Performance of Full-Span Panel-Form Bridges Under Repetitive Loading

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An experimental program to determine the effects of repetitive loading on the serviceability and strength of composite panel form bridges is described. Six simply supported bridge decks were tested. The specimens consisted of three precast, pretensioned panels spanning in the direction of traffic and composite with a cast-in-place topping slab. Bond between the topping slab and the roughened interface surfaces of the panels provided the only means of shear connection. Items considered in the study include the topping slab thickness, panel joint type (flat or beveled-edge), and the effect of longitudinal cracks in the topping slab. The specimens were loaded repetitively with 2 million cycles of design load (HS20-44 axle load with allowance for impact). The loading arrangement was such that maximum transverse shear and longitudinal bending stresses were produced during each cycle. Performance was evaluated primarily on the basis of flexural rigidity, differential deflection between panels, and the strength and ductility of the composite system. Several states have constructed bridges by using precast panels as full-span stay-in-place forms. Many of these bridges have developed longitudinal cracks in the topping slab over the panel joints. The study indicates that cracks of this type do not have a detrimental effect on the strength and serviceability of the bridge deck for the expected repetitive loading.

The number of highway bridges in the United States currently in need of replacement has been estimated to be in the tens of thousands. This need, coupled with increasing construction costs, has intensified the search for more economical bridge systems.

One recent development in bridge construction is the use of precast panels as stay-in-place forms for the bridge deck. Most of the applications of these panels have been for short spans in which the panels span transverse to the roadway and are supported by the girders. The panels serve initially to support the weight of a cast-in-place topping slab. After the slab hardens, the panels act compositely with it to resist traffic loads. Adequate performance of this system has been demonstrated in several research programs and by approximately 20 years of use in actual bridges (1).

Tests of short-span precast-form panels for use in highway bridge decks have been conducted in Florida (2), Pennsylvania (3), and Texas (4). The results of these tests have been summarized in a state-of-the-art report by Barker (5). Based on these tests and the performance of panels in actual bridges, design criteria were developed for stay-in-place precast panels (5). These have been incorporated in the latest American Association of State Highway and Transportation Officials (AASHTO) specifications (6).

Based on the satisfactory performance of the short-span stay-in-place panels, at least two states, Florida and Louisiana, have constructed bridges by using precast panels as full-span stay-in-place forms. These bridges are constructed with the panels spanning parallel to traffic and supported either by abutments or pile bents.

Many of the bridges built in this way have developed cracks in the topping slab approximately over the longitudinal joint between panels. These cracks are believed to be initiated by stresses induced by the drying shrinkage of the topping slab. Cracks of this type were detected in bridges built in Louisiana shortly after they were opened to traffic, and similar cracks have been documented for several recently constructed bridges in Florida (7).

Hays, Cox, and Obranic (7) have performed extensive numerical and experimental studies of the static behavior of full-span panel-form bridges. These studies involved both prototype bridges, which had been in service for periods of up to three years, and half-scale laboratory specimens. Both flat and ribbed form panels were considered in the studies.

It was concluded from these studies that the AASHTO effective width criterion for a one-way slab provides a reasonable and conservative estimate of the effective width of the composite panel-form deck. Finite element studies were performed that indicated that transverse bending moments caused tension in the bottom of the cast-in-place topping and predicted better performance from the ribbed than from flat panel decks. The authors recommended a panel that would result in a thickened topping slab with supplementary U-bar reinforcement over the joints between panels.

The formation of cracks in the topping slab so soon after the bridges were built has caused concern among some bridge designers. They perceive a need to verify and refine the design criteria for the full-span panel-form bridges under controlled laboratory conditions. The experimental program described in this paper was performed to fulfill these objectives.

The program involved testing six simply supported composite decks for 2 million cycles of service load and following this with a test to failure. Loads were applied so as to produce maximum transverse shear stresses in the topping slab under an HS20-44 design load.

