

Interface Horizontal Shear Strength in Composite Decks With Precast Concrete Panels



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Six stay-in-place precast, prestressed deck panels were tested to evaluate the horizontal shear strength between the precast panel and the cast-in-place portion of the bridge deck. Presently, the Indiana Department of Transportation requires a minimum of 20 shear connectors regardless of panel dimensions. The shear connectors are placed across a broom finished deck panel surface. Broom finish with approximately 0.05 to 0.075 in. (1.27 to 1.91 mm) total amplitude deformations is specified instead of a raked finish with 0.25 in. (6.35 mm) total amplitude because of the reduced 2.5 in. (63.5 mm) thickness of the panels used in Indiana. In this study, composite behavior at ultimate, including horizontal shear and interface slip characteristics, was evaluated. It is concluded that stay-in-place precast, prestressed deck panels with a broom finished surface do not require horizontal shear connectors if the nominal average horizontal shear stress at the interface is less than 116 psi (0.8 MPa).

One method of bridge construction consists of using precast, prestressed concrete deck panels as stay-in-place forms in conjunction with a cast-in-place concrete topping, replacing the traditional monolithic roadway deck slabs. The use of precast concrete for new bridge construction and for the rehabilitation of deteriorated bridges is economically and structurally attractive.^{1,2} Durability, ease and speed of construction,³ together with reduced need

for maintenance, are all advantages in using precast concrete.⁴

Composite construction is an economical way of combining precast and cast-in-place concrete and at the same time retaining the continuity and efficiency of monolithic construction. This method is more economical than the conventional monolithic deck construction due to the considerable field labor savings involved.^{5,6} These panels span the opening between the longitudinal bridge girders and serve as per-

Table 1. Concrete strengths and prestress losses of various specimens.

Specimen	Description of specimen	28-day concrete strength		Total prestress loss (percent)
		Precast (psi)	Cast-in-place (psi)	
1	Specimen with 18 strands and no shear connectors	7850	6932	14.8
2	Specimen with 18 strands and four shear connectors	7850	6932	14.9
3	Specimen with 18 strands and four shear connectors	5647	5747	12.3
4	Specimen with lubricated interface, 18 strands, and four shear connectors	5647	5747	13
5	Specimen with 12 strands and no shear connectors	6857	5657	14.9
6	Specimen with 12 strands and four shear connectors	6857	5657	15

Note: 1 in. = 25.4 mm; 1 psi = 0.00689 MPa.

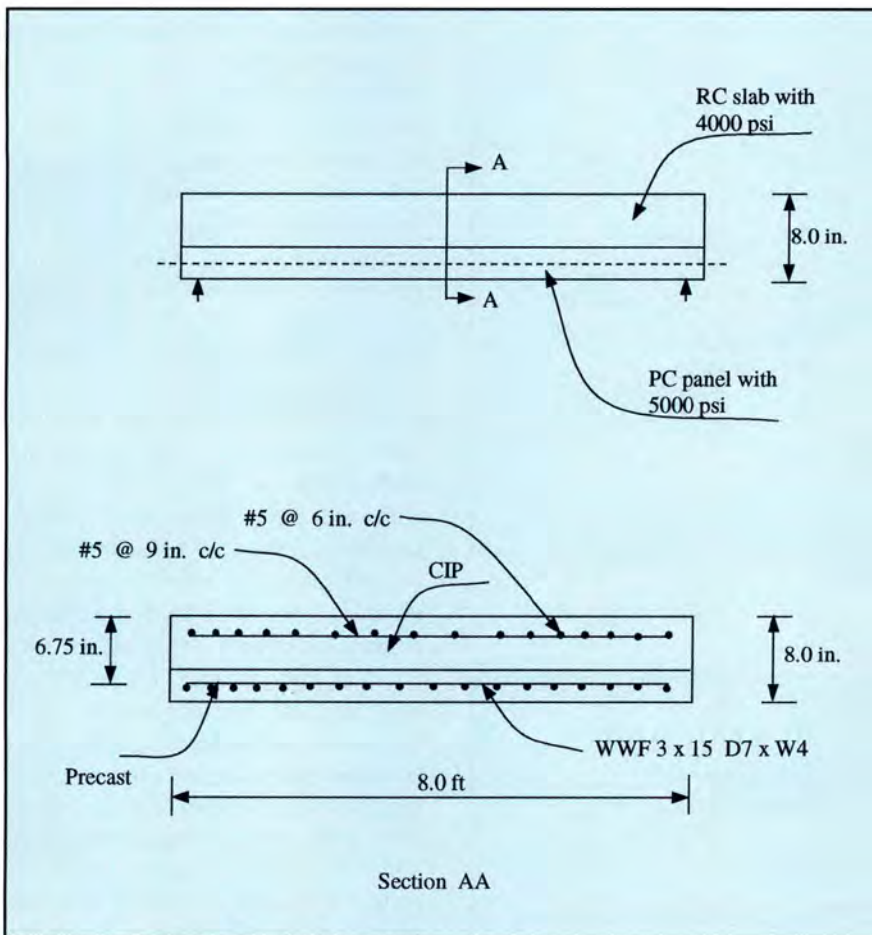


Fig. 1. Geometrical characteristics of deck slab.

shear connectors act as a combination of dowel and shear-friction reinforcement once the slip occurs at the interface. Currently, the Indiana Department of Transportation (INDOT) requires a minimum of 20 shear connectors across a broom finished interface of the deck panel. Broom finish is specified instead of raked finish with full-amplitude deformations of 0.25 in. (6.35 mm) due to the 2.5 in. (63.5 mm) thick panels [thinner than the normal 3.0 in. (76.2 mm)] used in Indiana.

This paper presents the results of an experimental investigation⁷ to evaluate the horizontal shear strength in broom finished surfaces and examines the need for shear connectors at service and ultimate load levels to obtain adequate composite action. This evaluation includes the performance of this type of bridge deck section under static and repeated loading.

This study addresses the specific concerns raised by INDOT dealing with the number of horizontal shear connectors required in precast deck panels with a broom finished top surface. The issues of potential horizontal slip at the interface, as well as the adequate development of positive moment and shear capacities of the composite bridge section, were examined experimentally.

LITERATURE REVIEW

Barker⁸ showed that specimens with and without the shear connectors developed adequate bond to ensure composite action. There was no indication that the shear reinforcement increased the bond performance under normal service load conditions. However,

manent forms for the cast-in-place concrete topping that completes the bridge deck.

The assumed performance of a composite element depends on the premise that precast and cast-in-place elements behave as a single unit. This monolithic behavior is possible only if the horizontal shear resulting from bending of the deck is effectively transferred at the interface between the pre-

cast and cast-in-place elements. Because panels and cast-in-place topping are cast at different times, there is a potential weak plane at the interface.

In general, horizontal shear stresses are transmitted across the interface due to chemical and mechanical bond between the precast panel and cast-in-place concrete acting in conjunction with any horizontal shear connectors provided across the interface. These

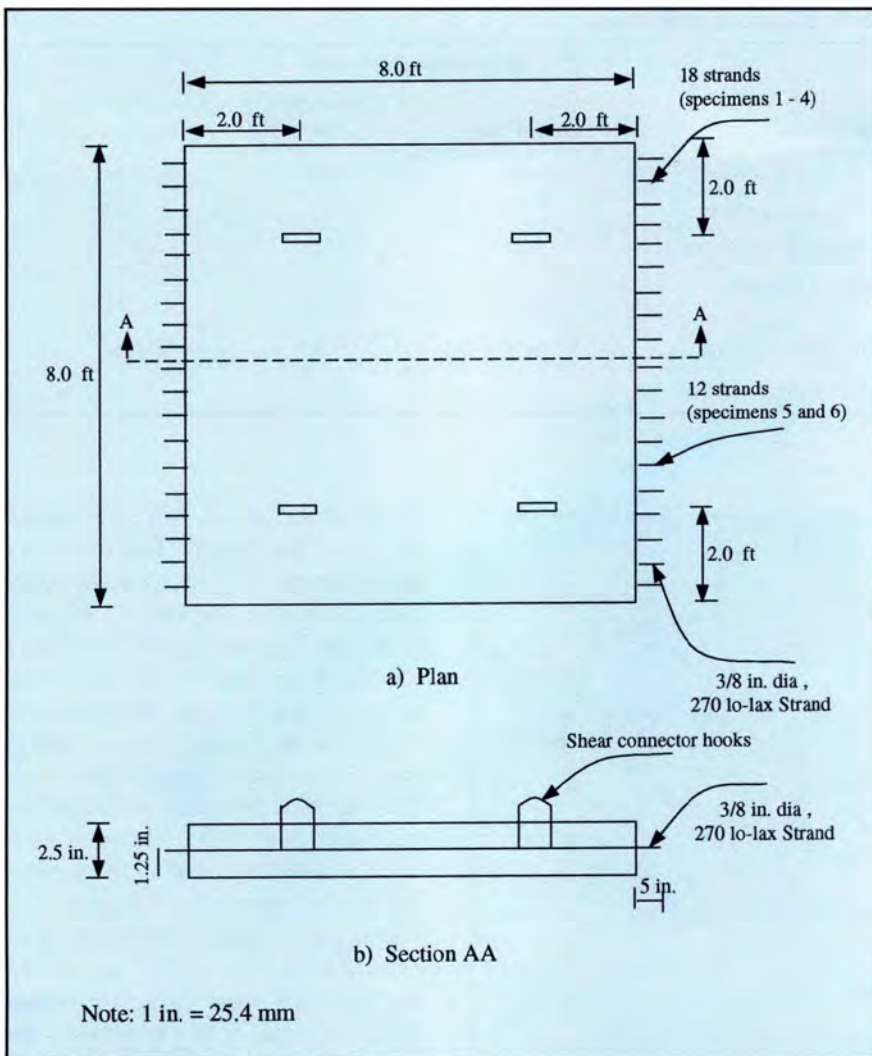


Fig. 2. Typical details of a panel with four stirrups.

