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STRUCTURAL TESTS OF PRECAST DECK PANELS

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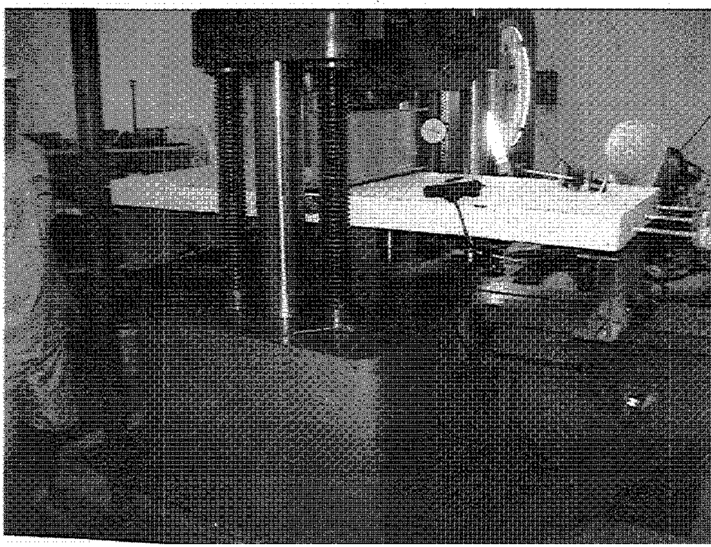

Figure 2-3. Isometric view of overall test setup (Test 1 precast slab shown)

Final Report on Research Sponsored by GMR, JV

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1.0 INTRODUCTION

In order to reduce the cost and construction time of the SR-22 HOV Widen Design/Build Project, contractor GMR and designer PBS&J are recommending the use of precast, prestressed deck panels with cast-in-place (CIP) topping for about 30 bridges. Deck panels will span between girders, placed one after another on camber strips, followed by the CIP concrete slab pour across the whole bridge deck (CIP slab is both structural and the final riding surface of the deck). To demonstrate the capability of the completed deck units (precast deck panels with cast-in-place topping and no rebar crossing the plane) to act compositely in flexure, with no longitudinal shear slip between slabs, GMR has sponsored San Diego State University (SDSU) to perform a series of full-scale structural tests and prediction analyses of this deck system. They consisted of precast deck panels, with various roughening techniques (Coarse Broom, Medium Broom and Carpet Drag), and a cast-in-place topping slab. Precast and CIP slab details are provided in [1]. The test units are considered full-scale because the full depth and full width (distance between face of girders) of the prototype slabs are included. Rather than the full length of the slab, a representative 2-ft length (along the girder line) is used. This reduced length of the slab along the girder line has no effect on its overall response and failure, with the same amount of prestressing steel and rebar per-ft-length as the prototype [1]. For any given displacement level the force per-unit-length is the same.

Structural tests of 5 precast deck panels with CIP toppings have been conducted at the SDSU Structural Engineering Laboratory to verify that longitudinal shear transfer between slabs is sufficient to prevent interface slip, and enforce composite flexural behavior to failure. To maximize interface shear demand for a given moment (worst possible loading scenario), the tests were conducted in simple bending, with the load applied at mid-span and reactions provided at either end of the precast deck panel. The tests investigated different finishes at the top of the precast panels, with the 1st benchmark test having no CIP topping. One of the tests was conduced in negative bending (upside down) with a Coarse Broom finish. Loading and reaction I-beams were used to ensure that the full slab width (2 ft) was effective throughout the test to failure.

Including the precast and CIP slabs, the total size of each test specimen was 7' long, 2' wide and 7-½" thick (precast slab dimensions are 5'- 4" long, 2' wide and 3-¼" thick). Precast deck slabs were cast by Pomeroy Corporation, at their Petaluma plant, and shipped to the bridge site in Orange County for the CIP toppings to be added by the contractor (GMR, JV). After curing, completed deck units were shipped to the SDSU Structural Engineering Laboratory for testing. Included with this shipment to SDSU were prestressing strand and rebar samples, as well as 5 concrete cylinders taken from the CIP slab pour. Pomeroy took cylinders from the precast pour and tested them on the days that the decks were tested.

2.0 TEST SETUP

The tests were conducted in simple bending with a point load applied at mid-span and reactions at the ends of the precast slab (Figures 2-1 and 2-3). As the loading head diameter and reaction table width were less than the 2 ft slab width, relatively rigid steel I-beams were designed to spread the applied load and reactions evenly across the section width. Vertical stiffeners were added to the I-beams at added to the I-beams at critical load points (under the loading head and at the sides of the reaction table – see Figure 2-1 and 2-3). At the reaction locations the 2-ft long steel I-beams were bolted down directly to the 10-inch wide loading table. Along the top of the 2-ft wide I-beams, 2-inch wide by ½-inch thick elastomeric bearing pads were secured with double-sided tape. The completed deck slabs were painted white (so that crack marking with felt pens during the tests would be clear) and accurately marked for positioning within the test setup.

