Structural Tests of Precast, Prestressed Concrete Deck Panels for California Freeway Bridges

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Full-scale structural tests of five precast, prestressed concrete bridge deck panels have been carried out in simple bending to failure at the San Diego State University Structural Engineering Laboratory in San Diego, Calif. The purpose of the tests was to demonstrate to Caltrans that the precast concrete panels act compositely in flexure with a cast-in-place topping slab so that they can be used for California bridges. To ensure that the full deck width was active in resisting the applied loads, steel I-beams the width of the deck were used under the load and at the reaction points. Neoprene bearing pads were incorporated at the supports to allow free rotations at the deck ends. Each bridge deck consisted of a precast, prestressed concrete deck panel with a cast-in-place topping slab, with no reinforcing bars crossing the interface between slabs. Of particular interest was the verification that no horizontal shear slip occurred between the two slabs and that the deck acted as a fully composite member to failure. One test featured the precast concrete slab by itself, three of the tests investigated the deck behavior in positive bending with different roughening levels applied to the top of the precast concrete slab, and the fifth test was performed in negative bending (upside down). Nonlinear prediction analyses were conducted for each of the tests, including the formation and spreading of flexural cracking along the deck and plastic hinging at the critical section in the center of the span. Test results, supported by detailed structural analyses, demonstrated that there was no horizontal shear slip between the precast concrete panels and cast-in-place slabs, allowing them to respond in flexure as a single composite slab.

I n order to reduce the cost and construction time of the SR-22 HOV Widen Design/Build Project, contractor Granite-Myers-Rados, Joint Venture (GMR-JV) and designer PBS&J are recommending the use of precast, prestressed concrete deck panels with cast-in-place (CIP) topping for about 30 bridges in California. Deck panels will span between precast, prestressed concrete girders, placed one after another on camber strips, followed by the CIP concrete slab placed across the whole bridge deck. (Note that the CIP topping slab acts as both a structural component and the final riding surface of the deck.) A similar application of precast concrete deck panels was used for widening the San Mateo-Hayward Bridge (**Fig. 1**).

To demonstrate the capability of the completed deck units (precast concrete deck panels with CIP topping and no reinforcing bars crossing the plane) to act compositely in flexure with no longitudinal shear slip between slabs, GMR-JV sponsored San Diego State University's (SDSU's) series of full-scale structural tests and prediction analyses of this deck system. The deck units consisted of precast concrete panels with various roughening techniques (coarse broom, medium broom, and carpet drag) and a CIP topping slab. Added details of the testing program can be found in references 1 and 2.

The test units are considered full-scale because the full depth and full length (distance between the faces of girders) of the prototype slabs are included. Rather than the full width of the slab, a representative 2 ft (0.6 m) width (along the girder line) is used. This reduced width of the slab along the girder line has no effect on its overall response and failure, with the same amount of prestressing steel and mild steel reinforcement per foot width as the prototype.[1] For any given displacement level, the force per unit width of the prototype and test unit is the same.

Structural tests of five precast concrete deck panels with CIP toppings have been conducted at the SDSU Structural Engineering Laboratory to verify that horizontal shear transfer between the 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 case), the tests were conducted in simple bending, with the load applied at midspan and reactions provided at either end of the precast concrete deck panel.

The tests investigated different finishes at the tops of the precast concrete panels, with the first benchmark test having no CIP topping. One of the tests was conducted in negative bending (upside down) with a coarse broom finish applied to the precast concrete slabs. Loading and reaction I-beams were used to ensure that the full 2 ft (0.6 m) slab width was effective throughout the test to failure.

Including the precast concrete and CIP slabs, the total size of each test specimen was 7 ft (2.1 m) long, 2 ft (0.6 m) wide, and 7.5 in. (190 mm) thick. The precast concrete slab dimensions were 5 ft 4 in. (1.6 m) long, 2 ft (0.6 m) wide, and 3.25 in. (83 mm) thick. The precast concrete deck panels were cast by Pomeroy Corp. at its Petaluma, Calif., plant and were shipped to the SR-22 HOV bridge site in Orange County, Calif., so that the CIP toppings could be added by contractor GMR-JV.[2]

Pomeroy is a plant-certified PCI producer member and is

Fig. 1. Precast concrete slabs are being placed on precast, prestressed concrete girders for widening the San Mateo–Hayward Bridge.

one of the major producers of precast, prestressed concrete components in California and the United States.

After curing, the completed deck units were shipped to the SDSU Structural Engineering Laboratory for testing. Included with this shipment were prestressing strand and reinforcing bar samples as well as five concrete cylinders taken from the CIP slab placement. Pomeroy took cylinder samples from the precast concrete casting and tested them on the days the decks were tested.

TEST SETUP

The tests were conducted in simple bending with a point load applied at midspan and reactions at the ends of the precast concrete slab (**Fig. 2**, **3**). Because the loading head diameter and reaction table width were less than the 2 ft (0.6 m) slab width, relatively rigid steel I-beams were designed to spread the applied load and reactions evenly across the section width. For both precast concrete and CIP slab details, see **Fig. 4**. Vertical stiffeners were added to the I-beams at critical load points (under the loading head and at the sides of the reaction table [**Fig. 2**, **3**, **5**]).