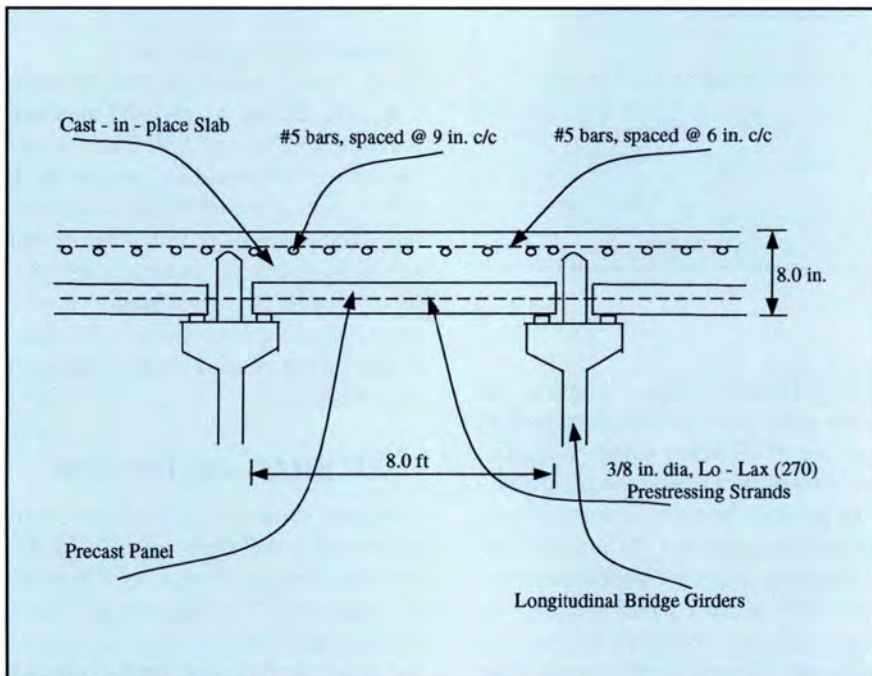


Fig. 3. Cross section details of bridge deck.

there was some indication that the panels with shear reinforcement were tougher under cyclic loading and appeared to be stiffer under higher static loads.

Barnoff et al.⁹ examined the behavior of a full-scale prestressed bridge with precast deck panels. This study concluded that full-composite behavior was observed at both service loads and overloads. Mechanical shear connectors are not required if the panel surface is given a scored finish. In this reference, the amount of roughness of the surface was not mentioned. Cyclic loading was not shown to have any detrimental effect on the performance of panels and the plane section hypothesis seemed to be valid even under loads 3.5 times the design load.

Hanson¹⁰ correlated push-off test results with those of girder tests. With this correlation he considered that the maximum horizontal shear strength of the beams tested was reached at slips of about 0.005 in. (0.13 mm). He pointed out that larger shear strengths would be recorded if larger slips were permitted. The girder and push-off tests reported a maximum shearing stress of 500 psi (3.45 MPa) to ensure composite action in the case of a rough interface ($3/8$ in. = 9.53 mm amplitude) and 300 psi (2.07 MPa) for a smooth interface (contact surface trowelled to a relatively smooth condition).

Shear connectors were effective only after the interface slip occurred. Girder tests indicated that when bond is absent at the connection, interface roughness can contribute up to 150 psi (1.04 MPa) towards shear strength. In addition to these values, approximately 175 psi (1.21 MPa) shear capacity can be added for each percent shear reinforcement crossing the interface. Finally, this study¹⁰ concluded that shear reinforcement across the interface was effective only at relative displacements of more than 0.005 in. (0.13 mm) at the interface.

Paulay et al.¹¹ conducted a comprehensive test series on horizontal construction joints in cast-in-place reinforced concrete with inter-layer reinforcement. They concluded that, for design purposes, contribution from the dowel action of the reinforcement should be ignored as significant dowel

forces can be generated only at excessive slip along a joint.

Seible and Latham^{12,13} concluded that the horizontal load transfer behavior of overlaid reinforced concrete slabs used frequently in bridge deck rehabilitation depends largely on the interface surface preparation. They also noted that horizontal shear reinforcement provided across a rough and clean horizontal construction joint interface cannot be effective unless delamination and interface slip occur.

Furthermore, the amount of minimum dowel reinforcement [0.083 percent for $f_y = 60$ ksi (413 MPa)] currently required by AASHTO¹⁴ is not sufficient to control the horizontal slip after the onset of delamination. They also concluded that small topological changes in the interface of about 0.125 in. (3.18 mm) deep at 1 in. (25.4 mm) spacing suffice to provide monolithic behavior up to critical flexural yield limit states, which make dowels virtually ineffective.

In the state of Indiana, a broom finish [approximately 0.05 to 0.075 in. (1.27 to 1.91 mm)] is given at the interface. The decrease in roughness and the irregular pattern of deformations associated with this finish justify the evaluation of the bond across the interface.

TEST PROGRAM

An experimental program involving six full-scale composite stay-in-place deck slabs was conducted. These specimens had different interface properties including different amounts of horizontal shear reinforcement across the interface. The first four specimens had 18 prestressing strands representing an upper limit for the amount of flexural reinforcement in this type of member. This allowed performance evaluation under the potential maximum horizontal shear demand.

The last two specimens had 12 prestressing strands. These specimens were used to evaluate the effect of repeated loading by causing an increase in mean shear stress at the interface. The shear stress corresponded to the level that needs to be transferred in deck panels designed according to AASHTO¹⁴ for HS20 loading on an