A given test unit was lifted with a crane over the upper-floor lab rail, lowered into the testing pit and onto over-sized rollers that were temporarily placed on the reaction table. Straps were removed and the deck slab was rolled into approximate position by hand. The straps were then wrapped around the test specimen and cross-head of the testing machine, allowing the deck slab to be lifted by the machine so that the rollers could be removed. The cross-beam was then slowly lowered while the deck was gently swung into its final position, lining up the deck marks with the centerline of the I-beam webs. This ensured an accurate simple span of 5'-2" between centerline of supports. The straps were then removed and the testing machine cross-head was lifted out of the way. At mid-span, on top of the deck, another 2" wide by ½" thick elastomeric bearing pad was accurately placed across the 2 ft deck width, with the loading I-beam placed on top of this (2-sided tape was used between the loading I-beam and bearing pad). A final ½" thick elastomeric bearing pad was placed between the loading head and the top steel I- girder.

Bearing pads provide an even and continuous reaction between structural elements that would otherwise have only a few contact points, such as steel against concrete. Furthermore, the bearing pads allow rotations at the supports, capturing the simple bending assumption. As the deck displacements increase the end rotations also increase, resulting in an uneven distribution of compressive stresses on the bearing pads. This causes slight shifting of the reaction centroid toward the span centerline, effectively reducing the span length by a small amount. By keeping the bearing pad width to 2 inches, the maximum centroid shift was calculated to be about 0.5 inches, having minimal effect on the results. Lateral stability of the slabs while testing was satisfied by the applied normal force (and weight of the deck) multiplied by the relatively high coefficient of friction of the bearing pads.

After painting the test specimens white, vertical red lines were drawn on both sides of the slabs at 6-inch spacing for the full length of the composite test units. This was to help determine if, during the tests, any longitudinal shear slip developed between the precast and CIP slabs. By simple observation any relative slip would appear as a definite horizontal offset of the vertical red line at the junction between slabs. Throughout the tests the vertical red lines were inspected for any signs of distress, slip or longitudinal cracking between slabs. As discussed in more detail later, composite response was also verified by comparing predicted force-deformation behavior (which assumes fully composite action with perfect bond and no slip between slabs) and predicted displaced shapes, at various target levels, to measured results. If relative slip begins to develop between slabs, some of the composite action is lost and the force-deformation curve will start to turn down and away from the predicted behavior and approach the expected response from the 2 slabs acting independently of each other (non-composite action).