At the reaction locations, the 2-ft-long (0.6 m) steel I-beams were bolted directly to the 10-in.-wide (250 mm) loading table. Along the top of the 2-ft-long (0.6 m)
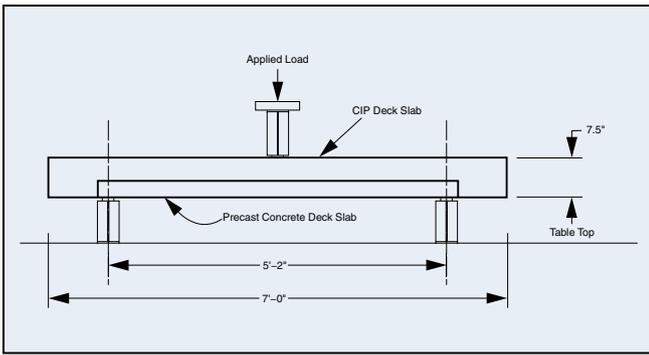
Fig. 2. Side view of overall setup for bridge decks tested in simple bending. Note: 1 in. = 25.4 mm.
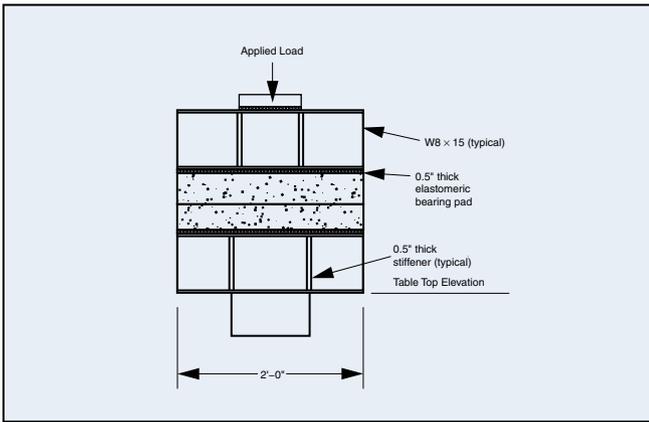
Fig. 3. End view of overall setup for bridge decks tested in simple bending. Note: 1 in. = 25.4 mm.

I-beams, 2-in.-wide (50 mm) × 0.5-in.-thick (13 mm) elastomeric bearing pads were secured with double-sided tape. The completed deck slabs were painted white (so crack marking with felt pens during the tests would be visible) and accurately marked for positioning within the test setup.

A given test unit was lifted with a crane over the upper-floor lab rail and lowered into the testing pit and onto oversized rollers that were temporarily placed on the reaction table (**Fig. 6**). Straps were removed and the deck slab was rolled into approximate position by hand. The straps were
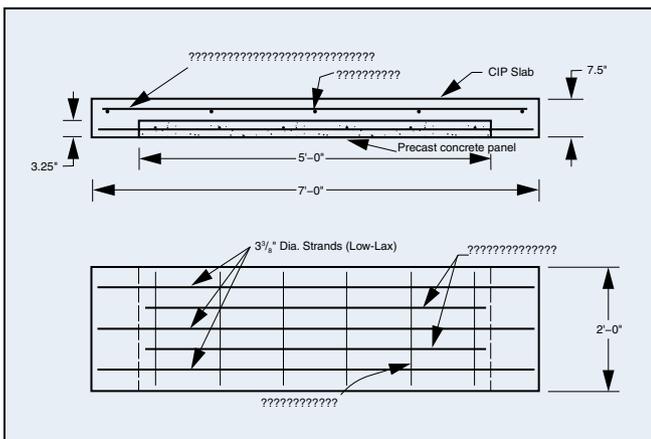
Fig. 4. Precast concrete panel and cast-in-place slab details. Note: 1 in. = 25.4 mm.

then wrapped around the test specimen and crosshead of the testing machine, allowing the deck slab to be lifted by the machine so that the rollers could be removed.

The crossbeam was slowly lowered while the deck was gently swung into its final position with the deck marks lined up with the centerline of the I-beam webs. This ensured an accurate simple span of 5 ft 2 in. (1.6 m) between centerline of supports. The straps were then removed and the testing machine crosshead was lifted out of the way.

At midspan on top of the deck, another 2-in.-wide (50 mm) × 0.5-in.-thick (13 mm) elastomeric bearing pad was accurately placed across the 2 ft (0.6 m) deck width with the loading I-beam placed on top (two-sided tape was used between the loading I-beam and bearing pad). A final 0.5-in.-thick (13 mm) 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 and high stress concentration of 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. For the thicker composite slabs, this effect was eliminated with an overall increase in member length due to plastic hinging at midspan and flexural cracking along half the span.

By keeping the bearing pad width to 2 in. (50 mm), the maximum centroid shift was calculated to be about 0.5 in. (13 mm), 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 against concrete and steel.

After the test specimens were painted white, vertical red lines were drawn on both sides of the slabs at 6 in. (150 mm) spacing for the full length of the composite test units (Fig. 6). This was done to help determine whether any horizontal shear slip developed between the precast concrete and CIP slabs during testing. By simple observation, any relative slip would appear as a definite horizontal offset of the vertical red line at the junction between the slabs. Throughout the tests, the vertical red lines were inspected for any signs of distress, slip, or horizontal cracking between the slabs.

Discussed in more detail later, composite response was also verified by comparing measured and predicted force-deformation behaviors, as well as measured and predicted displaced shapes at various force and displacement target levels. (Note that the prediction assumes fully composite action with perfect bond and no slip between slabs.) If relative slip starts to develop between the slabs, some of the composite action is lost and the force-deformation curve will begin to turn down and away from the predicted behavior, approaching the expected response from the two slabs acting independently of each other (non-composite action with much smaller force capacity).

Fig. 5. Isometric and end views of overall test setup with the Test 1 precast concrete panel.

PREDICTION ANALYSES

Prior to each of the five deck tests, the overall force-deformation behavior was predicted using detailed nonlinear analyses in SAP2000, Version 9.[3] Analyses were conducted in displacement control by pulling down on the center node of the computer model, allowing for good control of the analysis and allowing the post-peak force results to be obtained, which would not be possible if the analyses were run in force control. (Note that the structural tests were also run in displacement control for the same reasons that the analyses were run.) Moment-curvature analyses were conducted for the critical midspan section for each complete slab test unit and were used to develop moment-rotation nonlinear elements for the nonlinear SAP analysis.