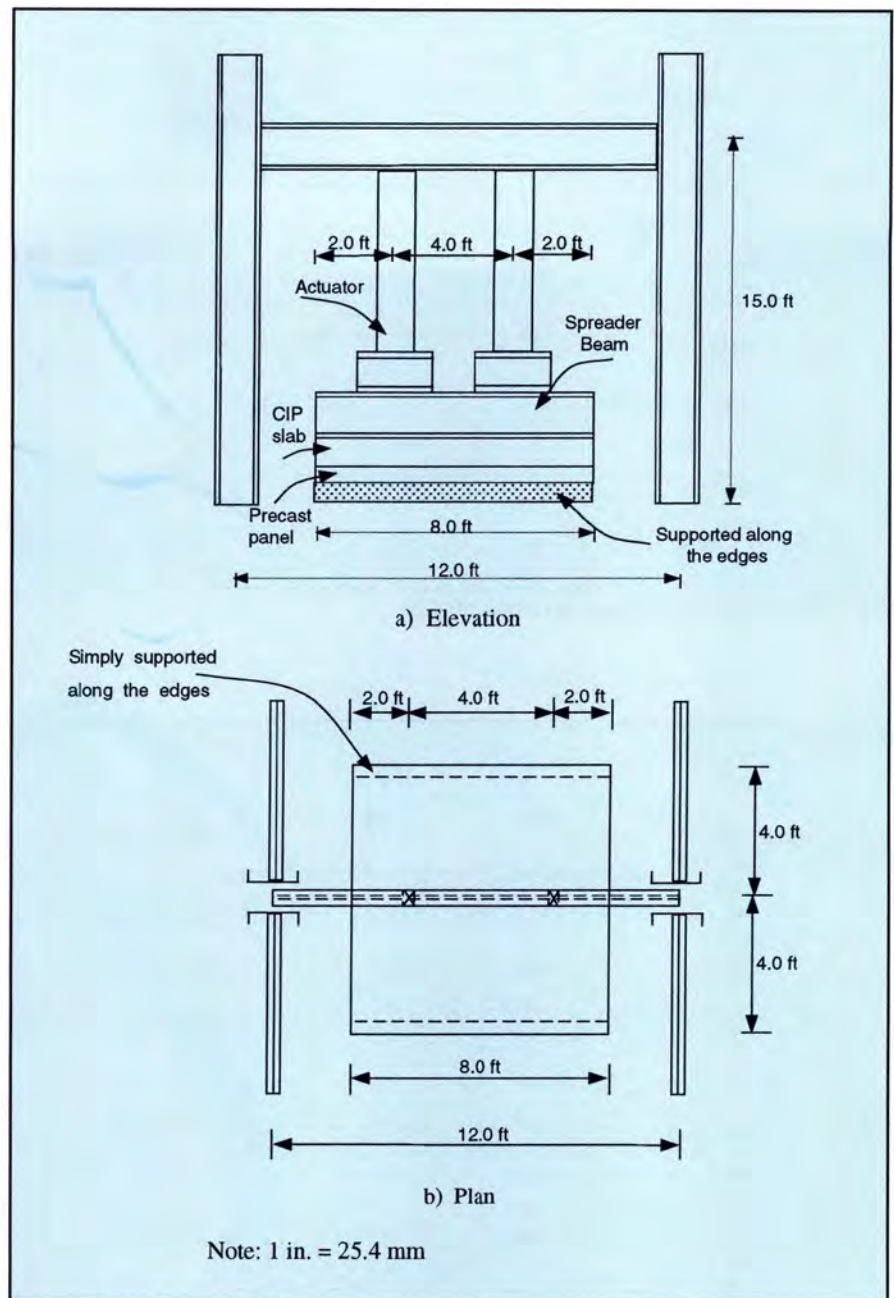


Fig. 4. Loading frame details.

8 ft (2.44 m) long simple span.

All six composite specimens consisted of a 2.5 in. (6.35 mm) thick precast, prestressed panel with an additional 5.5 in. (13.97 cm) thick cast-in-place reinforced concrete topping slab. Thus, an 8 in. (203.2 mm) standard total deck thickness, as commonly used in Indiana, is provided. The details of the various specimens are given in Table 1. Fig. 1 shows the geometrical characteristics of the composite specimens. Deck panel plan and elevation details are shown in Fig. 2. Locations of shear connectors are also indicated in Fig. 2.

In this study, four panels had four shear connectors as horizontal shear reinforcement. These four shear connectors represent a minimum practical limit for handling. Two panels, Specimens 1 and 5, had no shear connectors. In Specimen 4 with four shear connectors, bond was broken by applying a coating of form oil in order to assess the contribution of the shear connectors under conditions of bond deterioration.

A typical transverse section of a bridge superstructure is shown in Fig. 3. The precast panels rest on top of longitudinal bridge girders with some

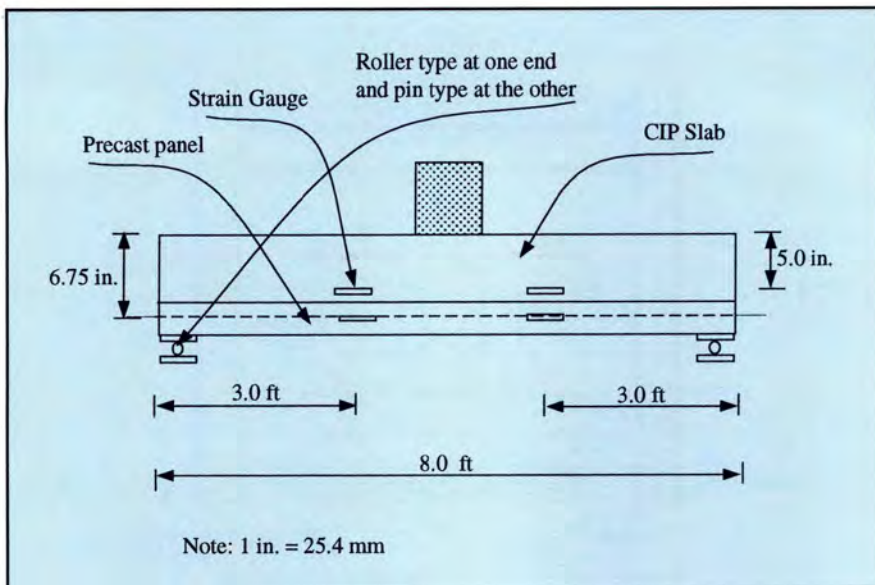


Fig. 5. Strain gauge locations on sides of deck.

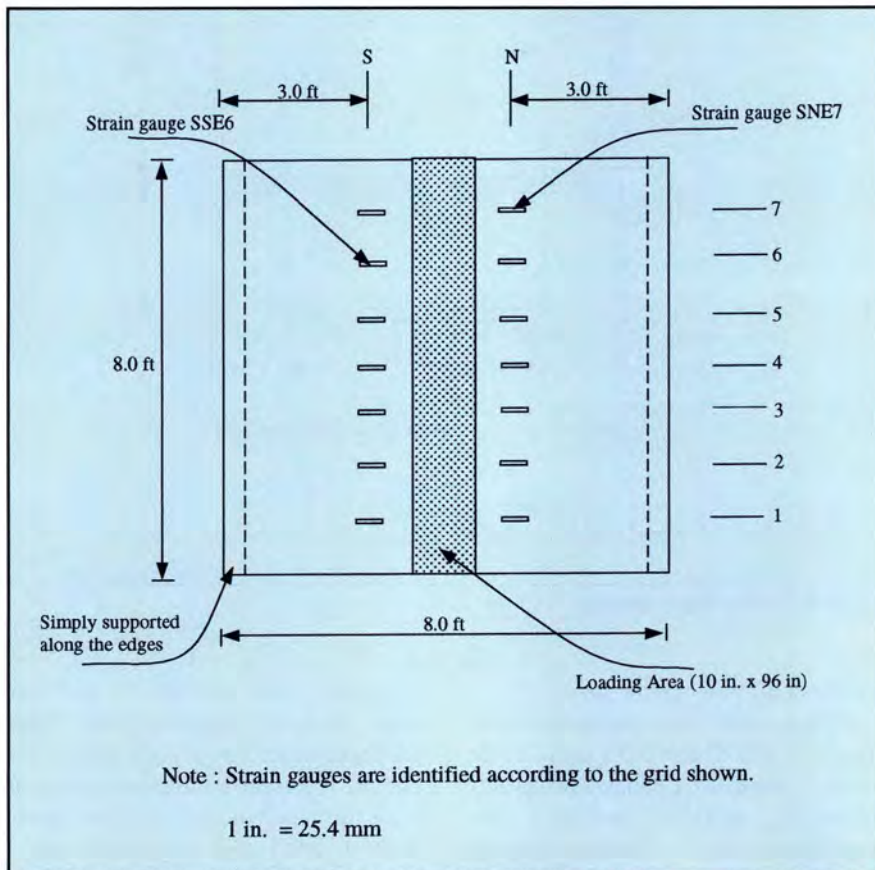


Fig. 6. Strain gauge locations on top of cast-in-place slab.

form of bearing material in between, and then the topping slab is placed on top of these deck panels. Strand extensions are typically used to anchor these deck panels¹⁵ and the entire positive moment reinforcement is kept in

the precast panels. The topping slab contains nominal reinforcement to take care of the temperature and shrinkage stresses.

In Indiana, a standard panel width of 8 ft (2.44 m) is used. The longitudinal

dimension (length) of the panel depends on the spacing of the longitudinal bridge girders. A length of 8 ft (2.44 m) was chosen as it falls in the range of medium to upper girder spacings. Hence, the dimensions of the composite slabs were 8 x 8 ft x 8 in. (2.44 x 2.44 m x 203.2 mm).

Though in reality, the cast-in-place topping slab offers restraint against rotation inducing negative moments at the ends of the precast panel, the specimens were tested in the simply supported condition. The simply supported condition, in fact, represents the worst possible case for the precast element in terms of loading.

Panels 1 through 4 contained 18 ³/₈ in. (9.53 mm) diameter seven-wire Grade 270 ksi (1862 MPa) low-relaxation prestressing strands positioned at mid-depth of the panel. Specimens 5 and 6 had 12 strands of the same type and size positioned at the mid-depth of the panel. Before the concrete was placed, the strands were prestressed to about 17.5 kips (77.88 kN), approximately equal to $0.75f_{pu}$, where f_{pu} is the ultimate strand tensile strength.

Each panel had a single layer of (WWF 3 X 15 D7 X W4) welded wire fabric of Grade 60 (413 MPa) located directly on top of the prestressing strands. In addition, the panels contained five #3 (9.53 mm) Grade 60 (413 MPa) reinforcing bars at the bottom of the panel. The prestressing steel in the panel runs perpendicular to the longitudinal bridge girders.