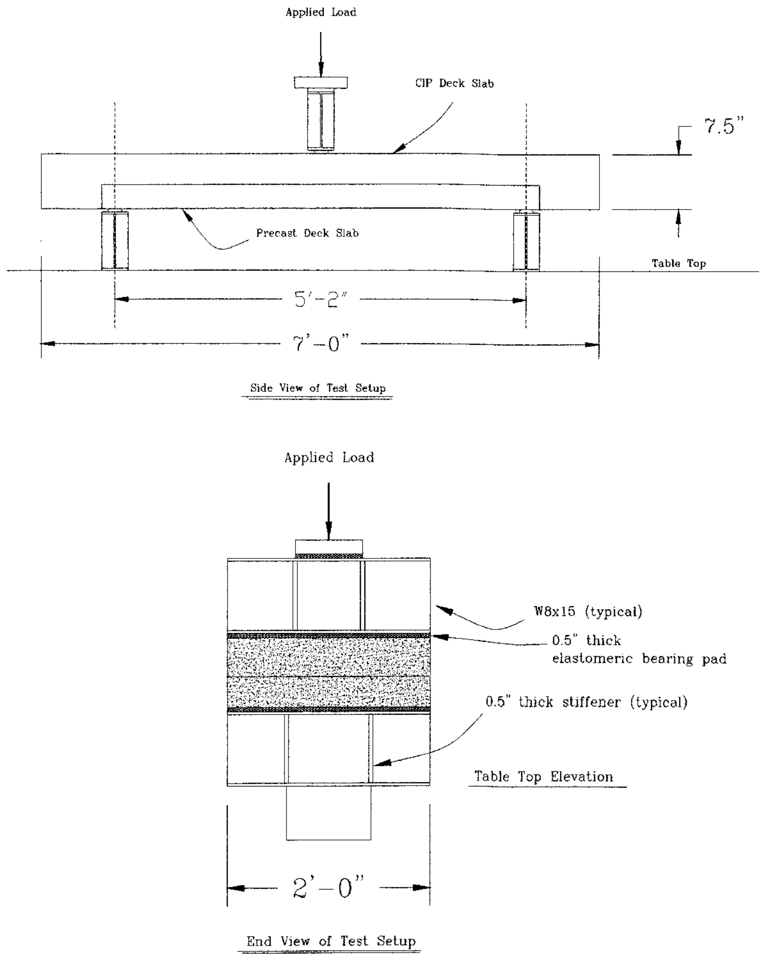

Figure 2-1. Overall setup for bridge decks tested in simple bending

3.0 PREDICTION ANALYSES

Prior to each of the 5 deck tests, the overall force-deformation behavior was predicted using detailed nonlinear analyses in SAP2000, Version 9 [2]. Analyses were conducted in displacement control by pulling down on the center node of the computer model, allowing good control of the analysis and allowing the post-peak force results to be obtained, which would not be possible if the analysis were run in force control (note that the structural tests were also run in displacement control and for the same reasons). Moment-curvature analyses were conducted for the critical section of each complete slab test unit and used to develop moment-rotation nonlinear elements for the SAP analysis. Often cracked properties are assumed for lateral seismic column analysis because bridge columns typically crack at a small lateral load. However, precast deck slabs that are prestressed crack at a very high load relative to their yield and ultimate capacities and, therefore, the pre-cracked state should be included in the prediction analysis. Furthermore, also because of the prestressing, only about half the deck length cracks at ultimate displacement and, therefore, gross section properties are more appropriate than cracked properties for the remaining un-cracked regions of the deck toward the supports. The deck analysis model had 20 beam elements between support-centerlines with nodes spaced at 3.1 inches. A single elastic beam element modeled the cantilevers that extend beyond the supports.

Nonlinear moment-rotation elements were provided across each of the nodes between supports and the end moment of one of the two connecting elastic beam members was released. This release forces the nonlinear moment-rotation element to resist any moment that develops at the joint. By having many closely spaced nonlinear elements, the spread and distribution of cracking (with increased applied load and displacement) along the length of the structure can be captured. An initial single crack forms at the maximum moment location directly below the load point and spreads to a series of well-distributed cracks at a given spacing over about half the deck length at ultimate. The cracks develop across the width of the slab due to the full-width loading and reaction steel I-beams used in the test setup. At the critical section of the slab, where the moment is a maximum (mid-span under the applied load), a second moment-rotation nonlinear element was used to model the formation of a plastic hinge. The moment-rotation behavior was derived from the same moment-curvature analysis used for the other nonlinear moment-rotation element, but with curvatures multiplied by a different length equal to the plastic hinge length of the critical section.

Rather than having many closely spaced nonlinear hinge elements to capture the spread of plasticity, a single plastic hinge element is used because it allows strain penetration on both sides of the critical section (where a wide-open flexural crack will form) to be directly included – in deriving the plastic hinge length. Thus plastic behavior is modeled with a single nonlinear moment-rotation element at the maximum moment location (mid-span) while the distribution of cracking is captured with a series of nonlinear moment-rotation elements that have a bilinear stiffness representing gross and cracked section properties.

Rotation of a beam between two points is defined by the integral of the curvatures between points. As the points move closer together, curvatures approach being constant over this reduced length and the beam rotation can be approximated as the curvature multiplied by the short member length. Thus with the relatively short 3.1-inch elastic beam elements in the analysis model, moment-rotation results can be determined directly from the moment-curvature results by simply multiplying curvatures at each analysis step by the length of 3.1 inches, with no adjustment required for the moment. It is important to be careful, however, not to allow the initial rotational stiffness of the nonlinear moment-rotation elements to increase the elastic flexibility of the structure, which is already represented by the elastic beam elements. This is ensured by (1) increasing the initial stiffness of the moment-rotation elements so that essentially no rotation occurs prior to cracking or by (2) increasing the stiffness of the elastic beam members so that they act as rigid links between nodes, and all initial elastic response is associated with the initial stiffness of the nonlinear rotation elements. Due to numerical challenges in the nonlinear analysis, the best solution was to use the rigid link approach and allow initial elastic rotations to develop at all hinges.

For the plastic hinge element at the maximum moment location at mid-span (under the applied load) a plastic hinge length was found based on an assumed strain-penetration length of the strand extending from both sides of the critical section. The code development length of 50 strand diameters was used to calculate the equivalent plastic hinge length (length over which critical section curvatures are taken as constant for analysis purposes). If a linear variation of strain is assumed over the development length given above, then the equivalent plastic hinge length is 25 strand diameters for each direction from the critical section. Hence the total plastic hinge length is taken as 50 strand diameters, or 50*3/8 = 18.75 inches. Based on the measured crack width of 0.59" at a mid-span displacement of 2.5" for the 1st test, strain-penetration of the prestressing steel is found to be about 17.6", in good agreement with the value of 18.75" given above that was used in all prediction analyses.

The source-code of the author's moment-curvature program ANDRIANNA [3] was modified to allow section analyses of bridge decks composed of precast and CIP slabs. It now recognizes that the precast slab is prestressed while the CIP topping slab has no prestressing and, thus, no initial compressive stresses and strains. Therefore in the moment-curvature analysis the linear strain assumption across the section depth is still valid, but with a discontinuity at the interface between slabs equal to the initial strain in the precast slab. At zero curvature there are no strains and stresses in the CIP slab, while the compressive strains in the concrete and rebar are the same across the prestressed slab section (strains are the same but not stresses). At the start of the analysis the prestressing strand has a large initial strain and stress associated with the stressing operation before the concrete was cast. ANDRIANNA was originally written for detailed seismic analysis of bridge columns and has been used extensively for prediction analyses of large-scale and full-scale column tests at universities such as UCSD, as well as for the seismic design and/or analysis of bridge columns and piles on major bridge projects such as the New East Spans of the San Francisco-Oakland Bay Bridge, San Diego-Coronado Bay Bridge and the new Tacoma Narrows Bridge. ANDRIANNA was also used for the seismic assessment of all bridge columns on the many miles of elevated portions of the BART System in the Bay Area.

