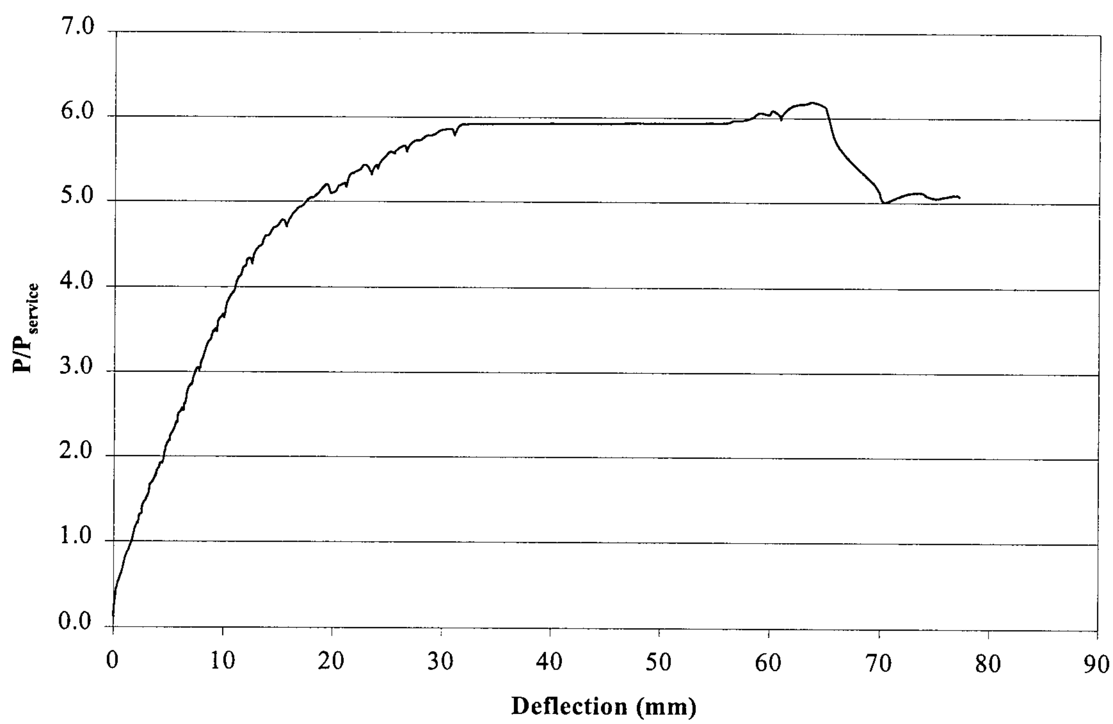

Figure 15. One-point Ultimate Load Test – Deflection Data at Midspan for Type I.