Unlike bridge columns, which typically crack at a small lateral load (allowing cracked column properties to be used for seismic bridge analysis), precast concrete deck slabs that are prestressed crack at a very high load relative to their yield and ultimate capacities. Therefore, the precracked state should be included in the prediction analysis.

Another effect of prestressing is that only about half of the deck length cracks at ultimate displacement and, thus, gross section properties are more appropriate than cracked properties for the remaining uncracked regions of the deck toward the supports (for the duration of the test to failure). The deck analysis model had 20 beam elements between support centerlines with nodes spaced at 3.1 in. (79 mm). A single elastic beam element modeled each of the two short 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 two connecting elastic beam members was released. This moment release forces the nonlinear moment-rotation element to resist any moment that develops at the joint. By having many closely spaced nonlinear elements, the formation and distribution of cracking (with increased applied load and displacement) along the length of the structure can be captured. An initial, single, full-width 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 of 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 (midspan under the applied load), an alternate 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

Fig. 6. Lowering a precast concrete panel onto rollers for positioning.
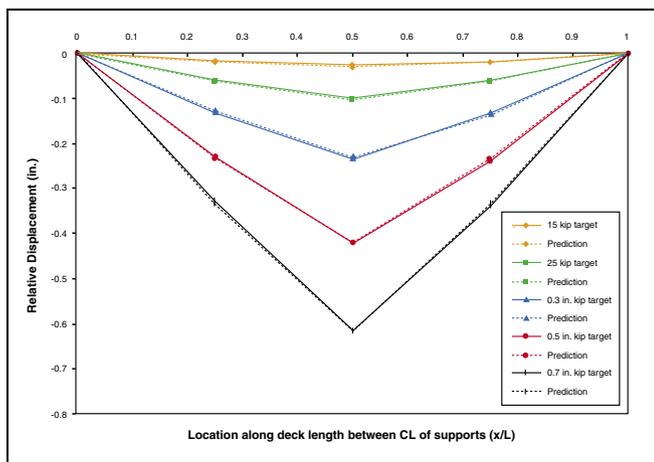
Fig. 7. Predicted and measured displacement profiles at various targets, Test 2. Note: 1 in. = 25.4 mm; 1 kip = 454 kg.
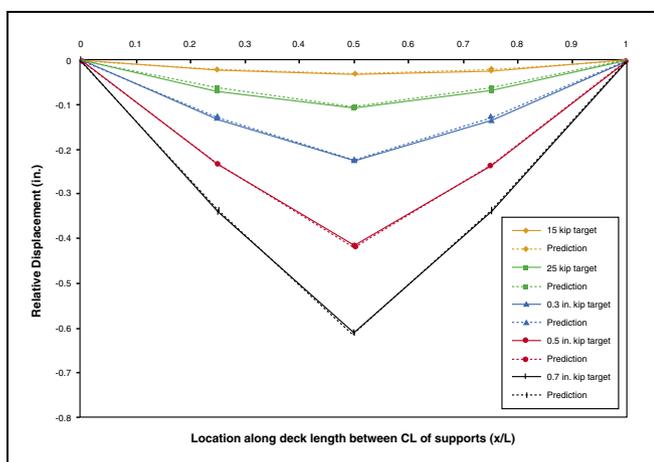
Fig. 8. Predicted and measured displacement profiles at various targets, Test 3. Note: 1 in. = 25.4 mm; 1 kip = 454 kg.

to develop the nonlinear cracking moment-rotation element. Curvatures were multiplied by the equivalent plastic hinge length of the critical section, however, rather than the distance between nodes.

Instead of having many closely spaced nonlinear hinge elements to capture the spread of plasticity from the critical region at midspan, 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 forms) 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 (midspan) while the formation and 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 curvatures between the points. As the points move closer together, the curvatures approach being constant over this reduced length, and the beam rotation can be approximated as a constant curvature multiplied by the short member length. Thus, with the relatively short 3.1 in. (79 mm) elastic beam elements in the analysis model, mo-

ment-rotation behavior for the cracking elements (all but the midspan) can be determined directly from the moment-curvature results. This is done by simply multiplying curvatures at each analysis step by the length between nodes of 3.1 in. (79 mm), with no adjustment required for the moment.

It is important to be careful 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 behavior is ensured by: (1) increasing the initial stiffness of the moment-rotation elements so that essentially no rotation occurs prior to cracking, or (2) by increasing the stiffness of the elastic beam members so 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 the simplicity of modifying the stiffness of the members and because of numerical challenges in the nonlinear analysis, the best solution proved to be the rigid link approach with initial elastic rotations developing at the nonlinear elements.

For the midspan plastic hinge element (at the maximum moment location under the applied load), the plastic hinge length was found based on an assumed strain-penetration distance 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 the critical section curvatures are taken as constant to determine plastic rotations for analysis purposes).

If a linear variation of strain is assumed over the development length given previously, 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 50×0.375 in. = 18.75 in. (476 mm).

Based on the measured critical section crack width of 0.59 in. (15 mm) at a midspan displacement of 2.5 in. (64 mm) for the first test, strain penetration of the prestressing steel is found from geometry and assumed linear variation in strain to be 17.6 in. (447 mm), which is in agreement with the previously given value of 18.75 in. (476 mm) that was used in all prediction analyses.

The source code of the first author's moment-curvature program, ANDRIANNA,[4] was modified to allow section analysis of a bridge deck composed of a precast concrete panel and a CIP topping slab. Originally written for a detailed seismic analysis of bridge columns, ANDRIANNA has been used extensively for prediction analyses of large- 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, seismic retrofit of the San Diego–Coronado Bay Bridge, and the new Tacoma Narrows Bridge. It was also used for the seismic assessment of all bridge columns on the many miles of elevated portions of the BART System in the San Francisco Bay area.