Reinforcement in the cast-in-place topping slab was similar to that specified for a conventional 8 in. (203.2 mm) bridge deck. The transverse (to the panel span) reinforcement consisted of #5 (15.88 mm) bars spaced at 9 in. (228.6 mm) on centers supported on 1.75 in. (44.45 mm) high bar chairs spaced at approximately 2 ft (609.6 mm) on centers.

The bar chairs rested directly on the top surface of the precast panels. The longitudinal reinforcement (parallel to the panel span) consisted of #5 (15.88 mm) bars spaced at 6 in. (152.4 mm) on centers placed on top of the transverse reinforcement. All the reinforcement in the cast-in-place topping was Grade 60 (413 MPa) steel.

Test Setup

The test setup is shown in Fig. 4. The two columns are 12 ft (3.66 m) apart and support a cross beam. The cross beam in turn supports two hydraulic actuators. The load is distributed to the specimen by means of a spreader beam, which distributes the load as a uniform load applied over the entire width of the panel. The load was distributed over an area of 10 x 96 in. (254 mm x 2.44 m).

As the vehicle moves on a bridge deck, it exerts pressure on the composite section of the bridge deck. Abendroth et al.¹⁶ concluded that punching shear usually governs the ultimate strength of the slab in a monotonic loading situation when the load is distributed over the wheel footprint. In these cases, the full flexural capacity of the panels is usually not achieved due to the short span lengths and concentrated load area and, thus, the panels are not loaded to have maximum uniform horizontal shear stress at the interface across the width of the panel. Thus, the loading area was extended over the entire width of the panel. In this study, the composite behavior is examined in terms of flexural and one-way beam shear capacities.

Instrumentation

Instrumentation was placed to monitor strains and deflections at various locations. To obtain the vertical deflections of the panel, three linear variable differential transducers (LVDTs) were placed on the sides of the panel at distances of 2.5, 4, and 5.5 ft (0.76, 1.22, and 1.68 m) from either of the supported edges.

Strain gauges were used to measure strains on the sides of the specimen (EA-06-20CBW-120) (see Fig. 5) and also to measure strains in the prestressing strands (EA-06-062DN-350). Strain gauges (EA-06-20CBW-120) were also placed on top of the cast-in-place slab, as shown in Fig. 6. On the prestressing strands, gauges were placed at the mid-section of the deck panel, as shown in Figs. 7 and 8.

Strain gauges were also placed on the reinforcing bars, located in the cast-in-place topping slab, at a distance of 3 ft (914.4 mm) from both

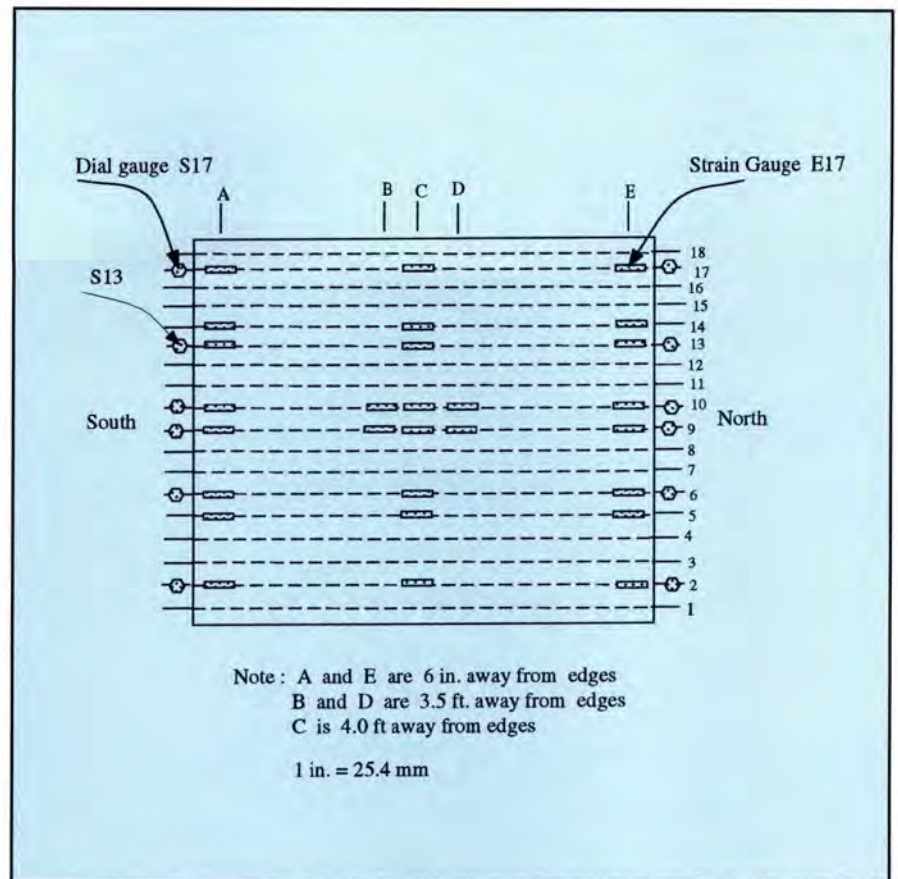


Fig. 7. Strain gauge and dial gauge locations on prestressing strands of Specimens 1 and 2.

supports. These strain gauges were in line with the gauges placed on the sides of panels and also on top of the deck slab. Interface strain gauges were placed at the mid-section of the top surface of precast panel. The interface gauges were placed to estimate the immediate loss in prestress after detensioning as well as subsequent losses up to the casting of the topping slab.

To determine the amount of strand slip, dial gauges were placed on selected strand extensions, as shown in Figs. 7 and 8. To determine the slip between the cast-in-place concrete slab and the prestressed panel, LVDTs were used as slip gauges. Slip between the cast-in-place slab and the prestressed panel is usually associated with diminishing composite behavior between the cast-in-place slab and the precast panel.

Along with horizontal slip, vertical separation also occurs between the cast-in-place slab and the precast panel as the deck loses composite action. This separation usually does not occur at the supports and the loading

point due to the compressive stress flow. Hence, the separation and thereby slip would only take place in between the points which are d effective depth of the slab, away from the loading point and the support reactions. To measure this slip, slip gauges were placed, as shown in Fig. 9.

In the case of panels with shear connectors, strain gauges were placed on the shear connectors as well in order to determine their contribution when slip occurs between the cast-in-place concrete and the precast panel. These shear connectors act as a combination of dowel and shear-friction reinforcement once the slip or separation occurs at the interface.

Test Procedure

All loads were applied by hydraulic actuators. Static load tests were performed at regular intervals between repeated loading applications. The repeated loading consisted of two phases. The load range for the first phase of repeated loading in the four