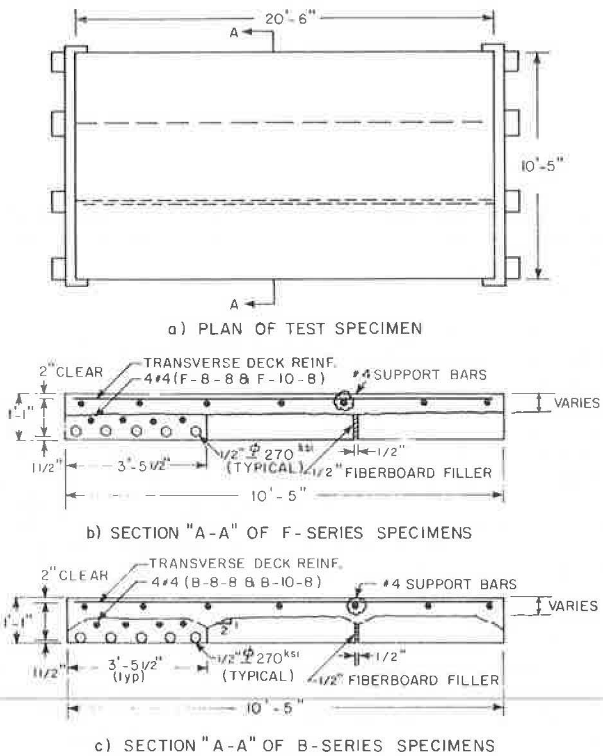
The items considered in the study included the topping slab thickness, panel joint type (flat or beveled), and the effect of longitudinal cracks in the topping slab. Performance was evaluated primarily on the basis of the flexural rigidity of the deck, the differential deflection between adjacent panels, and the strength and ductility of the composite deck. Visible cracks in the concrete, slip of prestressing strands, and strains in transverse steel were also considered in evaluating the specimens.

DESCRIPTION OF SPECIMENS

Six simply supported composite specimens were constructed. Each specimen had an overall thickness of 13 in, an overall width of 125 in, and a span length of 20 ft.

The thicknesses of the precast panels and the cast-in-place topping slab were varied, while a constant overall thickness of 13 in was maintained. One set of two specimens was constructed of panels 5.5 in thick with a complementary topping slab thickness of 7.5 in. The 5.5-in thickness was established as a lower bound for an unshored panel on a 20-ft simple span. Two specimens were constructed of 10-in-thick panels with 3-in topping slabs. The 3-in thickness was selected as a lower bound for the topping slab to allow for a minimum cover of 2 in. A slab of this thickness would probably not be considered a practical minimum when tolerances in precast dimensions and differential cambers are considered. It is believed that the satisfactory behavior of the upper- and lower-bound thicknesses will indicate satisfactory behavior for similarly

Figure 1. Details of test specimens.



designed specimens of intermediate thickness.

Two specimens with 8-in panel thickness and 5-in topping were also tested. These specimens were used to study the effect of longitudinal cracks in the topping slab on the behavior of the composite decks.

For each set of specimens of a particular panel thickness, one was constructed by using flat precast panels and one had beveled edges, as shown in Figure 1. It was thought that the composite decks constructed with beveled panels might perform better than those with flat panels due to improved shear transfer between panels.

Each specimen was constructed of three panels 3 ft, 5.5 in wide. One of the longitudinal joints between panels had a 0.5-in gap filled with fiberboard to minimize shear transfer by friction and to allow more freedom for transverse shrinkage. The other joint was a tight butt joint, which is the usual construction procedure.

Details of the test specimens are shown in Figure 1. Test specimens are identified by a symbol of the form F-n_1-n_2 or B-n_1-n_2. The letters F and B refer to flat and beveled panels, respectively; n_1 is the overall panel thickness and n_2 is the number of 0.5-in-diameter, 270-ksi strands per panel.

Design of Specimens

The test specimens were designed as a one-way slab with an effective width for distribution of wheel loads computed in accordance with the 1977 AASHTO specifications (6). The design live load was the HS20-44 highway loading. The effects of this loading were increased by 30 percent to allow for impact, in accordance with AASHTO.

Complete composite action was assumed between the panels and the cast-in-place slab. Design was based on normal-weight concrete with specified compressive strengths of 5000 psi in the precast and 4200 psi in the cast-in-place topping.

The computed stresses in the specimen at various loading stages are summarized in Table 1. These stresses were computed based on the transformed concrete section but neglecting the transformed steel areas. Sample calculations for these stresses are included in the final report of the study (8).

Manufacture of Precast Panels

The precast concrete panels were manufactured by Biloxi Prestress Concrete Company of Biloxi, Mississippi. The long-line production system was used with all 8-strand panels cast on one line and all 10-strand panels cast on another.