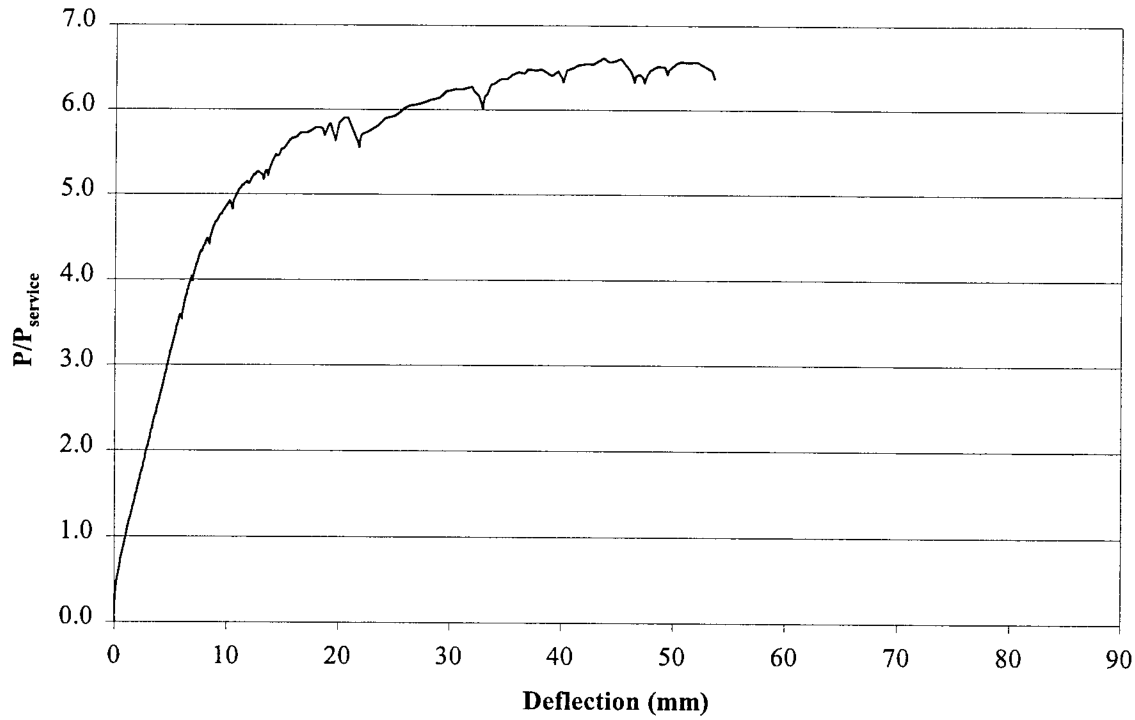

Figure 16. One-point Ultimate Load Test – Deflection Data at Midspan for Type III.

Summary

The objective of this project was to design, construct and test a number of precast panel systems. Initial designs were established in coordination with FDOT and four panels were constructed and tested. The panels were reinforced with welded wire fabric and were not prestressed. Tests proved the precast panels could easily withstand the factored construction load. Two final designs emerged after careful analysis of the initial panel test results. The new designs were constructed and used in full-scale, composite bridge cross-sections. These composite systems were then subjected to service, repeated, and ultimate load tests. Each bridge system supported over six times the service-level loading before catastrophic failure occurred. See Reference 1 and 2 for detailed results.

Conclusions

The following conclusions resulted from this research:

1. The proposed panels had sufficient shear and flexural strength to resist factored construction loads that included the self-weight of the panel, the self weight of the cast-in-place (CIP) deck, and an additional construction live load.
2. The welded wire fabric (WWF) reinforcement used in the notch detail of both the panel and girder support, for Option 1, provided adequate shear reinforcement while satisfying cover requirements.
3. The WWF used as flexural reinforcement provided adequate flexural resistance for the precast panel under construction loads without violating over-reinforcement stipulations.
4. The use of WWF decreased both the amount of time and number of workers needed to install reinforcement.
5. The panel supports provided a rigid, incompressible bearing surface.
6. A raked finish induced composite action between the panels and the CIP layers with no signs of delamination.
7. The composite systems, as a whole, were successful in preventing cracking at the support detail, with credit going to both the joint detail and the use of WWF.
8. Arching action within each composite system contributed to an increase in the ultimate strength of each system above that predicted using conventional reinforced concrete beam theory.

Recommendations and Further Considerations

1. The FDOT should consider the new panel system as a viable alternative to the bridges currently constructed using traditional formwork.
2. Additional tests need to be conducted to investigate the discontinuities between adjacent panels combined with the continuous CIP deck above. Possible options include slanting the sides of the panel forming a V-shape wedge when placed next to each other (Figure 17) or using a notch interface similar to the end supports from Option 1.
3. The bridge constructed in the laboratory had girders that were slanted to represent a pier cap with a 3% slope. Some piers are formed in a step-like pattern leaving the girders perpendicular to the ground but at different heights (Figure 18). Unlike the sloped pier caps, the stepped pier caps cause the precast panels to lie at an angle on the girders. By inducing a chamfer of 19mm ($^3/_4$in) at the ends of the panels, the concentrated stress in the contact corner may be alleviated (Figure 18). Because the angle of the bridge tested was so small (1.7 degrees), the chamfered ends may not be necessary, but bridges at steeper grades will present a problem.
4. In conventional bridge construction, the thickness of the CIP deck varies with the camber of the girder. Figure 19 depicts the placement of short (2.0m, 6.0ft) panels side by side on top of a prestressed girder. The weight of the precast panels may aid to diminish the camber of the girder allowing a more uniform thickness for the CIP deck.

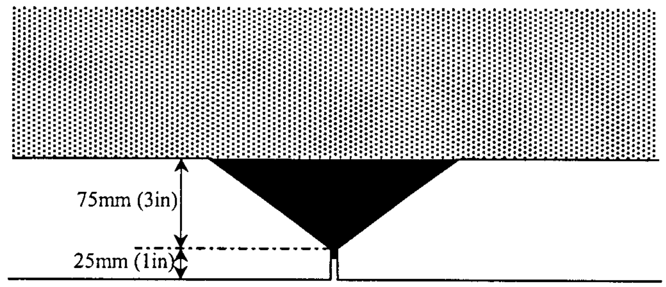

Figure 17. V-Shaped Interface.