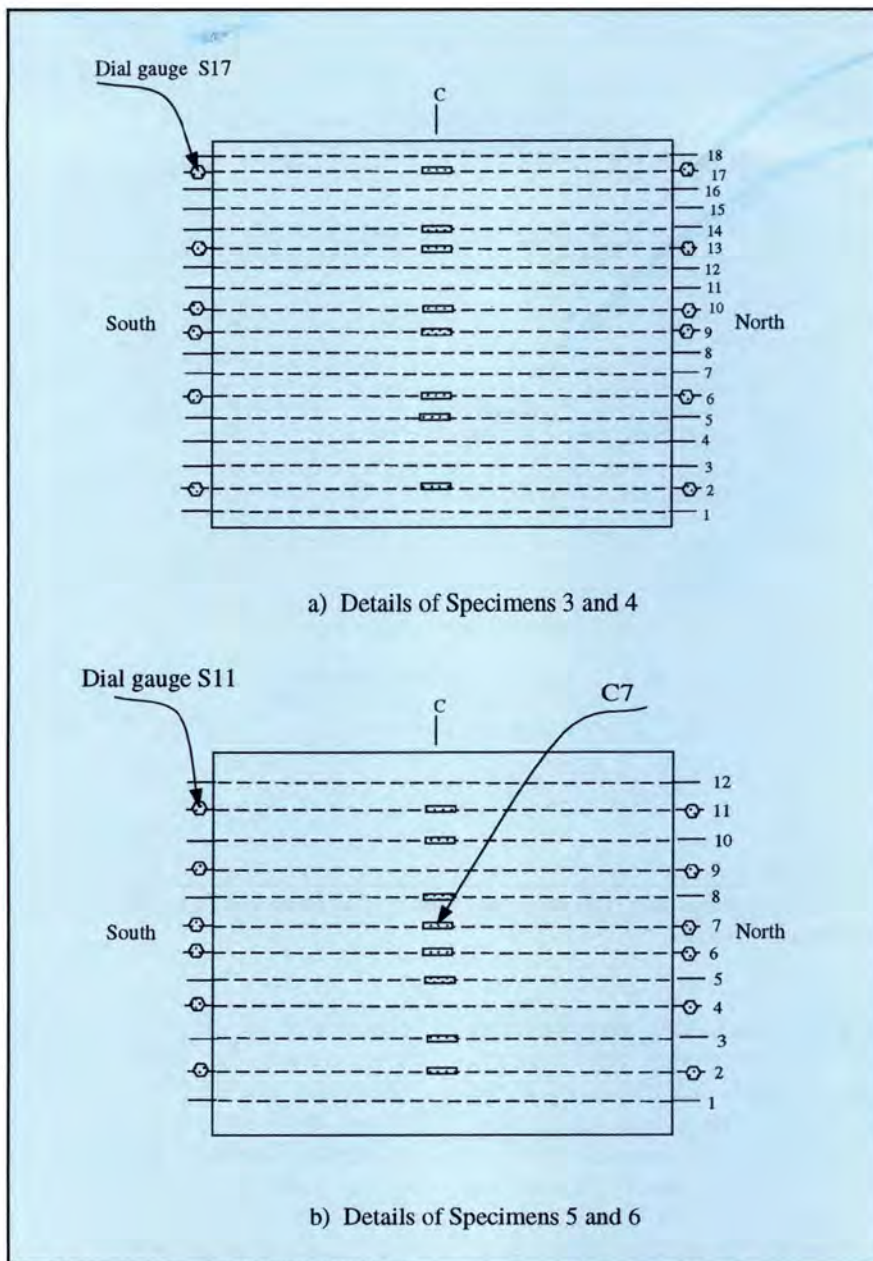


Fig. 8. Strain gauge and dial gauge locations on the prestressing strands.

Table 2. Estimated capacities of specimens based on AASHTO Specifications.

Specimen	Cracking load (kips)	Strand slip capacity* (kips)	Flexural capacity† (kips)	Horizontal shear capacity (kips)	Beam shear capacity (kips)
1 to 4	60	99	158	98	158
5 and 6	40	62	128	98	158

Note: 1 kip = 4.46 kN.

* Strand slip capacity based on the development length criterion of AASHTO.

† Flexural capacity including the contribution from the cast-in-place reinforcing bars and the wire mesh.

panels with 18 strands was from 26.4 to 38.2 kips (117.5 to 170 kN).

The 26.4 kips (117.5 kN) load corresponds to an equivalent load producing the same positive midspan mo-

ment as that of HS20 wheel load with an impact factor of 30 percent, that is:

$$M = [(S + 2)/32](P_{HS20})(width) = P_{eq}L/4$$

where $L = S = 7.5$ ft (2.3 m).

The 38.2 kips (170 kN) load corresponds to a tensile stress of $6\sqrt{f'_c}$ at the bottom fiber of the precast panel. The above loading range was chosen to allow for possible overloading of the deck in the early stages of its life.

At the end of the first stage, the specimen was monotonically loaded until first flexural cracking was observed. In the second stage of repeated loading, the specimen was cycled from 2.4 kips (10.7 kN) (corresponding to all superimposed dead loads) to 28.8 kips (128.2 kN) (corresponding to a tensile stress of $3\sqrt{f'_c}$ at the bottom fiber of the precast panel) keeping the load range equal to 26.4 kips (117.5 kN). This load range is equal to an equivalent HS20 wheel load with an impact factor of 1.3.

Intermediate static loadings were performed at regular intervals in both the uncracked and cracked stages of the specimen. In Specimens 1 through 4, Stage 1 of repeated loading was applied for the first 600,000 cycles and Stage 2 continued thereafter until one million cycles. After one million cycles, all specimens were tested monotonically to failure.

Specimens 5 and 6 were first loaded monotonically to their cracking load prior to the beginning of the cyclic loading. Repeated loading was applied thereafter up to one million cycles using the same loading as in Stage 2 of the first four specimens. This loading sequence was selected to observe the effect of cyclic loading on a cracked panel, which would occur due to overloading of the bridge deck during the early stages of its life. The upper cyclic load level corresponded to a stress limit of $7.5\sqrt{f'_c}$ at the bottom fiber of the precast panel. This addresses the effect of lower prestressing and, hence, higher stress level at the interface on the fatigue life of specimens.

TEST RESULTS AND ANALYSIS

The major experimental results are presented and discussed in this section. Only typical diagrams are shown in order to illustrate the behavior of the specimens. A complete summary of all test results is given in Ref. 7.

Strand Slip Results

The determination of anchorage lengths necessary to reach full development of strands was not the objective of this study. But a qualitative evaluation of the strand development lengths was carried out by monitoring the ends of selected strands for their slip relative to the panel ends. This slippage occurs when the actual bond stresses between the strand and the surrounding concrete exceed the bond strength along some portion of the strand anchorage length. Strand slip load was determined by a slip of 0.01 in. (0.254 mm) in the monitored strands.

Estimated capacities of specimens based on AASHTO Specifications are given in Table 2. The cracking load is calculated using the average modulus of rupture of the concrete. Because the available anchorage length of the strands was less than the required development length based on the AASHTO Specifications for a $\frac{3}{8}$ in. (9.53 mm) strand, the available maximum stress in the strand was calculated based on the available anchorage length according to AASHTO Specifications. Ultimate flexural capacity of the panel included the contributions from the cast-in-place reinforcing bars,¹⁷ wire mesh placed in precast panel, and using the ultimate tensile strength of strands. Horizontal shear capacity was assumed to be $80b_vd$ even though four shear connectors were provided across the interface in some of the specimens.

Similarly, beam shear capacity was

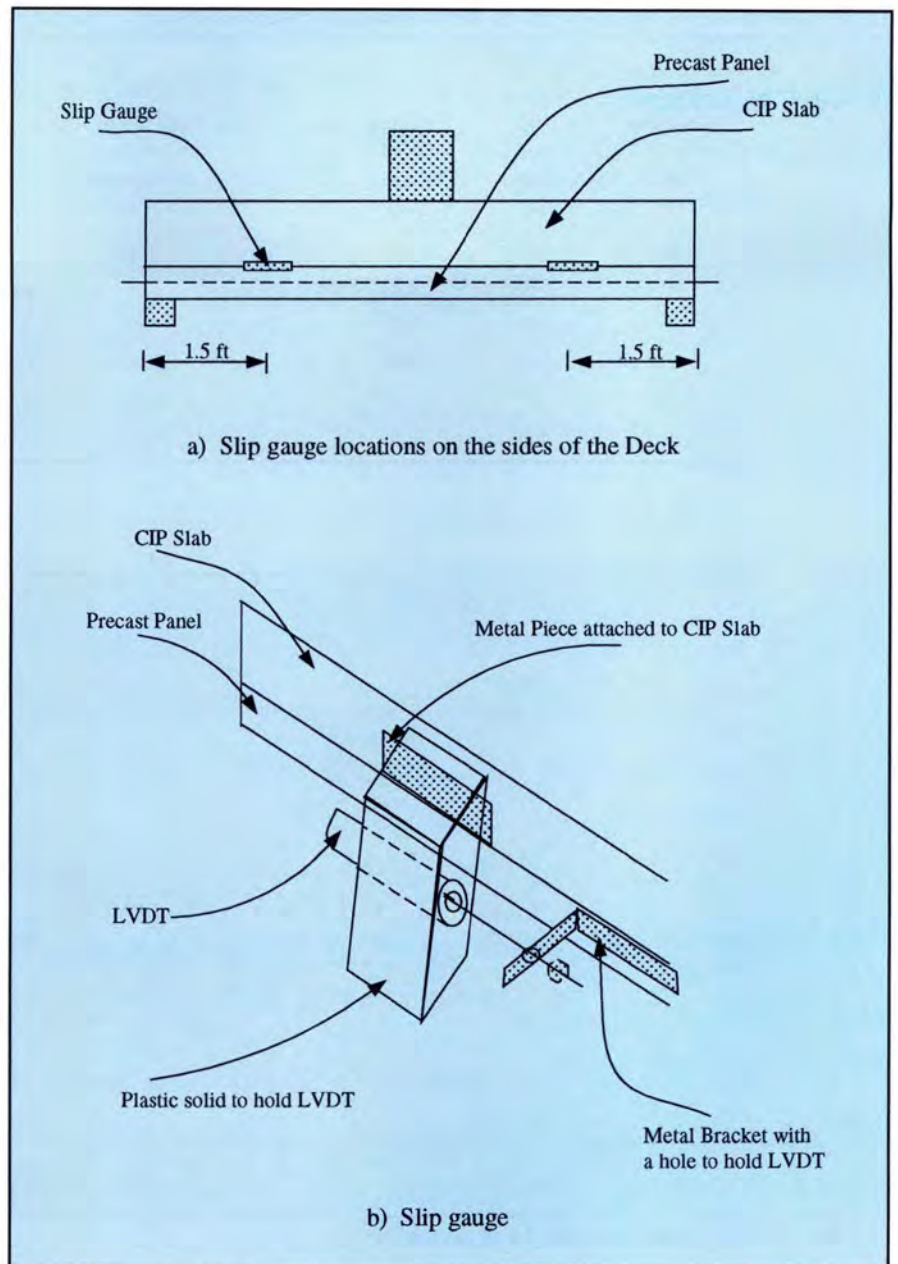


Fig. 9. Slip gauge details.