The beveled edges of the B-series panels were rough-shaped by using an appropriate screed and then hand-floated to yield an acceptable shape. At the approximate time of initial set, the top and beveled surfaces of each panel were raked transversely to depths of approximately 0.125 in.

The panels were steam-cured for 12 h, at which time control cylinders indicated a compressive strength in excess of 4000 psi. The panels were stored at the prestress plant until the control cylinders had reached a compressive strength in excess of 5000 psi. The panels were then shipped to Louisiana State University, unloaded, and stored outside until they were moved inside the laboratory for construction of the test specimens.

Construction of Specimens

The precast panels were moved into the laboratory and set over concrete support beams as indicated in Figure 1. After the panels had been placed and aligned, the support beams were shimmed so that the soffits of the panels were bearing accurately at 25 in above datum. The panels were cambered due to the prestress force and therefore were 0.25-0.5 in higher near midspan. Differential camber between adjacent panels was less than 0.125 in in all specimens.

Formwork for the cast-in-place topping slab was then erected. The elevation of the formwork was adjusted by shimming the base so that the top edge was 38 in above datum. The top edge of the formwork supported a steel angle that was used to screed the concrete after it was placed.

Steel reinforcement for the topping slab was placed and supported so that there was 2-in cover from the top of the transverse steel to the top surface. Lifting loops, which were embedded in the top of the precast panels, were burned off to eliminate mechanical shear connection between the precast and cast-in-place concrete. Formwork and reinforcement for a typical specimen are shown in Figure 2.

Approximately 15 min before concrete was to be placed, the top of the panels was saturated with water (water puddled in low spots and in scratch marks on the panel). The panels were then air-blasted until all free water was removed. The surface was still wet when concrete placement began.

Concrete was discharged directly from the truck onto the panels and consolidated by vibration. The top was screeded and finished with a float. The specimen was then covered with polyethylene sheet for curing.

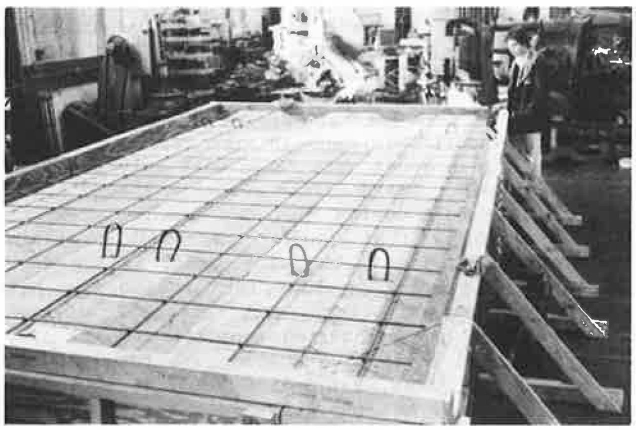
Four of the specimens (B-5.5-10, B-10-8, F-5.5-10, and F-10-8) were cured under plastic for seven days and then exposed to air. None of these specimens developed visible shrinkage cracks on the top surface. The temperature and relative humidity of the air in the vicinity of the specimen were recorded during the curing period by using a hygrothermograph. The mean and range of these values are documented (8).

Table 1. Summary of computed design stresses.

Specimen	At Transfer of Prestress[a]						Service Load (Dead + Live + Impact) at Midspan[b]				Stress Range (Live + Impact) at Midspan (psi)	
	2 ft from Ends			At Midspan				Stress (psi)				
	Prestress Force (kips)	Stress (psi) Bottom	Stress (psi) Top	Prestress Force (kips)	Stress (psi) Bottom	Stress (psi) Top	Prestress Force (kips)	Bottom	Top Panel	Top Slab	Horizontal Shear at Interface	Prestress Strands
F-5.5-10	260	2132	142	261	1712	578	220	-319	1426	695	19	3700
B-5.5-10	259	2121	223	261	1701	701	220	-433	1676	695	19	3700
F-8-8	212	1541	-268	213	1256	28	176	-61	583	695	18	3700
B-8-8	212	1576	-297	213	1291	43	176	-116	750	695	18	3700
F-10-8	213	1377	-351	214	1147	-115	176	40	492	686	13	3700
B-10-8	212	1420	-437	213	1187	-141	176	13	584	686	13	3700

[a] Assuming strands initially stressed to 189 ksi and 24 h of relaxation prior to release.
[b] Assuming total prestress loss of 45 ksi prior to placing topping slab; based on gross transformed area of concrete.