Before comparing predicted and measured relative displacement profiles, a typical example of measured absolute displacement profiles is given in Figure 3-1 (Test 2 results are shown, other test results are similar). This clearly shows vertical displacements of the slab at the supports due to deformation of the elastomeric bearing pads (Figure 3-1). To compare against prediction analyses, relative measured displacements were determined by subtracting displacement components associated with measured deformations at the supports from the absolute measured deck displacements. Predicted and measured relative displacement profiles are given in Figures 3-2 through 3-5 at specific force and displacement target levels for Tests 2 through 5. Test 1 (precast deck panel) is not included here because displacements were not measured at the ¼ – span locations. The 1st target level in each of the figures represents linear behavior, with the 2nd target including some cracking. The remaining 3 targets represent increasing cracking and development of plastic hinging at the critical mid-span section. These excellent comparisons demonstrate that the prediction analyses accurately reflect the deformations along the length of the deck in the elastic stage, through flexural cracking and into large plastic deformations. It is clear that at large displacement targets, most of the deformations are coming from concentrated rotations that develop at mid-span. Force-deformation prediction analyses are compared against the measured results for all 5 test slabs in the following section.

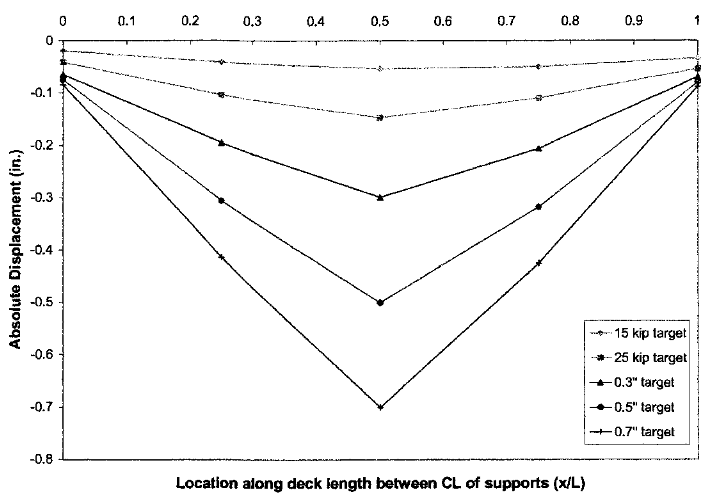

Figure 3-1. Measured absolute displacement profiles at various targets (Test 2)

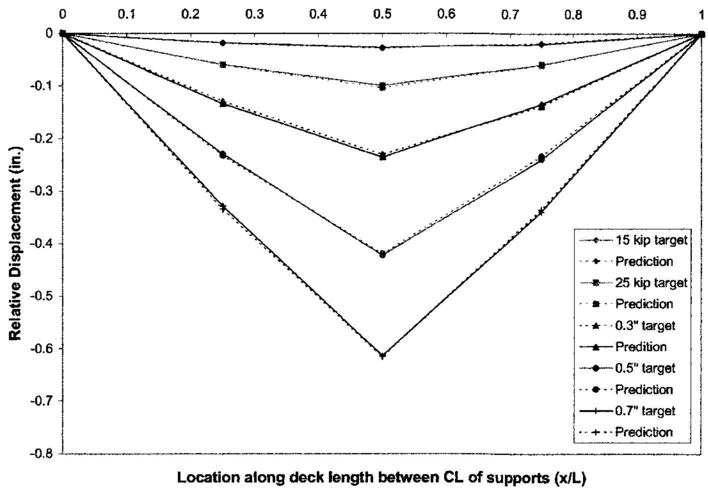

Figure 3-2. Predicted and measured displacement profiles at various targets (Test 2)

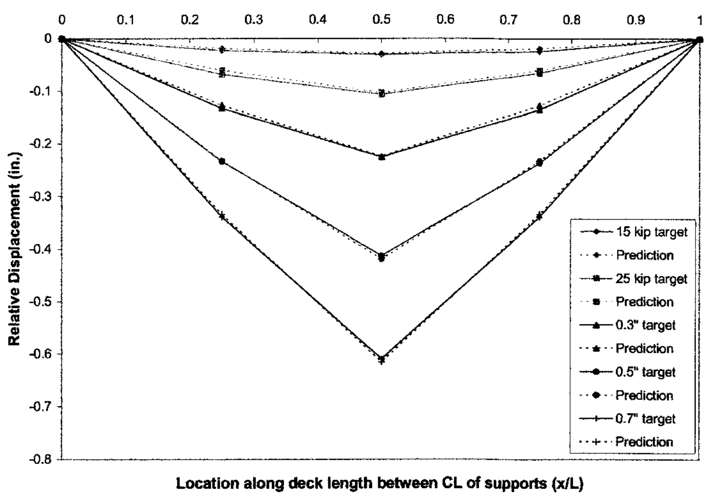

Figure 3-3. Predicted and measured displacement profiles at various targets (Test 3)