Table 3. Load capacities of various specimens.

Specimen	Test values			Test/Predicted values				Test loads ratio		Type of failure mode
	Cracking load (kips) (a)	Strand slip load (kips) (b)	Failure load (kips) (c)	Cracking load ratio	Strand slip load ratio	Failure load ratio*	Failure load ratio†	(c)/(b)	(c)/(a)	
1	60	115	160	1.00	1.16	1.62	1.01	1.39	2.67	Shear
2	65	110	170	1.08	1.11	1.72	1.07	1.54	2.62	Shear
3	75	155	200	1.25	1.56	2.02	1.26	1.29	2.67	Shear
4	65	140	180	1.08	1.41	1.82	1.14	1.29	2.77	Shear
5	47.5	95	130	1.18	1.53	2.10	1.01	1.37	2.74	Flexure
6	50	100	140	1.25	1.61	2.26	1.09	1.40	2.80	Flexure

Note: 1 kip = 4.46 kN.

* Taking strand slip load as ultimate load.

† Taking failure mode capacity.

Table 4. Actual ultimate capacities vs. service load and factored load.

Specimen	Test values			(a)	(b) divided by equivalent service load (26.4 kips)	(c)	(a)	(b) divided by equivalent factored load (57.3 kips)	(c)
	Cracking load (kips) (a)	Strand slip load (kips) (b)	Failure load (kips) (c)						
1	60	115	160	2.27	4.36	6.06	1.05	2.00	2.79
2	65	110	170	2.46	4.17	6.44	1.13	1.92	2.97
3	75	155	200	2.84	5.87	7.57	1.31	2.70	3.49
4	65	140	180	2.46	5.30	6.82	1.13	2.44	3.14
5	47.5	95	130	1.80	3.60	4.92	0.83	1.66	2.27
6	50	100	140	1.89	3.79	5.30	0.87	1.75	2.44

Note: 1 kip = 4.46 kN.

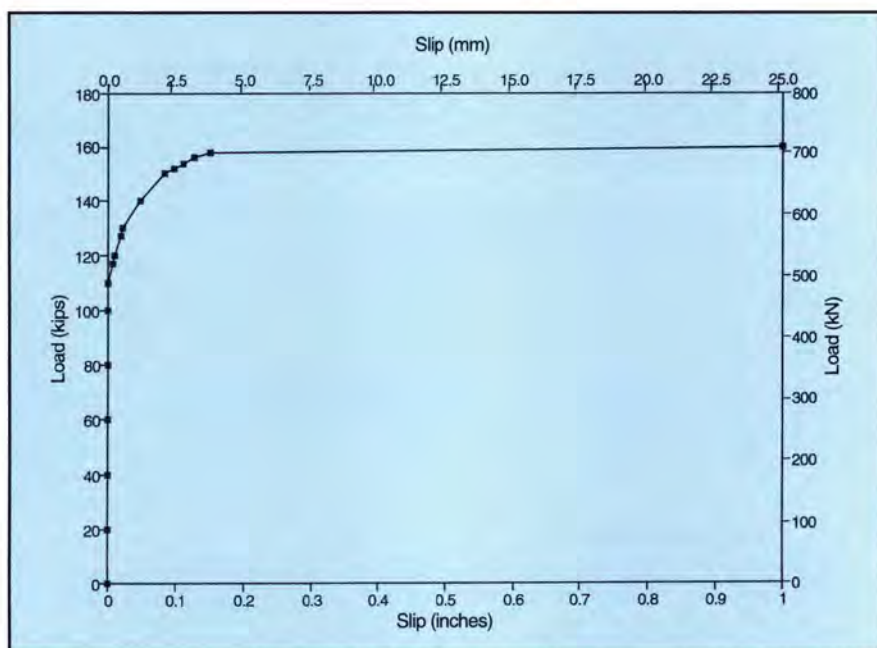


Fig. 10. Load vs. strand slip at #S13 of Specimen 1.

calculated based on a nominal shear stress of $2\sqrt{f'_c}$ over the cross section. In all the specimens, the dead load effects were subtracted from the predicted capacities to represent the amount of live load that can be taken by the specimen prior to failure. Observed load capacity of all the specimens and the respective ratio of test to predicted values based on AASHTO¹⁴ and ACI¹⁸ are listed in Table 3. A comparison of test capacities of all specimens with estimated service and factored loads is given in Table 4.

After one million cycles of repeated loading, the strands did not slip by a significant amount in any of the specimens tested. After the repeated load-

ing phase, the panels were subjected to monotonic loading. Strand slip readings were also monitored during the final monotonic loading phase. Slips did not occur at the flexural cracking load or at the factored loads. The slippage that occurred in various strands had different characteristics. In some cases, there was a gradual slip at loads below the failure load (see Fig. 10), whereas in other cases slip occurred suddenly at the failure load. From dial and strain gauge readings, the side on which failure occurred is easily detected. There was a considerable amount of strength remaining in the slab beyond the first strand slip (see Table 3).

Composite Behavior of Deck Slabs

For composite action, transfer of horizontal shear stress across the interface without significant interface slip must take place. Hence, the interface slip is a good indicator of the composite deck behavior under the applied loading.

Interface slip was not observed until the loads approached failure levels in the first four specimens. In Specimens 5 and 6, significant interface slip was not observed even at failure levels. In the first four specimens, at the ultimate loads, interface slip occurred in the critical region mentioned earlier. This typical behavior is illustrated in Fig. 11 for Specimen 1.

Deflection Characteristics

During the cyclic loading phase, intermediate static tests were performed at regular intervals. The behavior during these intermediate static tests was used to judge the effect of repeated loading on the panel performance. A typical load-deflection behavior during the intermediate tests for Specimen 1 is shown in Fig. 12. Clearly, cyclic loading had a negligible effect on the stiffness of Specimen 1 in the first one million cycles. The attempt to make cyclic loading more critical by decreasing the number of strands in Specimens 5 and 6 also did not have any appreciable effect on the load-deflection behavior of the specimen during the cyclic loading phase (see Fig. 13). This conclusion is in agreement with Buckner et al.¹⁹

Before the onset of the final monotonic loading phase, cracks that appeared during the previous monotonic cycles disappeared due to the prestressing forces. During the final monotonic loading phase, as the load increased, deflection at midspan also increased linearly with the applied load conforming to the uncracked section stiffness. As the load was further increased and reached a load level close to the specimen cracking load, there was a slight reduction in the slope of the linear behavior due to the reappearance of the cracks formed during the previous monotonic cycles. The subsequent linear behavior of the specimen with further applied load reflected the cracked stiffness of the composite deck slab.

As the load increased, further reduction in the stiffness of the deck slab was observed. Deflection varied with a very flat slope (very large increase in deflection was observed for relatively small increase in load) until the LVDTs were removed due to excessive deflection of the deck slab as it reached its ultimate capacity. Load vs. deflection characteristics of all specimens are shown in Fig. 14 up to the loads corresponding to LVDT removal. The actual failure loads are listed in Table 4. Specimens with shear connectors were stiffer near ultimate loads than specimens without shear connectors or with a lubricated interface. Due to the reduction in the number of strands and, hence, the prestressing in Specimens 5 and 6, there was a decrease in the stiffness and an increase in the ductility (see Fig. 14).