Concrete with two different sets of mechanical properties (precast and CIP) can now be modeled in ANDRIANNA. The program recognizes that the precast concrete slab is pre-
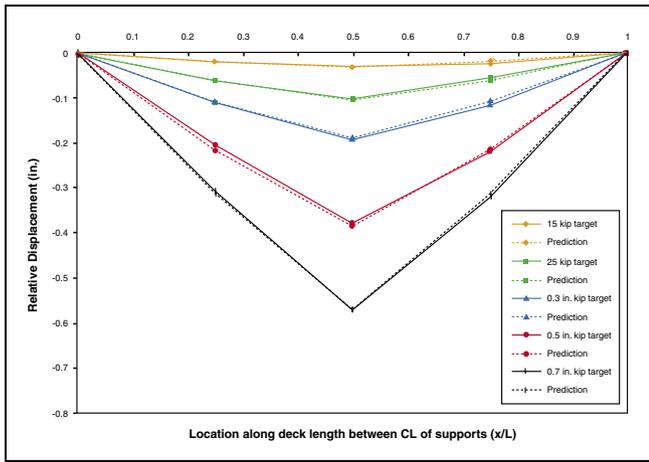
Fig. 9. Predicted and measured displacement profiles at various targets, Test 4. Note: 1 in. = 25.4 mm; 1 kip = 454 kg.
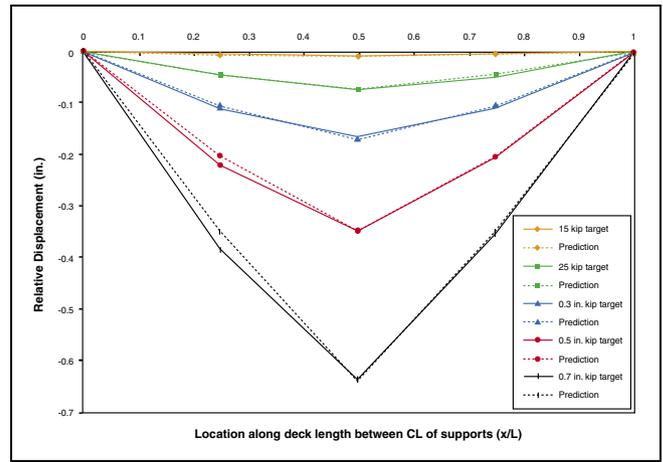
Fig. 10. Predicted and measured displacement profiles at various targets, Test 5. Note: 1 in. = 25.4 mm; 1 kip = 454 kg.

stressed, while the CIP topping slab has no prestressing and, thus, no initial compressive stresses and strains. Therefore, the linear strain assumption through the section depth is still valid in the moment-curvature analysis but with a discontinuity at the interface between slabs that is equal to the initial compressive strain in the precast concrete slab from prestressing.

At zero curvature, there are no strains and stresses in the CIP slab, while the precast concrete slab compressive strains in the concrete and reinforcing bars are the same across the prestressed slab section (strains in the reinforcing bar and concrete are the same but the stresses are different due to different modulus of elasticity values). At the start of the analysis, the prestressing strand had a large initial strain and stress associated with the stressing operation before the concrete was cast (minus effects from the initial elastic shortening).

To compare the results against prediction analyses, the relative measured displacements of each deck were determined by subtracting displacement components associated with measured deformations at the supports (elastomeric bearing pads) from the absolute measured deck displacements.

Predicted and measured relative displacement profiles are displayed in **Fig. 7**, **8**, **9**, and **10** at specific force and displacement target levels for Tests 2 through 5. Test 1 (precast concrete deck panel only, Fig. 5) is not included here because displacements were not measured at the one-quarter span locations. The first target level in each of the figures represents linear behavior, with the second target including some cracking.

The remaining three targets represent increasing cracking and full development of plastic hinging at the critical midspan section. These excellent comparisons demonstrate that the prediction analyses accurately reflect the deformations along the length of the deck in the elastic range, through extensive flexural cracking, and into large plastic deformations.

It is clear from the predicted and measured results that at large displacements, most of the deformations are due to concentrated rotations that develop at midspan. Force-deformation prediction analyses are compared to measured results for all five test slabs in the following section.

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 crosshead of the testing machine down and measuring the force that develops. The crosshead moves or displaces up and down by rotating a loading button on the control panel, which activates the machine to turn three large spirally threaded shafts on either side of the crosshead (Fig. 5).

Force and displacement targets were used to pause the tests, permitting force and deformation measurements, as well as permitting photographs to be taken. Cracks were marked with felt pens so they would be clearly visible in photographs, with force or displacement target levels written adjacent to the crack and a perpendicular tick mark given that indicates the end of the crack at that target level. Thus, following the test, the development and growth of the cracks are clear in the photographs by the series of tick marks and target levels provided.

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 distinct advantage over force targets because it is guaranteed that the target level can be reached, regardless of whether the force degrades. Note that the targets are used only to get the structure close to the level of interest.

Once the force or displacement target level is approximately achieved, the test is halted and the measured force and displacements are accurately recorded. It is important to note that the chosen displacement target represents the absolute displacement at midspan, measured from a digital readout displacement gauge. As the crosshead was moving, the digital displacement gauge was watched and the crosshead was paused when the gauge approximately reached the target. Then the displacements and forces were read and recorded.