Figure 2. Formwork and reinforcement for topping slab.



It was not practical to cure the specimens until all potential drying shrinkage had occurred. Thus, although there were no visible longitudinal cracks of the type that have been primarily attributed to shrinkage in prototype bridges, there is no assurance that such cracks would not have occurred in the specimens eventually.

As a measure of potential shrinkage, three volume-change prisms, conforming to ASTM C341, were cast for each specimen. These prisms were cured with the specimen and used to measure the unrestrained drying shrinkage that occurred during the curing period. The measurements indicated shrinkage at time of test of about one-half the ultimate value expected for the class of concrete.

Two of the specimens (B-8-8 and F-8-8) were cured under plastic for only 48 h and then exposed to air. The shorter curing time was intended to simulate the relatively poor curing conditions that are likely to occur in real bridges. In each of these specimens a longitudinal crack was induced in the topping slab approximately over the joint that contained the 0.5-in-wide fiberboard-filled gap. This crack was induced by holding down the outside edge of the deck and jacking up on the panel soffit along the joint. This produced a fine flexural crack that was visible in the top surface along the entire length of the deck. A chalk line was snapped on the top surface above the longitudinal joint. The crack meandered across this line several times and deviated from the line by less than 2 in at all points.

Six locations, at approximately 3-ft intervals along the crack, were monitored for crack width growth. Locations were selected where the crack approximately paralleled the chalk line. The width was measured by using a direct reading microscope graduated to 0.01 mm.

Material Properties

The materials used in the test decks were specified to conform to the standard specifications of the Louisiana Department of Transportation and Development (9).

The concrete in the precast panels was specified as air-entrained, normal weight, with minimum cement content of 6.5 sacks/yd³, compressive strength of 5000 psi, and air content in the range of 3-7 percent by volume. The concrete was placed with a slump of approximately 3 in. Twenty-one 6-in-diameter by 12-in cylinders were cast with the panels. Three of these were cured under standard conditions and tested at age 28 days. The remaining cylinders were cured with the panels and tested on the day that the topping slab was cast.

The concrete for the cast-in-place topping slab was air-entrained, of normal weight, with specified cement content of 6.5 sacks/yd³, compressive strength of 4200 psi, and air content 3-7 percent by volume. The concrete was placed with a slump of 3-5 in. Nine 6-in-diameter by 12-in cylinders and three 3x3x11-in volume-change prisms were cast with each pour. Three of the cylinders were cured under standard conditions and tested at age 28 days. Six of the cylinders were cured with the deck. Three of these were tested on the day that repetitive loading began, and three were tested on the day that ultimate loading was performed. The results of the tests for both the panels and the topping slab are summarized in the study final report (8).

The prestressing strand was specified to be 0.5-in-diameter, uncoated, seven-wire strand conforming to ASTM A416 Grade 270K. Mild steel reinforcement was specified as ASTM A615 Grade 60K.