Strand and Surface Concrete Strain

Before casting of the panels, strands were pretensioned to a load of 3 kips (13.35 kN) at which point strain gauges were placed on a number of strands. The strands were next pretensioned to an extra 14.5 kips (64.5 kN) corresponding to a maximum limit $[0.75f_{pu} = 202.5 \text{ ksi} (1396 \text{ MPa}) \text{ plus correction for slip loss}]$. Strain gauges at this point recorded $5800\mu\epsilon$ (microstrains) corresponding to 14.5 kips (64.5 kN). After detensioning of the strands, there was an initial loss in the prestress. The average immediate pre-

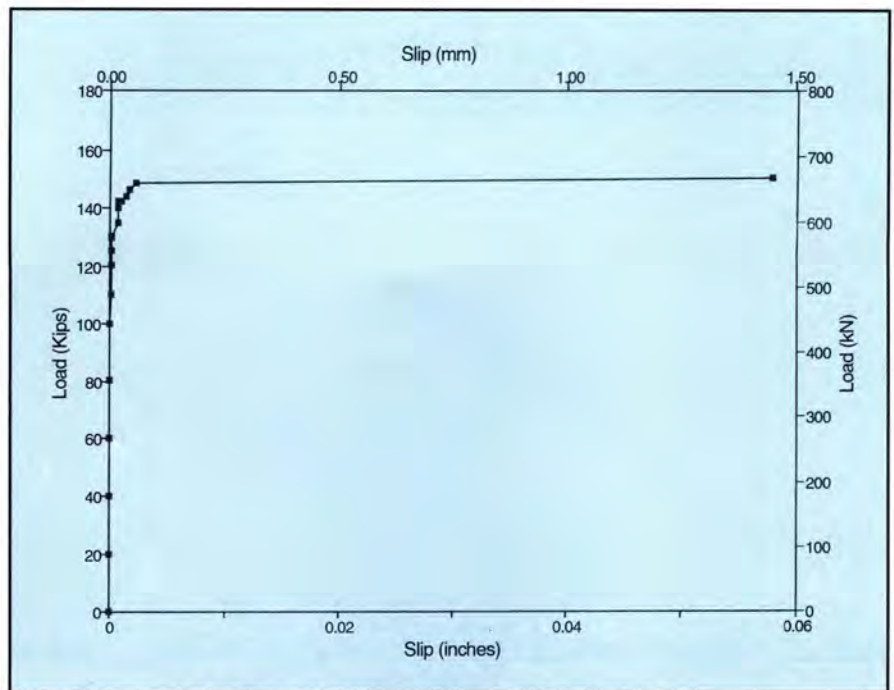


Fig. 11. Load vs. slip at interface (Gauge LVDT4) of Specimen 1.

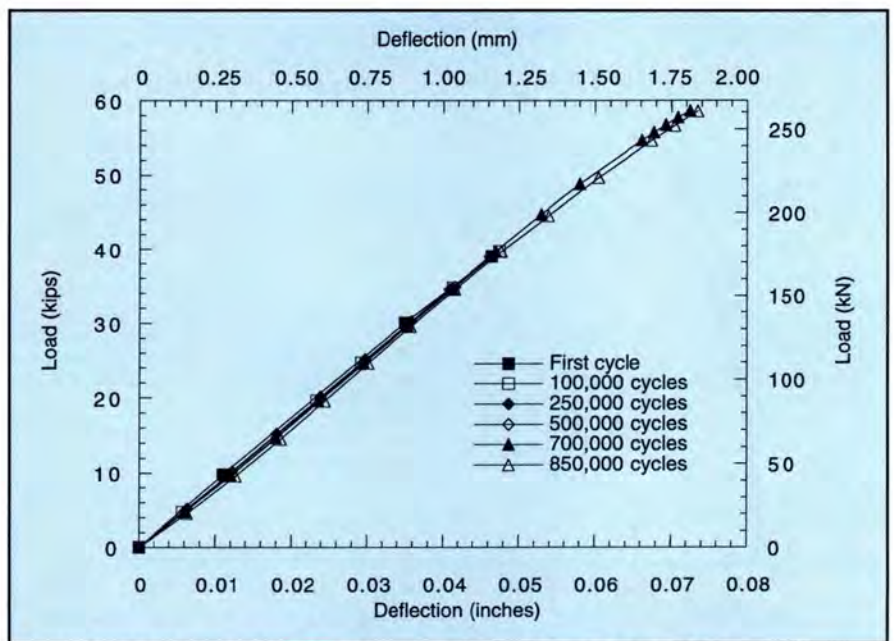


Fig. 12. Effect of cyclic loading on midspan deflection of Specimen 1.

stress loss was about $200\mu\epsilon$. In Fig. 15, this initial strain along with other prestress losses was deducted to show the effect of applied load only.

Strains increased linearly up to the flexural cracking load. In some cases, the first crack appeared near midspan, very close to the estimated value; however, it did not appear over the entire width of the specimen. Consequently, the load vs. average strain plot did not show a significant deviation from the original slope until it reached a slightly

higher load level at which cracks extended through the entire panel width. At this point, there was a noticeable change in the stiffness of the slab and, thus, there was a change in the slope of the load vs. strain plot. As the load increased further, strains varied linearly until the load reached strand slip levels. This also explains the deflection characteristics of the specimens.

In Specimens 5 and 6, the strain recorded at the ultimate load exceeded $12,000\mu\epsilon$. This indicates that the

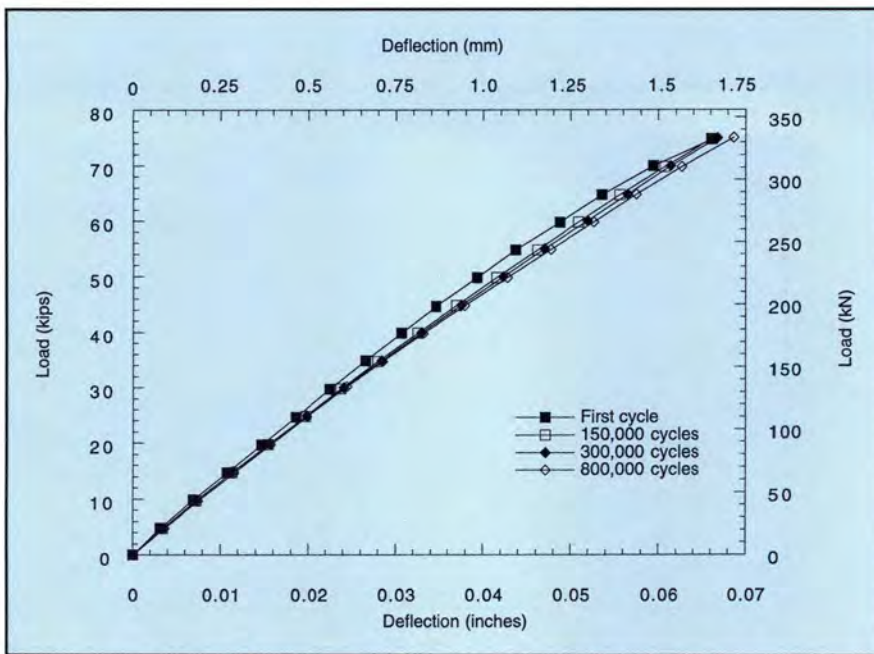


Fig. 13. Effect of cyclic loading on midspan deflection of Specimen 6.

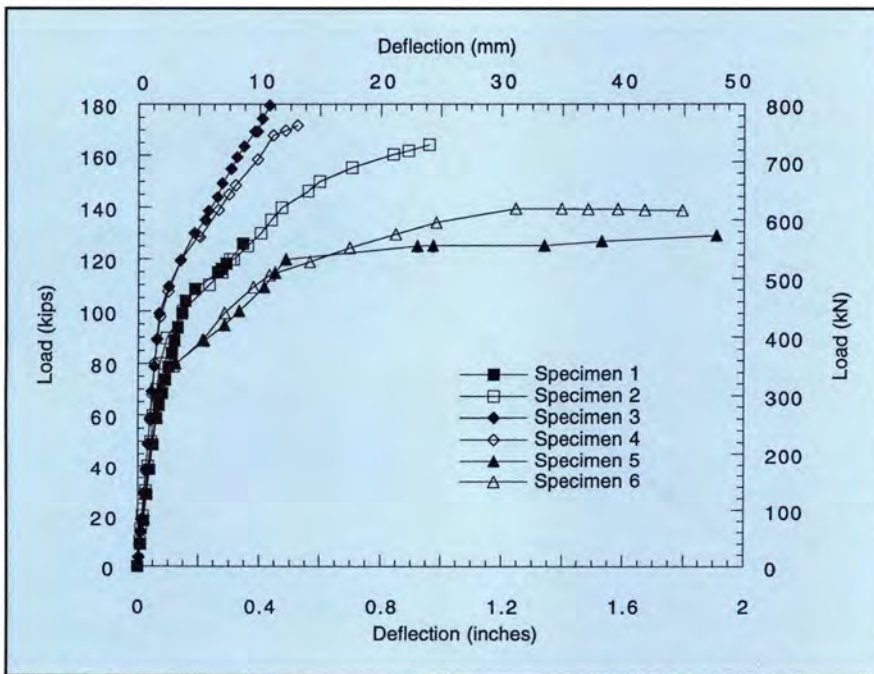


Fig. 14. Comparison of load vs. midspan deflection characteristics during final monotonic load phase.

strands had reached yield and a close look at some of the strands revealed that some of the wires had in fact fractured at failure. This indicates redistribution of load and large deformations. This behavior can be observed in Fig. 15. Gauge C7 corresponds to the midspan location of the seventh strand as shown in Fig. 8. The strain in the strands at which strand slip occurred in all the specimens was almost the

same. But the load levels at which strand slip occurred were different due to the varying number of prestressing strands present in the deck panels.