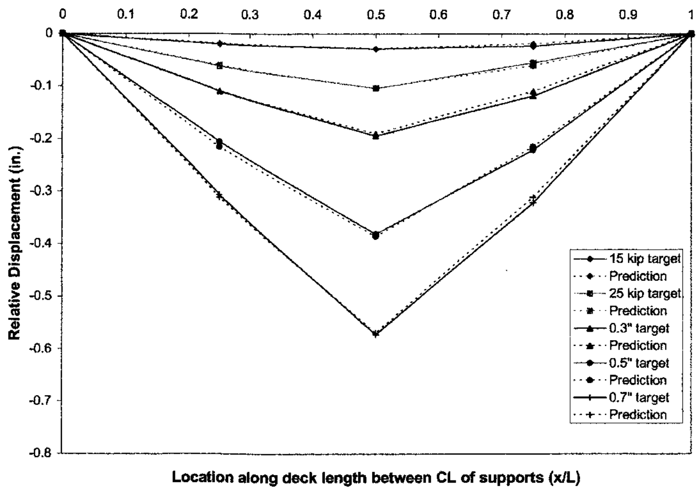

Figure 3-4. Predicted and measured displacement profiles at various targets (Test 4)

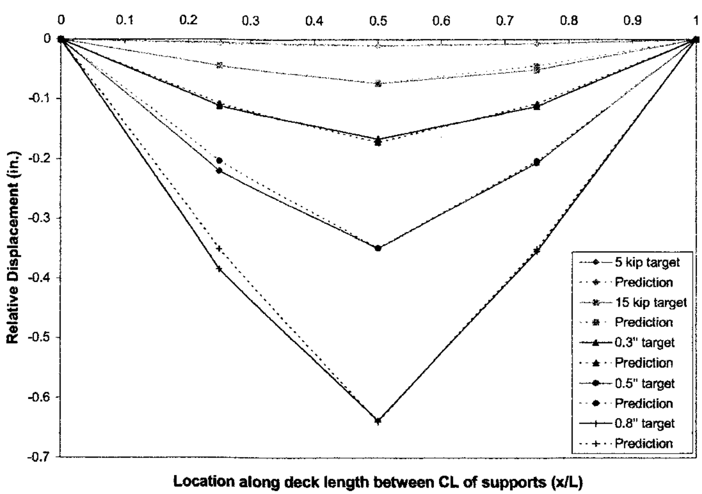

Figure 3-5. Predicted and measured displacement profiles at various targets (Test 5)

4.0 TEST RESULTS

The structural tests were performed monotonically in displacement control to failure. It is considered displacement control because the structure is loaded by slowly moving the cross-head of the testing machine down and measuring the force that develops. The cross-head moves or displaces up and down by rotating a loading button on the control panel that turns 4 large spirally threaded shafts on either side of the cross-head (see Figure 2-3). Force and displacement targets were used to decide when to pause the loading and take force and deformation measurements. Because the deck structures are initially stiff, force levels were used as targets until cracking had developed and then, as the structure softened, the targets were switched to various displacement levels. Using displacement targets at increased force and displacement levels has the advantage that the target can be reached, regardless if the force starts to degrade. Note that targets are used only to get the structure close to the level of interest.

Once the force or displacement target level is approximately achieved then the measured force and displacements are accurately recorded. It is also important to note that the chosen displacement target measurement was the absolute displacement at mid-span from a digital readout displacement gage. As the cross-head was moving, the digital displacement gage was watched and the cross-head was paused when the gage approximately reached the target. Then the displacements and force were read accurately and recorded. In the following, results are discussed from each of the 5 tests and sometimes the term relative displacement is used and at other times the terms target displacement or absolute displacement are used. Relative displacement indicates that the bearing pad deformations have been subtracted from the absolute displacements, while target displacement is the absolute displacement that was approximately achieved at mid-span, but is not the final measured value.

Displacements were measured at mid-span, the bearing pads and (for the 4 composite slab units) at the ¼-span locations. As the ends of the deck panels were supported on elastomeric bearing pads it was important to measure the amount they compressed so that this could be removed from the measured absolute structure displacements, resulting in measured deformations of the structure that can be directly compared to the predicted response that assumed unyielding supports. For the 4 tests with a CIP topping, displacements were measured at mid-span, the ¼ points and at the bearing supports so that the displaced shapes could be plotted at any displacement level (measured versus predicted displacement profiles are given in Section 3). Results and observations for each of the 5 tests are given in the following.