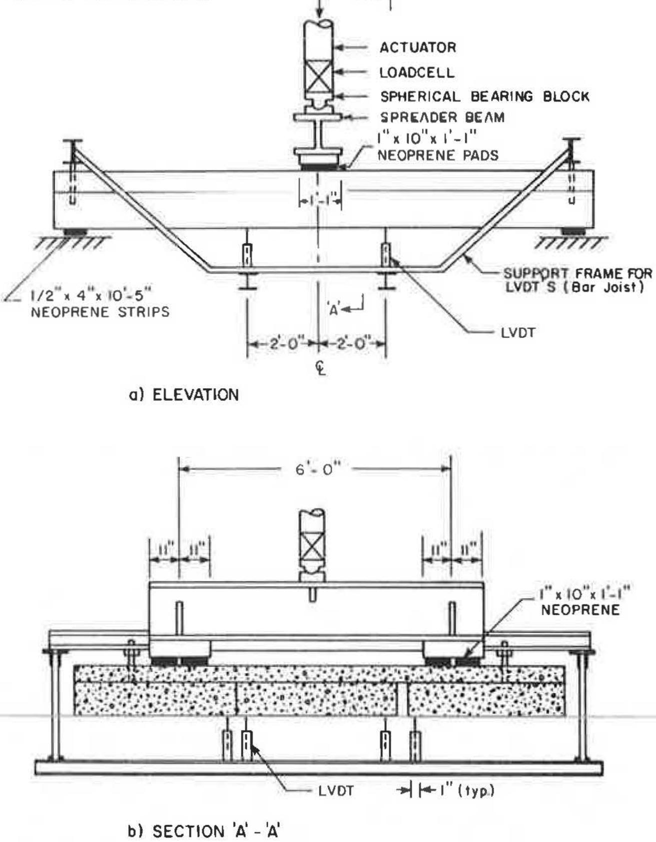
EXPERIMENTAL PROGRAM

The test specimens were loaded 2 million times with a cyclic load to simulate the stresses expected during the life of an actual bridge. The performance criteria used to evaluate the deck were the flexural rigidity of the composite unit, the differential deflection between adjacent precast panels, strains in transverse reinforcing bars, slip of the prestressing strands, visible cracks in the concrete, and the strength and ductility of the composite deck.

Loading Arrangement

In most previous studies that have involved repetitive loads on simple-span bridges, the loads have

Figure 3. Test equipment.



been applied as a series of concentrated loads positioned to approximate the moment envelope. Because a primary consideration in this study is the shear transfer across the joint between panels, it was decided that a better loading arrangement would be a single concentrated load applied at midspan. This arrangement creates maximum shear stress at the longitudinal joint and maximum bending stress at the critical (midspan) section during each cycle.

The load was applied by an actuator controlled by an Instron Series 2150 servohydraulic system. The concentrated load was spread into two "wheel" loads and applied to the slab through 1-in-thick neoprene bearing pads that were sized and positioned to simulate tire prints. The arrangement for the loading is shown in Figure 3.

With the load applied through the spreader beam, approximately one-third of a wheel load is transferred across each longitudinal joint into the middle panel. This yields a reasonable approximation to the maximum shear transfer that would occur in an actual bridge.

Instrumentation

Vertical deflections were measured at transverse sections located 2 ft on each side of the span centerline, as indicated in Figure 3. Four Schaevitz linear variable differential transformers (LVDTs) with a range of ±1 in were positioned at each transverse section. Data from the LVDT units were recorded with the aid of a multichannel data logger.

Strain gages were mounted in a half-bridge on two transverse bars in each cast-in-place topping slab. The active bridge arms were approximately over the longitudinal joints between the precast sections. The bars were approximately above the location where the LVDTs were mounted. Strain readings were obtained with the aid of a Vishay-Ellis switch and balance unit with a digital indicator.

Strand slip was measured by using a caliper with dial gage graduated to 0.001 in. Metal tabs were epoxied to the strands and to the end of the panel approximately 0.5 in above the strand to serve as reference points for these measurements. Two strands on each end of every panel were monitored in this fashion.

Test Procedure

Application of repetitive loads began when the concrete in the topping slab had reached an age of 32 days. The design load, including the allowance for impact, is 41.6 kips. The spreader beam and bearing blocks produced a tare of approximately 1.5 kips. To prevent separation between the actuator and the bearing block, an additional 1.5-kip load was maintained; thus, the repetitive load was varied between 3 and 41.6 kips. The repetitive load was applied at a rate of 500 000 cycles per 48 h (2.89 Hz). This rate was such that dynamic stresses were negligible, and it yielded a convenient stopping time for intermediate static tests that were performed after each 500 000 cycles.

Initial measurement of strand slip and crack width (when applicable) and initial readings of the LVDTs and strain gages were taken with only the tare on the specimen. The specimen was then loaded statically to the full design load, and the measurements were repeated at that load. Intermediate static tests were performed in the same manner as the initial static test except that strand slip was not measured until the final test after 2 million cycles. These tests required approximately 10 min, after which the repetitive loading was resumed. In two of the specimens, F-5.5-10 and B-8-8, equipment breakdowns caused an interruption in the repetitive loading. For specimen F-5.5-10, this interruption occurred after approximately 1.3 million cycles were applied and lasted for two days; for specimen B-8-8, it occurred after approximately 1.8 million cycles and lasted for three days. Otherwise, the repetitive loading was applied continuously except for the brief interruptions for the static tests.

The specimen was then loaded to failure. This loading was applied in increments of 10 kips but was reduced to 5-kip increments near ultimate. The LVDT readings were recorded after each load increment.

TEST RESULTS

The measured structural performance of the composite decks was satisfactory in all six specimens tested. There was no evidence of fatigue in either concrete or reinforcement or of deterioration of composite action, shear transfer strength, or bond during the cyclic loading. Generally, the LVDT readings indicated a slight increase in panel stiffness during the test period. This increase can be attributed to the small increase in modulus of elasticity of the concrete due to cement hydration during this period.