Measured, observed, and predicted results are discussed for each of the five tests. Sometimes, the term *relative displacement* is used, and at other times the terms *target displacement*
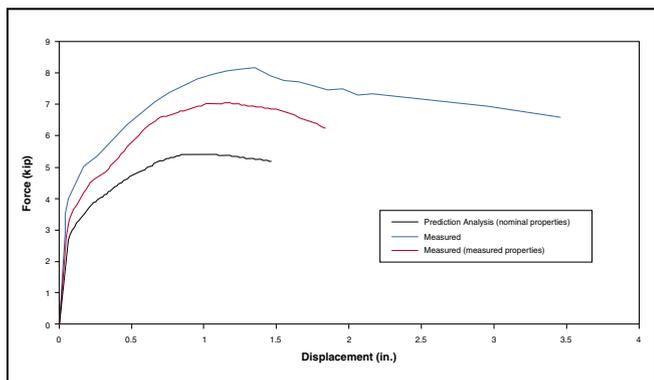
Fig. 11. Precast concrete deck panel, measured and predicted behaviors, Test 1. Note: 1 in. = 25.4 mm; 1 kip = 454 kg.

or *absolute displacement* 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 midspan but is not the final measured value. Absolute displacement is the measured total displacement that includes structural deformations and any deformations from the bearing pads.

Because the ends of the deck panels were supported on elastomeric bearing pads, it was important to measure the

Fig. 12. Displacement target of 0.6 in. (15.2 mm), Test 1.

amount they compressed so that this could be removed from the measured absolute structure displacements, resulting in measured relative deformations of the structure that can be directly compared to the predicted analytical response that assumed rigid supports.

For the four tests with a CIP topping, displacements were measured at midspan, the one-quarter points, and the bearing supports so that the displaced shapes could be plotted at any displacement level (measured versus predicted displacement profiles are given previously in this article). Results and observations for each of the five tests are given in the following sections.

Test 1: Precast Concrete Deck Panel without CIP Topping

Force-deformation results from the first test show that the force and deformation capacity of the precast concrete slab are somewhat greater than expected (**Fig. 11**). Note that two analysis results are shown in Fig. 11 because in the original prediction the concrete strength was taken as 4 ksi (28 MPa) and the prestressing strand ultimate strength was assumed to be 270 ksi (1860 MPa). However, measured material strengths of approximately 7 ksi (50 MPa) for precast concrete and 328 ksi (2260 MPa) for prestressing strand were much higher than assumed, requiring the second analysis shown in the figure (which resulted in much closer outcomes). Modification to the material strengths was the only change to the prediction model.

Cracking began at the target force of 4.275 kip (19.0 kN) with a full-width crack developing on the bottom of the slab at midspan at the maximum moment location. Additional cracks formed on either side of the center crack at the 0.3 in. (7.6 mm) target displacement. At the target displacement of 0.6 in. (15.2 mm), well-distributed cracking was observed with an average spacing of 4.5 in. (114 mm) (**Fig. 12**).

Concrete spalling began at the structure centerline on the top surface at the target displacement of 1.8 in. (46 mm). Large flexural cracks opened up at 1.5 in. (38 mm) of vertical displacement and became much wider at 2.5 in. (64 mm) of vertical displacement. The crack width at the center of the slab was measured to be about 0.59 in. (15 mm) at 2.5 in. (64 mm) vertical displacement.

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 in. (447 mm), which is in good agreement with the assumed value of 18.75 in. (476 mm) (50 strand diameters) used in the predictions for all of the positive bending tests.

By 3 in. (75 mm) of vertical displacement, the center crack was so large that some vertical shear slip was evident. Following the test and removal of the loading beam and bearing pad, the top surface spalling at the maximum moment location was clear. The test was stopped at the very large target displacement of 3.5 in. (89 mm) following continued strength degradation with increasing displacement.

No sudden failure was observed and the test was only 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 in. (33.6 mm)—relative displacement of 1.35 in. (34.3 mm)—with a maximum

Fig. 13. Displacement target of 0.6 in. (15.2 mm), Test 2.

Fig. 14. Displacement target of 1.0 in. (25.4 mm), Test 2.

force of 8.15 kip (36.3 kN) (Fig. 11). Predicted maximum force and associated relative displacement were 7.02 kip (31.2 kN) and 1.2 in. (31 mm), respectively (Fig. 11). Predicted displacement capacity was 1.84 in. (46.7 mm) at a compressive strain for the unconfined concrete of 0.005.

Test 2: Precast Concrete Deck Panel with Coarse Broom Finish and CIP Topping Slab

Flexural cracking began at 25 kip (111 kN). 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 part of the thickness of the member.

More extensive cracking developed at the target vertical displacement of 0.5 in. (13 mm) (with an average crack spacing of 5 in. [127 mm]), and the tops of the cracks began to rotate from the vertical as they extended higher, indicating the influence of shear. Inclined cracking became much more extensive at the target displacement of 0.6 in. (15.2 mm), with significant flexure-shear cracks forming (**Fig. 13**).

Note that the crack progression through the section at different target levels can be seen from Fig. 13 by reading the felt pen numbers adjacent to transverse tick marks that indicate the end of the crack up to that target level. Thus, one photograph taken near the end of the test can show the complete

history of a crack, with a series of tick marks indicating its progression through the different loading target levels.