Test 1 (Precast Deck Panel without CIP topping)
Results from the 1st test show that the behavior and deformation capacity of the precast slab are somewhat greater than expected (Figure 4-1). Note that two analysis results are shown in Figure 4-1 because in the original prediction the concrete strength was taken as 4 ksi and the prestress strand ultimate strength was assumed to be 270 ksi. However, measured material strengths of approximately 5 ksi for CIP concrete, 7 ksi for precast concrete and 328 ksi for prestress strand were much higher than assumed, requiring the 2nd analysis shown in the figure (which resulted in much closer results). Note that modification to the material strengths was the only change to

the prediction model. Cracking began at the target force of 4.275 kips, with a full-width crack developing on the bottom of the slab at the maximum moment location (Figure 4-5). Additional cracks formed on either side of the center crack at the 0.3" target displacement (Figure 4-6). At the target displacement of 0.6", well distributed cracking was observed with an average spacing of 4.5" (Figure 4-7). Concrete spalling began at the structure centerline on the top surface at the target displacement of 1.8". Large flexural cracks opened up at 1.5" of vertical displacement (Figure 4-8) and became much wider at 2.5" of vertical displacement (Figure 4-9). The crack width at the center of the slab was measured to be about 0.59" at 2.5" vertical displacement (Figure 4-9). From these measurements it was possible to back-calculate the strain-penetration (extending in both directions from the center crack) to be about 17.6", which is in good agreement with the assumed value of 18.75" (50 strand diameters) used in the predictions for all of the tests.

By 3" of vertical displacement the center crack was so large that some vertical shear slip is evident (Figure 4-10). Following the test and removal of the loading beam and bearing pad, the top surface spalling at the maximum moment location is clear (Figure 4-11). The test was stopped at the very large target displacement of 3.5", following continued strength degradation with increasing displacement. No sudden failure was observed and the test was stopped when displacements and end rotations were close to the capacity of the test setup. Measured strength degradation began at the target displacement of 1.4" (relative displacement of 1.35") with a maximum force of 8.15 kips (Figure 4-1). Predicted maximum force and associated relative displacement were 7.02 kips and 1.2", respectively (Figure 4-1). Predicted displacement capacity was 1.84" at a compressive strain for the unconfined concrete of 0.005.

<u>Test 2 (Precast Deck Panel with Coarse Broom finish and CIP topping slab)</u>
Flexural cracking began at 25 kips (Figure 4-12). It is clear that at this level of loading only flexural cracks had developed (no shear cracks), as indicated by the vertical cracks extending from the extreme tension fiber through the thickness of the member (Figure 4-12). More extensive cracking developed at the target vertical displacement of 0.5" (with an average crack spacing of 5") and the tops of the cracks began to rotate from vertical as they extended higher (Figure 4-13), indicating the influence of shear. Inclined cracking became much more extensive at the target displacement of 0.6", with significant flexure-shear cracks forming (Figure 4-14). The flexure-shear cracks began at the extreme tension fiber (bottom of the slab) in bending. As the flexural cracks widened and extended vertically up through the section depth (with increased structure displacement) the shear influence turned them into shear cracks at an angle. It is of interest that shear cracks formed only as an extension of deep flexure cracks, even though the shear force is approximately constant between applied load and reaction points. Concrete spalling started at 0.6" target displacement (Figure 4-14). More extensive flexure-shear cracks developed at 0.8 inches of displacement, resulting in significant loss in force (Figure 4-15). Wider flexure-shear cracks were observed at 0.9" target displacement (Figure 4-17). Sudden shear failure occurred at the 1.0" target displacement, ending the test (Figure 4-18).

The failure developed across the full width of the section along a flexure-shear crack line, and resulted in a large visible offset between the two sides of the structure (Figure 4-18). Prediction analyses and real-time measured test data were projected on the laboratory wall so that visitors could watch the comparisons as they developed (Figure 4-16). Note that no horizontal shear slip was observed or measured between precast and CIP slabs, indicating that fully composite behavior was maintained throughout the tests. This is demonstrated by the close correlation between measured and predicted force-deformation responses as well as by the various Test 2 Photo Journal pictures that demonstrate the vertical red lines spaced at 6" remained straight across slab boundaries, with no longitudinal offset at the slab interface. The maximum measured force was 36.5 kips at 0.71" relative displacement versus the predicted maximum force of 35.0 kips at a relative displacement of 0.86". Predicted relative displacement capacity was 1.15" at a compressive strain of 0.005 compared to the measured relative displacement at failure of 0.946". Measured and predicted force-deformation results are given in Figure 4-2.

<u>Test 3 (Precast Deck Panel with Medium Broom finish and CIP topping slab)</u>
A small partial-width flexural crack developed at the 15 kip target level (Figure 4-19) with a full-width crack at 20 kips. More extensive cracking occurred at 25 kips with 5" average crack spacing (Figure 4-20). Flexure-shear cracks started to develop at the 0.3" target displacement and increased at the 0.4" target (Figure 4-21). Spalling initiated at the 0.5" target and became more severe at the 0.7" target displacement (Figure 4-22). By the 0.9" target displacement, fully extended flexure-shear cracks had developed on either side of the load point (Figure 4-23). At the 1" target level the flexure-shear cracks were visibly widening and extending toward each other (Figure 4-24). Flexure-shear cracks widened more at target displacements of 1.2" and 1.3" and formed a long arc from either side of the structure centerline, extending toward the maximum moment location under the applied load (Figures 4-25 and 4-26). Spalling at the top of the slab at the centerline become more extensive, with an arc forming in the opposite direction to the flexure-shear cracking (Figures 4-25 and 4-26).