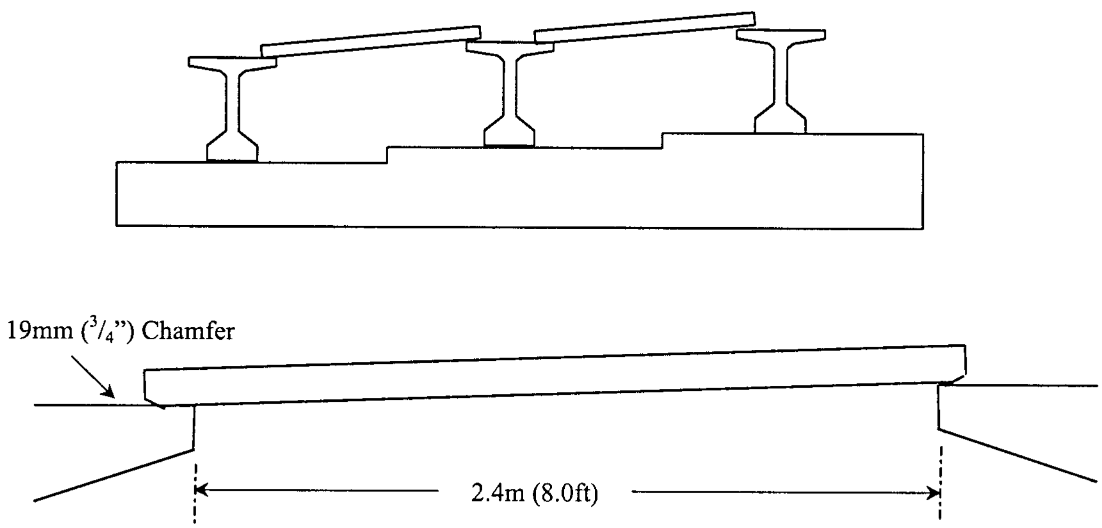

Figure 18. Chamfer Solution.

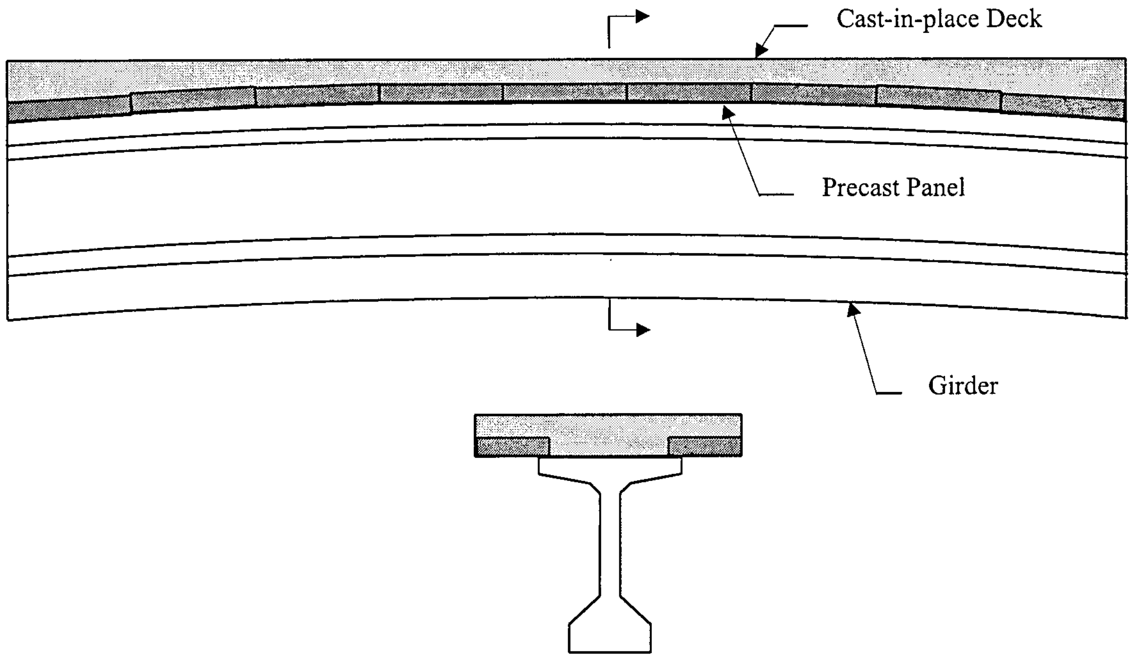

Figure 19. Camber in Girders.

References

1. *Use of Precast Concrete Deck Panels, Panel Type I (Notched Support)*, FDOT/UF Structures Research Report No. 99-4b, June 1999, 132 pages.

2. *Use of Precast Concrete Deck Panels, Panel Type II (Steel Support) Panel Type III (Constant Thickness)*, FDOT/UF Structures Research Report No. 99-4c, June 1999, 147 pages.