The flexure-shear cracks began as flexure cracks 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 the applied load and reaction points.
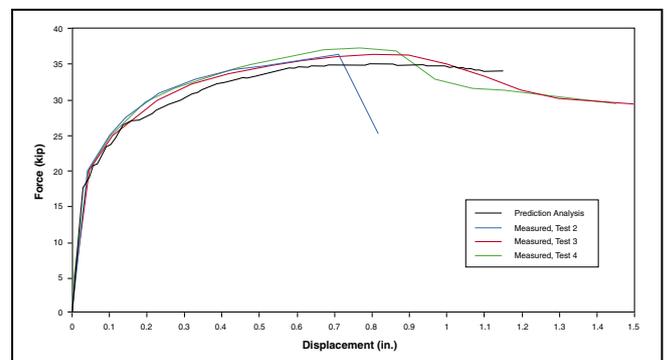
Fig. 15. Precast concrete panel and cast-in-place slab, measured and predicted behaviors, Tests 2–4.
Note: 1 in. = 25.4 mm; 1 kip = 454 kg.
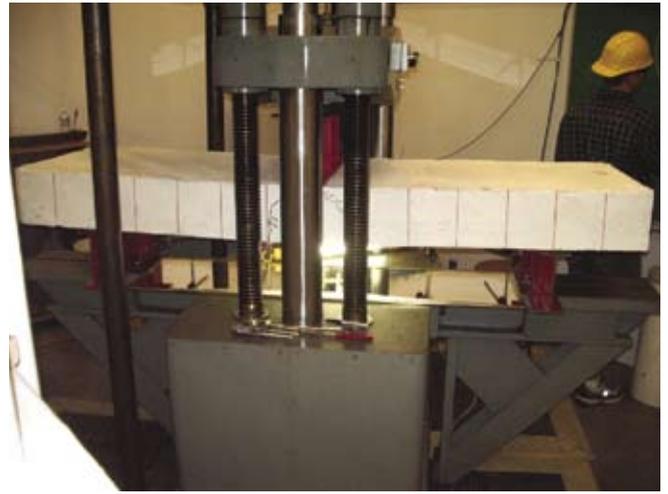
Fig. 16. Displacement target of 1.0 in. (25.4 mm), Test 3.

Fig. 17. Displacement target of 1.0 in. (25.4 mm), Test 4.

Concrete spalling started at 0.6 in. (15.2 mm) target displacement. More extensive flexure-shear cracks developed at 0.8 in. (20.3 mm) of displacement, resulting in significant loss in force. Wider flexure-shear cracks were observed at 0.9 in. (22.9 mm) target displacement. Sudden shear failure occurred at 1.0 in. (25.4 mm) target displacement, ending the test (**Fig. 14**).

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 (Fig. 14). Prediction analyses and real-time measured test data were projected on the laboratory wall so that visitors could watch the comparisons as they developed.

Note that no horizontal shear slip was observed or measured between the precast concrete panels and CIP slabs, indicating that fully composite behavior was maintained throughout the test. This is also demonstrated by the close correlation between the measured and predicted force-deformation responses (**Fig. 15**).

Vertical red lines spaced at 6 in. (150 mm) remained straight and continuous across slab boundaries, with no horizontal offset at the slab interface, also validating that no horizontal shear slip had occurred. The maximum measured force was 36.5 kip (162 kN) at 0.71 in. (18.0 mm) relative displacement versus the predicted maximum force of

35.0 kip (156 kN) at a relative displacement of 0.86 in. (21.8 mm). Predicted relative displacement capacity was 1.15 in. (29.2 mm) at a compressive strain of 0.005, compared with the measured relative displacement at failure of 0.946 (24.0 mm). Measured and predicted force-deformation results are given in Fig. 15.

Test 3: Precast Concrete Deck Panel with Medium Broom Finish and CIP Topping Slab

A small partial-width flexural crack developed at the 15 kip (66.8 kN) target level with a full-width crack forming at 20 kip (89.0 kN). More extensive cracking occurred at 25 kip (111 kN) with 5 in. (127 mm) average crack spacing. Flexure-shear cracks started to develop at the 0.3 in. (7.6 mm) target displacement and increased at the 0.4 in. (10.2 mm) target.

Spalling initiated at the 0.5 in. (13 mm) target and became more severe at the 0.7 in. (17.8 mm) target displacement. By the 0.9 in. (22.9 mm) target displacement, fully extended flexure-shear cracks had developed on either side of the load point. At the 1 in. (25 mm) target level, the flexure-shear cracks were visibly widening and extending toward each other (**Fig. 16**).

Flexure-shear cracks widened more at target displacements of 1.2 in. and 1.3 in. (31 mm and 33 mm) and formed a long

arc from either side of the structure centerline, extending toward the maximum moment location under the applied load. Spalling at the top of the slab at the centerline became more extensive, with a compression arch forming in the opposite direction to the flexure-shear cracking.

The future failure plane was clear from the extended flexure-shear crack at a target displacement of 1.7 in. (43.2 mm). Shear failure occurred at approximately 2.5 in. (63.5 mm) target displacement based on scaling measured one-fourth span displacements (centerline displacement gauges were lost due to extensive spalling and cracking).

The measured crack width where failure occurred opened up from a stable 0.315 in. (8 mm) just before failure, to 3.54 in. (90 mm) following failure. The maximum measured force was 36.2 kip (161 kN) at a relative displacement of 0.9 in. (22.9 mm) compared with the predicted maximum force of 35.0 kip (156 kN) at a relative displacement of 0.86 in. (21.8 mm). Measured and predicted force-deformation plots are given in Fig. 15.

The predicted relative displacement capacity was 1.15 in. (29.2 mm) at a compressive strain of 0.005 compared with the measured relative displacement at a shear failure of about 2.3 in. (58.4 mm). There was no indication of horizontal shear slip between the precast concrete panel and CIP slab, with flexure and flexure-shear cracks forming across boundaries as if they were one slab.

Test 4: Precast Concrete Deck Panel with Carpet Drag Finish and CIP Topping Slab

Flexural cracking began at the 25 kip (111 kN) target load. At the 0.3 in. (7.6 mm) target displacement, the flexure cracks started to turn into flexure-shear cracks and continued to extend and turn at the 0.4 in. (10.2 mm) target level. Onset of crushing started at the 0.7 in. (17.8 mm) target displacement.

A loss in force occurred at the 1 in. (25 mm) target level (0.865 in. [22.0 mm]) relative displacement (**Fig. 17**), 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 in. and 1.4 in. (31 mm and 36 mm) target displacement levels.