The future failure plane is clear from the extended flexure-shear crack at a target displacement of 1.7" (Figure 4-27). Shear failure occurred at approximately 2.5" target displacement (Figure 4-28) - based on scaling measured ¼-span displacements (centerline displacement gages were lost due to extensive spalling and cracking). The measured crack width where failure occurred opened up from a stable 8mm just before failure to 90 mm following failure. Maximum measured force was 36.2 kips at a relative displacement of 0.9" compared to the predicted maximum force of 35.0 kips at a relative displacement of 0.86". Measured and predicted force-deformation plots are given in Figure 4-2. Predicted relative displacement capacity was 1.15" at a compressive strain of 0.005 compared to the measured relative displacement at shear failure of about 2.3". There was no indication of longitudinal shear slip between the precast and CIP slabs, with flexure and flexure-shear cracks forming across boundaries as if they were one slab.

<u>Test 4 (Precast Deck Panel with Carpet Drag finish and CIP topping slab)</u>
Flexural cracking began at the 25 kip target load (Figure 4-29). At the 0.3" target displacement the flexure cracks started to turn into flexure-shear cracks and continued to extend and turn at the 0.4" target level (Figure 4-30). Onset of crushing started at the 0.7" target displacement. A loss in force occurred at the 1" target (0.865" relative displacement) level (Figure 4-31), with flexure-shear cracks that developed on either side of the applied load joining arcs at the slab centerline, just below the crushing and spalling of the concrete. Flexure-shear cracks widened noticeably, as did the extent of spalling, at the 1.2" and 1.4" target displacement levels (Figures 4-32 and 4-33).

Figure 4-34 shows the deck just before shear failure at a displacement target of 2.8". Shear failure occurred at 3 inches of relative displacement (Figure 4-35), measured with a taught string line between centerline of supports following shear failure (before removing the load). String measurement was required because the displacement gage targets came off due to extensive spalling and cracking in the critical section region. The last measured target was 2.8" (2.65" relative displacement). Maximum measured force was 37.3 kips at a relative displacement of 0.77" compared to the predicted maximum force of 35.0 kips at a relative displacement of 0.86". Predicted relative displacement capacity was 1.15" at a compressive strain of 0.005 compared to the measured relative displacement at failure of 3". There was no indication of longitudinal shear slip between the precast and CIP slabs, with flexure and flexure-shear cracks forming across boundaries as if they were one slab.

<u>Test 5 (Precast Deck Panel with Coarse Broom finish and CIP topping slab- Negative Bending)</u>
Flexural cracking began at 10 kips (Figure 4-36) with more full-width cracks occurring at 15 kips (Figure 4-37). By the target displacement of 0.3", previously formed flexure cracks started turning in as they extended up, showing the influence of shear (Figure 4-38). By the target displacement of 0.8" the flexure-shear cracks had taken form, but were not yet very wide (Figure 4-39). Note how in Figure 4-39 the flexure-shear cracks cross the marked vertical red lines without any slip between slabs. At target displacement of 1" the flexure-shear cracks began to widen significantly (Figure 4-40), resulting in a sudden shear failure at target 1.2" (Figure 4-41). Note that the final failure plane follows a straight shear line that is an extension of the flexure-shear crack. Maximum measured force is 36.9 kips at a displacement of 0.83" compared to the maximum predicted force of 30.5 kips at a displacement of 0.85". Predicted relative displacement capacity was 1.04" at a compressive strain of 0.005 compared to the measured relative displacement at shear failure of 1.06".

This test was conducted in negative bending (upside down), with the precast slab at the top and the CIP slab at the bottom. Additional reinforcement was used in the CIP slab (4 #5 bars). In the prediction analysis the added rebar was missed, resulting in predicted force levels that are about half of the measured values. This was realized while the test was in progress and a second analysis was conducted following the test that included the 4 #5 bars rather than the 2 # 5 bars used in the prediction analysis. Such a modification brought the force levels much closer to the measured results. At this point the stress-strain properties of the #5 bars have been assumed and closer correlation between measured and analysis results is expected once tensile tests are conducted for the #5 bars.

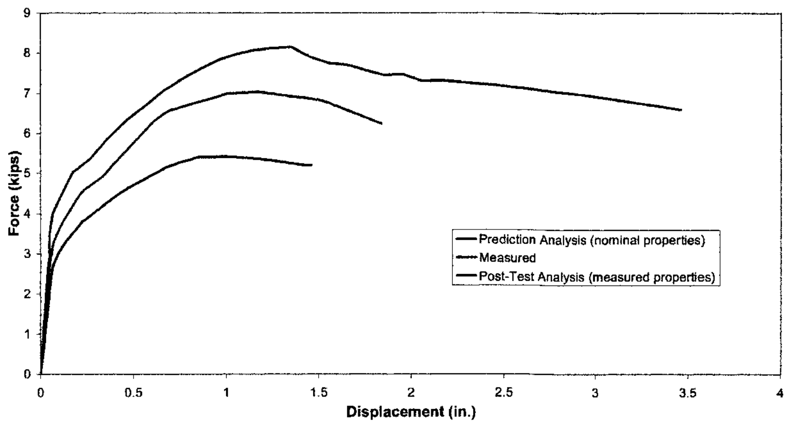

Figure 4-1. Precast deck panel, measured and predicted behavior (Test 1)