The specimens were loaded to failure after 2 million cycles of design load. Primary failure in every specimen was in flexure by yielding of the reinforcement. Secondary failures were either by crushing of concrete or in a shear mode. The measured loads at secondary failure were in all cases above the computed ultimate load and occurred after the specimen had demonstrated adequate ductility.

The behavior of the test specimens and the analysis of the test data are described below.

Primary Performance Criteria

The performance of the test specimens was evaluated

Table 2. Deflection readings at LVDT locations.

Specimen	Avg Initial Deflection (in)	Avg Deflection/Avg Initial Deflection by No. of Cycles				Maximum Ratio Differential Deflections by No. of Cycles				
		0.5×10^6	1×10^6	1.5×10^6	2×10^6	0	0.5×10^6	1×10^6	1.5×10^6	2×10^6
B-5.5-10	0.0739	0.995	0.980	0.946	0.946	1.08	1.08	1.08	1.08	1.07
F-5.5-10	0.0682	0.970	0.994	N.A.	0.990	1.04	1.03	1.04	N.A.	1.06
B-8-8	0.0926	0.975	0.966	0.966	0.950	1.02	1.02	1.03	1.03	1.03
F-8-8	0.0842	1.000	1.002	1.003	1.017	1.02	1.02	1.02	1.02	1.02
B-10-8	0.0780	0.949	0.933	0.932	0.946	1.02	1.03	1.02	1.03	1.04
F-10-8	0.0746	1.005	0.996	1.006	0.998	1.04	1.08	1.02	1.03	1.04

Table 3. Moment strength and cracking load of composite deck.

Specimen	Age of Precast at Time of Test (days)	Computed Loss of Prestress (ksi)	Moment Strength (kip-ft)			Cracking Load (kips)	
			Computed (M_n)	Experimental (M_u)	M_u/M_n	Computed	Experimental
B-10-8	99	33.3	856	957	1.12	79.2	125.5
B-5.5-10	127	42.1	1012	1213	1.20	54.8	87.5
F-5.5-10	160	43.1	1010	1130	1.12	61.4	118.2
F-10-8	194	35.7	860	886	1.03	81.7	113.5
B-8-8	243	38.2	884	992	1.12	72.7	103.5
F-8-8	242	38.4	886	942	1.06	76.6	110.8

primarily on the basis of flexural rigidity, differential deflection between panels, and moment strength and ductility of the composite deck. The computed flexural rigidities of the composite specimens, based on linear elastic theory for an uncracked section, are 2.06-3.80 times larger than for their cast-in-place topping and precast panels acting noncompositely. Because the deflection of the deck is inversely proportional to its flexural rigidity, the measured deflection is a sensitive indication of deterioration of composite action. The deflections of the deck at locations 2 ft to either side of midspan are tabulated in Table 2. These deflections are essentially the same at the end of 2 million load applications as at the beginning, which indicates that there was no significant loss of composite action.

The differential deflection between adjacent panels is a measure of shear transfer across the longitudinal joint. If there were a differential deflection between panels, then one panel would have to resist a larger proportion of load and hence be subjected to larger bending stress than was assumed in the design.

The differential deflection readings after each stage of cyclic loading are summarized in Table 2, where comparison is made on the basis of the ratio of larger to smaller adjacent deflections. These data indicate a maximum value of this ratio of 1.08. By a simple elastic analysis, if the panels resist an equal share of load when their deflections are equal, when the deflection ratio is 1.08 the share of load resisted by the more severely stressed panel would be increased about 4 percent. This computed increase is not considered significant. Thus, the measured differential deflections indicate satisfactory shear transfer behavior for all specimens under the cyclic loading.

The moment strength and ductility of the deck provide vital measures of endurance under the cyclic loads. To alleviate stress concentrations at the supports, the precast panels were supported at both ends by neoprene bearing pads measuring 0.5 in by 4 in by 10 ft, 5 in. The pads restrain horizontal movement and create a horizontal thrust that was believed to be negligible under service loads but significant at loads near ultimate. To account for this thrust, the shear stiffness of the support pads was determined and the soffit chord extension was measured as the specimen was loaded to failure. These values, together with the measured applied load and deflections, were used to determine the bending moment at the critical midspan section and are given in Table 3.