Shear failure occurred at 3 in. (76 mm) of relative displacement, measured with a taught string line between centerline of supports following shear failure (before removing the load). String measurement was required because the displacement gauge targets came off due to extensive spalling and cracking in the critical section region.

The last measured target was 2.8 in. (71 mm) (2.65 in. [67.3 mm] relative displacement). The maximum measured force was 37.3 kip (166 kN) at a relative displacement of 0.77 in. (19.6 mm) compared with the predicted maximum force of 35.0 kip (156 kN) at a relative displacement of 0.86 in. (21.8 mm) (Fig. 15).

The predicted relative displacement capacity was 1.15 in. (29.2 mm) at a compressive strain of 0.005 compared with the measured relative displacement at failure of 3 in. (76 mm). There was no indication of horizontal shear slip between the precast concrete panel and CIP slab, with flex-

ure and flexure-shear cracks forming across boundaries as if they were one slab.

Test 5: Precast Concrete Deck Panel with Coarse Broom Finish and CIP Topping Slab—Negative Bending

Flexural cracking began at 10 kip (44.5 kN), with more full-width cracks occurring at 15 kip (66.8 kN) (**Fig. 18**). By the target displacement of 0.3 in. (7.6 mm), previously formed flexure cracks started turning in as they extended upward, showing the influence of shear. By the target displacement of 0.8 in. (20.3 mm), the flexure-shear cracks had taken form but were not yet very wide. The flexure-shear cracks crossed the marked vertical red lines without any slip between the slabs.

At a target displacement of 1 in. (25 mm), the flexure-shear cracks began to widen significantly, resulting in a sudden shear failure at a target of 1.2 in. (31 mm) (**Fig. 19**). In the top picture of this figure, a short horizontal crack is seen on the right side at the interface between the slabs that developed at the 0.4 in. (10.2 mm) target level. The short interface crack in this negative bending test did not result in any horizontal slip between the slabs and had no effect on the final failure mode. This type of interface crack was not seen in any of the positive bending tests.

Note that the final failure plane follows a straight, inclined shear line that is an extension of the previously formed
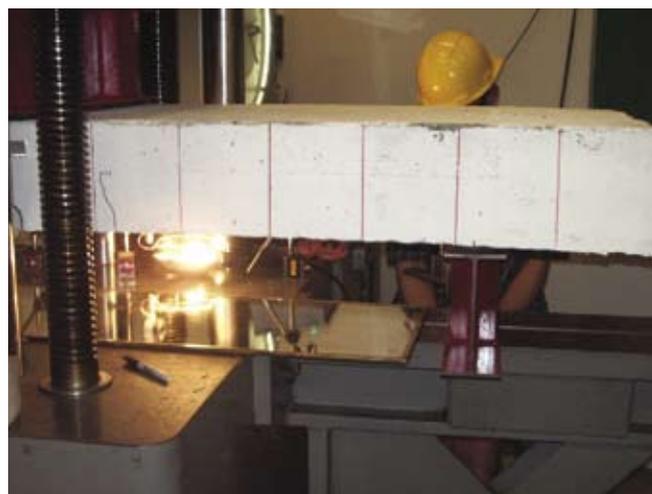
Fig. 18. Force target of 15 kip (67 kN), Test 5.
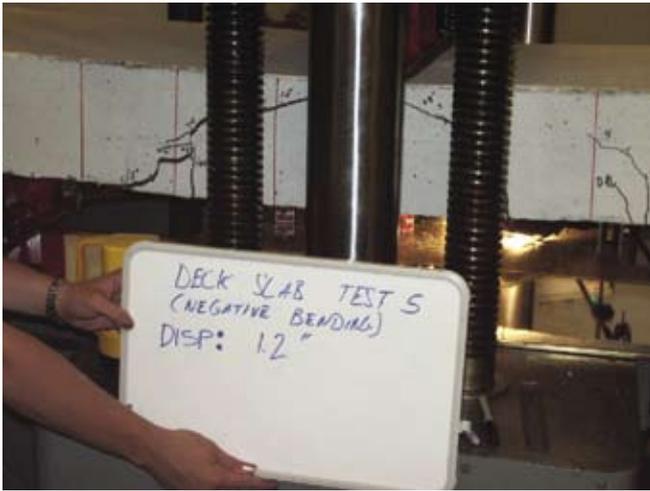
Fig. 19. Displacement target of 1.2 in. (31 mm), Test 5.

flexure-shear crack (the failure plane extends across the top of the original flexure crack and continues at the angle of the flexure-shear crack, allowing the structure to separate into two pieces).

The maximum measured force is 36.9 kip (164 kN) at a displacement of 0.83 in. (21.1 mm) compared with the maximum predicted force of 30.5 kip (136 kN) at a displacement of 0.85 in. (21.6 mm). The predicted relative displacement capacity was 1.04 in. (26.4 mm) at a compressive strain of 0.005 compared with the measured relative displacement at a shear failure of 1.06 in. (26.9 mm).

This test was conducted in negative bending (upside down), with the precast concrete panel at the top and the CIP slab at the bottom. Additional reinforcement was used in the CIP slab (four #5 bars) for this test. In the original prediction analysis, the added reinforcing bar was missed, resulting in predicted force levels that are about one-half of the measured values (**Fig. 20**). This was realized while the test was in progress, and following the test a second analysis (modified prediction) was conducted that included the four #5 bars rather than the two #5 bars used in the original prediction analysis.

Such a modification brought the force levels much closer to the measured results. For these prediction analyses, the stress-strain properties of the #5 bars were assumed and closer correlation between measured and analysis behavior is found in the post-test analysis given in Fig. 20. Because the #5 reinforcing bar samples could not be found following the deck tests, a couple of sample #5 bars were removed by jack-hammer from the uncracked region of one of the deck units and tested to failure, providing measured material properties for the #5 bars and improved force-deformation analysis results as indicated in Fig. 20.