Strands in the last two deck panels were able to reach such high strains compared to the first four panels due to the formation of wider cracks and larger deflections as a result of reduced prestressing, as compared with the specimens having 18 strands. Due

to these large deflections and crack widths, a plastic hinge formed at the midspan and strands were locally stressed to yield level even though the average stress over the entire length of the strand might be comparatively low. This explains the large strain recorded by Gauge C7 and the fact that this strand, instrumented for slip, recorded no slip up to failure.

Compressive concrete strains at the top of the cast-in-place slab increased linearly up to the flexural cracking load. As the load increased further, cracks appeared in the flexural tension zone of the specimen directly in the section corresponding to the surface strain gauge locations. At this point, there was a sudden increase in strain. As the load was further increased, strain increased non-linearly at a very rapid rate up to failure. In all specimens, the average strain recorded at failure by these gauges on the side of the specimen where failure occurred was close to $3000\mu\epsilon$, indicating that the unconfined concrete was almost crushed.

Behavior of Horizontal Shear Connectors

Specimens 1 and 5 did not contain horizontal shear connectors. All the other specimens contained four shear connectors, a practical minimum number required for handling purposes. Strain in shear connectors did not show significant deviation until specimens reached failure. Before failure, the strain recorded in the shear connectors was about $40\mu\epsilon$. Just at the instant of failure, strain increased suddenly and, in some cases, it reached yield levels. This behavior is illustrated in Fig. 16.

The above observation shows that all specimens behaved as composite members up to failure. Horizontal shear stress across the interface was adequately transferred through the broom finished top surface of the precast panel. Connectors across the interface were not required for the development of flexural capacity but a minimum of four shear connectors usually will be placed for shipping and handling purposes.

Just before failure was imminent, strains in these shear connectors

reached their yield capacity in the specimens with 18 strands. These specimens failed in beam shear, as can be seen in Fig. 17. In Specimens 5 and 6 with 12 strands, there was not a significant change in the strain from its initial value. Specimens 5 and 6 failed in flexure without significant diagonal cracking.

This suggests that specimens designed with flexure as the controlling capacity using a broom finished type surface and the current 80 psi (0.55 MPa)¹⁴ maximum horizontal shear stress will not require horizontal shear reinforcement across the interface.

CONCLUSIONS

The primary objective of this study was to evaluate the performance of thin precast, prestressed concrete panels with a broom finish surface and to determine if horizontal shear-friction reinforcement is needed to ensure adequate composite behavior between the panel and the cast-in-place portion of the bridge deck.

The following conclusions can be stated based on the observations from this study:

1. Slippage of prestressing strands was recorded during the final monotonic cycle to failure. However, an excessive capacity of at least 29 percent at failure above that of the strand slip loads observed and at least 11 percent at strand slip loads above the strand slip loads predicted by current AASHTO procedures were observed in all the specimens.

2. All the specimens showed composite behavior up to failure. Interface slip was observed neither at the equivalent service load of 26.4 kips (117.5 kN) nor at the equivalent factored wheel load of 57.3 kips (255 kN). Interface slip in the first four specimens with 18 strands occurred only at failure loads at least 54 percent above the predicted AASHTO capacity controlled by development length. No interface slips were recorded in Specimens 5 and 6 with 12 strands.

3. Specimens with shear connectors were stiffer near ultimate loads than those without shear connectors (Specimens 1 and 5) but having the same number of strands. Specimens 3 and 4

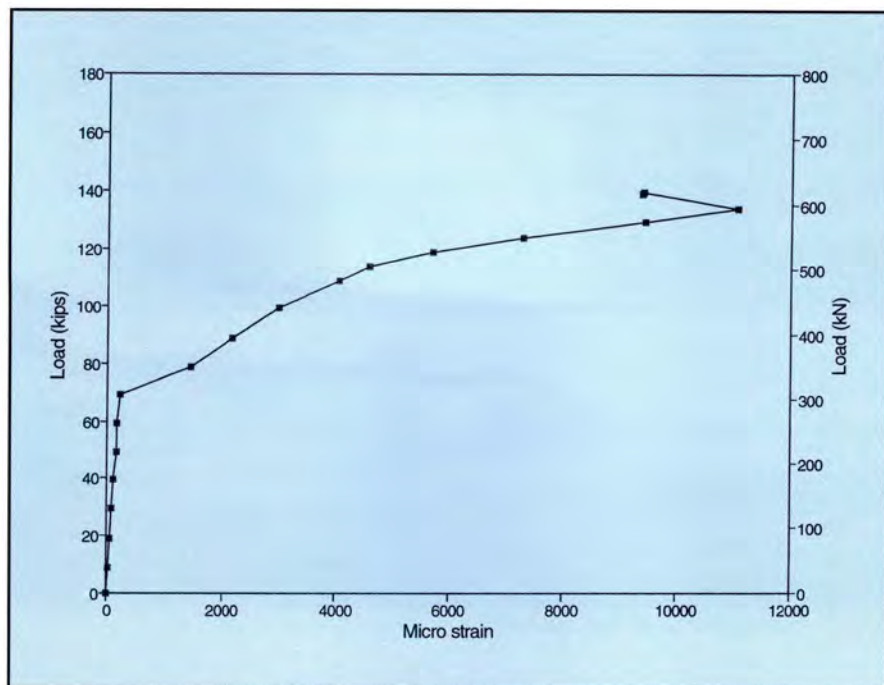


Fig. 15. Load vs. strain in strand at #C7 of Specimen 6.

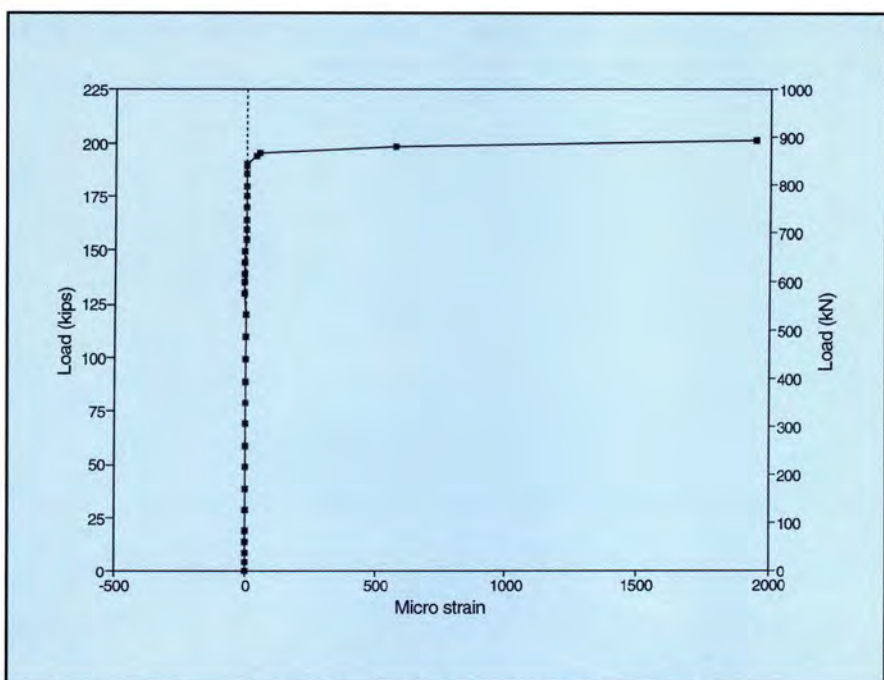


Fig. 16. Load vs. strain in horizontal shear connector (STIR #2) of Specimen 3.

were stiffer than Specimens 1 and 2 due to less prestress loss. Specimens 1 to 4 failed in beam shear whereas Specimens 5 and 6 failed in a ductile flexural mode.

4. Shear connectors were not strained prior to ultimate load. At failure loads in excess of predicted flexural capacity, interface slip occurred suddenly and an increase in strain of shear connectors was observed in

these specimens. No increases in shear connector strains were recorded in Specimens 5 and 6 with 12 strands. This indicates that horizontal shear connectors across the interface are not required but a minimum of four shear connectors can be placed for shipping and handling purposes.

5. The lowest load at which interface slip was recorded in these specimens was 150 kips (668 kN), as seen



Fig. 17. Failure mode of Specimen 3.

in Fig. 11. Based on this minimum load, an upper limit for the average horizontal shear stress of 116 psi (0.8 MPa) is recommended in deck panels with a broom finished surface and

without horizontal shear connectors.

6. The first one million cycles of cyclic loading did not have any appreciable effect on the stiffness of the deck slabs.

RECOMMENDATION

Based on the test results, stay-in-place precast, prestressed deck panels with a broom finished surface do not require horizontal shear connectors if the nominal average horizontal shear stress at the interface is less than 116 psi (0.8 MPa).

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