The moment strength of the deck was computed based on generally accepted assumptions of the strength design method (10). The stress-strain relation for the prestressing steel was furnished by the manufacturer. The compressive strength of the concrete was taken as the average cylinder strength of concrete in the cast-in-place deck at the time of test. The effective stress in the prestressing steel was estimated by the general method recommended by the Prestressed Concrete Institute (PCI) Committee on Prestress Loss (11). The stress in the prestress strands at ultimate was computed by a trial-and-error procedure by using the appropriate strain-compatibility and equilibrium equations.

The computed and experimental moment strengths of all specimens are given in Table 3. In every case, the experimentally determined moment is larger than the computed moment strength.

A typical load-deflection curve for one of the specimens is plotted in Figure 4. For comparison all of the curves are shown superimposed in Figure 5. These curves indicate that primary failure occurred in each specimen in flexure and that the specimens exhibited adequate ductility prior to secondary failure.

Secondary failure of specimens B-10-8, B-5.5-10, F-5.5-10, and B-8-8 occurred by crushing of the concrete at midspan. Secondary failure in specimen F-10-8 was by shear transfer in the 3-in-thick topping slab. Inspection of the topping slab after secondary failure indicated that a vertical crack had formed over the longitudinal joint and a U-shaped diagonal crack around the load point. The cracks in this region indicated both direct shear and diagonal tension failure.

The failure load for F-10-8 was 168 kips, which corresponds to a wheel load of 84 kips--5.25 times the design load of 16 kips. This indicates that an uncracked topping slab of the thinnest feasible size has adequate shear transfer strength for an HS20-44

Figure 4. Typical load-deflection curve for specimen B-8-8.

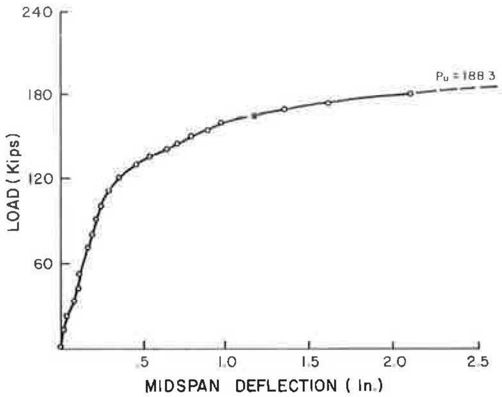


Figure 5. Load-deflection curves for all specimens.

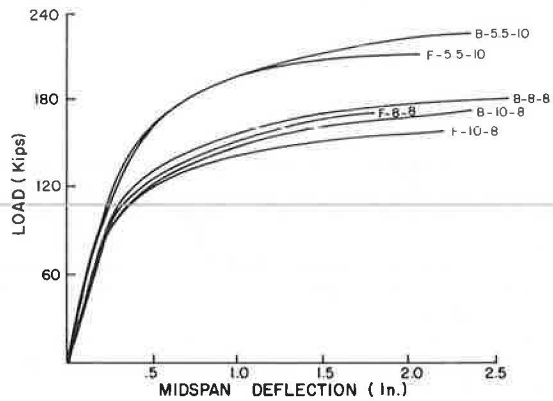
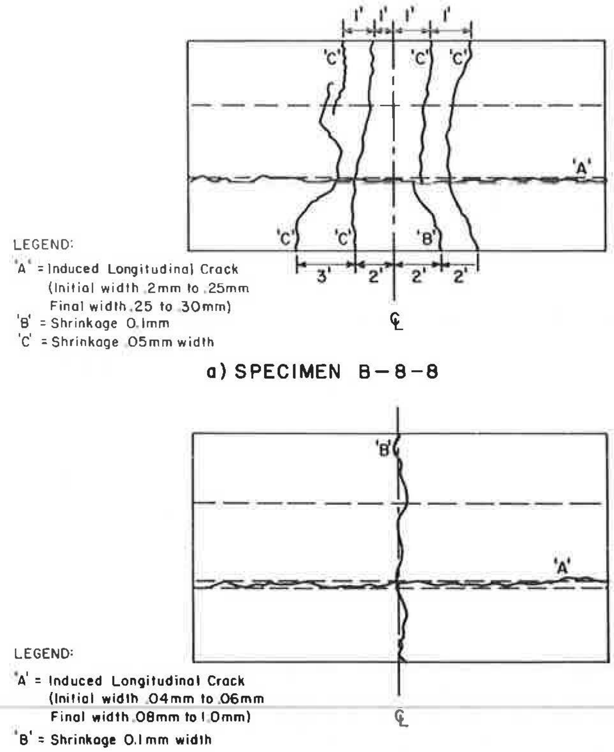


Figure 6. Visible top surface cracks after repetitive loading.



loading. However, it should be noted that a preexisting crack across a shear transfer plane, such as can be caused by shrinkage in an actual deck, has been shown to significantly reduce the shear transfer strength (12).