CONCLUSION

A series of full-scale structural tests was performed to failure in the Structural Engineering Laboratory at SDSU to characterize the response of precast, prestressed concrete bridge deck panels with CIP deck topping. One test was performed on the precast concrete deck panel by itself (Test 1) with no CIP topping, while the remaining four tests consisted of the precast concrete panel with various levels of surface roughening and a CIP topping slab (composite tests). One of the four 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 composite decks (precast concrete panels and CIP slabs) would act as a single composite slab in flexure, with no horizontal shear slip at the slab interface. Because no reinforcement crosses the plane between slabs, various surface-roughening techniques were applied to the top of the precast concrete deck panel to help transfer horizontal shear stresses that develop from composite action and the simple beam loading applied. These included coarse broom and medium broom brush finishes and a carpet drag finish.

Based on the results of this investigation, the following conclusions can be made:

- All five test specimens exceeded their nominal and ultimate force capacities from the prediction analyses.
- No sign of horizontal shear slip was observed in any of the four composite deck tests all the way to failure.
- Flexure cracks and flexure-shear cracks formed across slab boundaries as if the precast concrete panels and CIP slabs were cast as one, with minor exceptions for negative loading.
- All of the vertical red lines that were marked on both sides of the test units at 6 in. (150 mm) spacing remained straight and continuous across slab boundaries throughout the tests to failure, also verifying that no horizontal shear slip developed between the slabs.
- Following large flexural displacements and loss in force associated with wide-open flexure-shear cracks, all four 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 a flexure crack. In all cases, the final failure plane cut across the top of the original vertical flexure crack separating the test units in two.
- As the tests progressed, one of the flexure-shear cracks widened 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 (0.6 m)
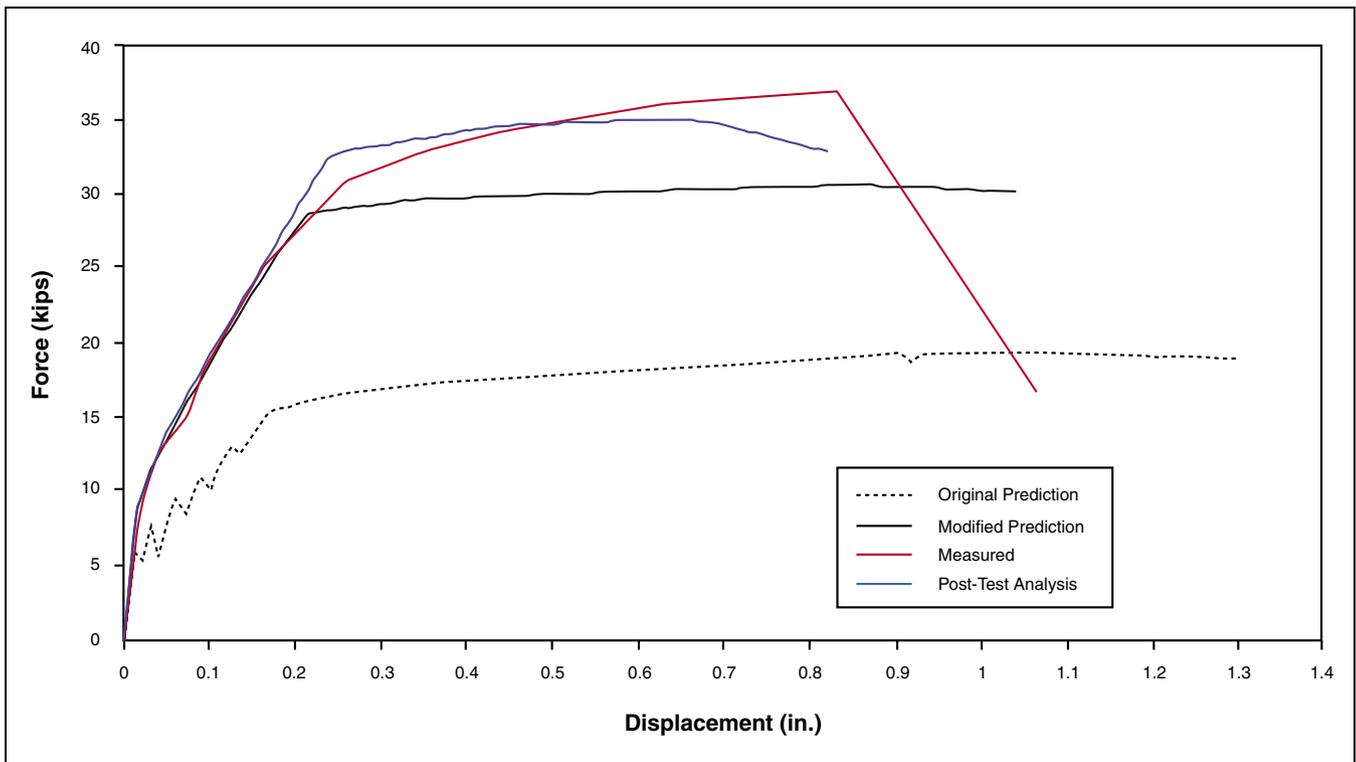
Fig. 20. Precast concrete panel and cast-in-place slab (negative bending), measured and predicted behaviors, Test 5.
Note: 1 in. = 25.4 mm; 1 kip = 454 kg.

width of the structure, indicating that the steel I-beams properly spread the applied load and reactions across the full deck width.

RECOMMENDATIONS

It is recommended that this precast concrete panel and CIP bridge deck system be used for new bridge construction to save time and reduce construction costs. Any of the precast concrete slab roughening techniques used in this testing program will work to prevent horizontal 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.

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