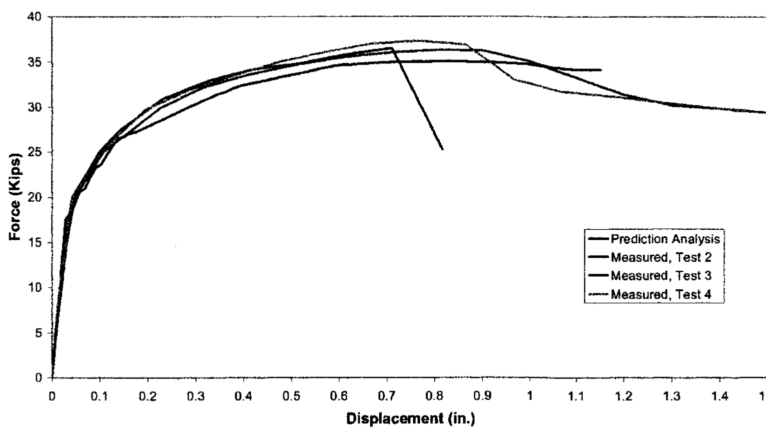

Figure 4-2. Precast with CIP slab, measured and predicted behaviors (Tests 2 through 4)

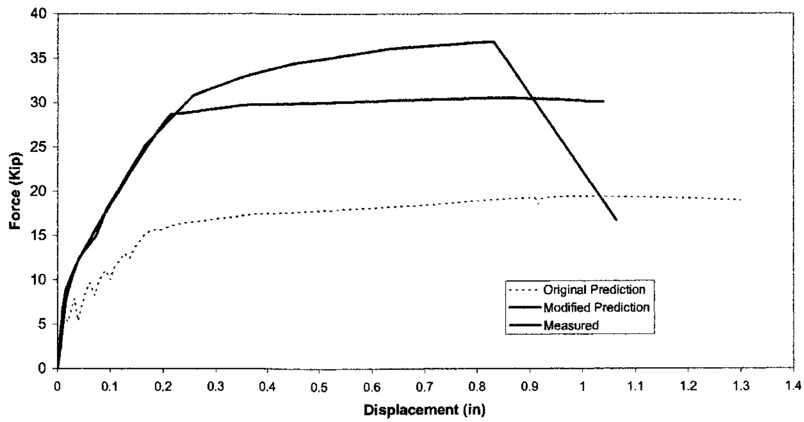

Figure 4-3. Precast with CIP slab (neg. bending), measured and predicted behavior (Test 5)

5.0 CONCLUSIONS AND RECCOMMENDATIONS

A series of full-scale structural tests were performed to failure in the Structural Engineering Laboratory at SDSU to characterize the response of precast bridge deck panels with CIP deck topping. One test was performed for the precast deck panel by itself (Test 1), with no cast-in-place topping, and the remaining 4 tests included the precast panel with various surface roughening techniques and cast-in-place topping slab (composite tests). One of the 4 composite structural tests (Test 5) was performed in negative bending (upside down) to characterize the response in this direction.

The purpose of the structural tests was to demonstrate that that the composite decks (precast and CIP slabs) would act as a single composite slab in flexure, with no longitudinal shear slip at the slab interface. As no reinforcement crosses the plane between slabs, various surface roughening techniques were applied to the top of the precast deck panel to help transfer longitudinal shear stresses that develop from composite action and the simple beam loading applied. These included Coarse Broom and Medium Broom brush finishes as well as a Carpet Drag finish.

All 5 test specimens exceeded their nominal and ultimate force capacities based on prediction analyses. No sign of longitudinal shear slip was observed in any of the 4 composite deck tests all the way to failure. Flexure cracks and flexure-shear cracks formed across slab boundaries as if the precast and CIP slabs were cast as one. All of the vertical red lines that were marked on both sides of the test units at 6-inch spacing remained straight across slab boundaries throughout the tests to failure, also verifying that no longitudinal slip developed between slabs. Following large flexural displacements and loss in force associated with wide-open flexure-shear cracks, all 4 composite decks failed in diagonal shear along an angled failure plane defined by one of the flexure-shear cracks that had developed earlier in the test. As the tests progressed, one of the flexure-shear cracks would start to widen more than the others, ultimately resulting in this being the failure location (crack widths were measured throughout the tests). Failures developed across the 2-ft width of the structure, indicating that the steel I-beams properly spread the applied load and reactions across the full deck width.

It is recommended that this precast and CIP bridge deck system be used for new bridge construction to save time and reduce construction costs. Any of the precast slab roughening techniques used in this testing program will work to prevent longitudinal shear slip at the slab interface, as demonstrated by the test results and by comparisons to predicted force-deformation and displacement profile results, which assumes fully composite behavior.

6.0 REFERENCES

[1] SR 22 HOV Widen Design Build Precast/Prestressed Concrete Composite Deck Panels, Test Procedures and Performance Criteria, Report for GMR, JV by PBS&J, July, 2005.

[2] SAP2000, V9, Reference Manuals, CSI, 2004.

[3] Dowell, R., K., ANDRIANNA User's Guide, Dowell Engineering, 2005