Specimen F-8-8, which had a 5-in topping slab and an induced longitudinal crack over a panel joint, failed at a measured load of 179.6 kips. This indicates that a 5-in topping slab, when transversely reinforced with pf_y = 200 psi, has adequate shear transfer strength even across a preexisting crack. Secondary failure of this specimen occurred in horizontal shear on the exterior panel of one quadrant.

In a sense, the behavior of specimens B-10-8 and B-8-8 was better than that of the companion specimens F-10-8 and F-8-8 because secondary failures occurred at larger loads and deflections in these specimens. However, since all specimens developed resisting moments in excess of their computed capacities and demonstrated adequate ductility prior to secondary failure, the behavior of all specimens is considered adequate.

Additional Performance Criteria

The flexural cracking load, visible cracks in the specimen, slip of the prestressing strands, and strains in several transverse rebars were recorded for each specimen.

The cracking load is defined as the load at which the first flexural crack was observed in the exterior side of the specimen. This load was in every case more than 40 percent greater than the computed cracking load when based on a modulus of rupture of $7.5\sqrt{f'_c}$ and on the effective prestress and concrete strength at time of test. The discrepancy between computed and observed values can be partly attributed to the simplifications made in computing the cracking moment. Another likely factor is that visual observation was limited to the exterior edges of panels. The first cracks very likely occurred in the interior. The cracking load was not considered an important indication of performance in this test.

Visible cracks provided information that was useful in explaining failure modes and certain aberrations in the load-deflection curves for the specimens. The only visible cracks in the top surface of the topping slabs after 2 million cycles of design load were in specimens B-8-8 and F-8-8. One of the cracks was the induced longitudinal crack that was discussed earlier in this paper. The other cracks are believed to have been caused by shrinkage. The approximate locations of visible cracks are shown in Figure 6.

The longitudinal cracks were measured shortly after they were induced and periodically during the curing and loading periods. In both specimens they were found to widen about 0.05 mm during the curing period, but there was no measurable increase in width during the cyclic load period.

The transverse cracks had a significant effect on the stiffness of the decks in the service load range, as can be seen from the data given in Table 2. The service load deflections in specimens B-8-8 and F-8-8 are approximately one-quarter greater than in the specimens that did not contain transverse cracks. These cracks are believed to be a consequence of the relatively poor curing condition for these specimens.

The transverse crack in specimen F-8-8 is believed to have caused the reduction in rigidity that was measured between the initial and subsequent static tests. All other specimens showed a small increase in rigidity during the cyclic loading, but the rigidity of specimen F-8-8 was found to diminish about 2 percent. The transverse crack in this specimen was approximately across the midspan, and it is believed that the expansion of the neoprene pads under the load points caused this crack to enlarge during the first series of cyclic loads.

Bond and development of the prestressing strands were not thought to be likely problems with the specimens. However, the slip on two strands per panel was measured to verify this performance. There was no indication of strand slip in any of the specimens tested at any stage of loading. The strain gages that were mounted on the transverse rebars did not provide a reliable basis for evaluating the performance of the transverse reinforcement. Several of the gages were apparently damaged during placement or curing of the topping slab and could not be initially balanced. The readings that were obtained varied erratically from one quadrant to another.

The strain measurements did provide some qualitative information on the performance of the decks. The measured strains in the transverse rebars were relatively small for application of the live loads. The strains were in every case less than $75x10^{-6}$ in/in, which corresponds to a stress of about 2 ksi. This indicates that, regardless of the actual stress level caused by shrinkage plus live load effects in the rebars, the stress range due to live load is likely to be so small that fatigue of these bars should not be a problem.

CONCLUSIONS

All of the specimens tested in this program performed satisfactorily for the 2 million cycles of repetitive load. Visible cracks did not develop in any concrete surface, and there was no measurable increase in the width of any preexisting crack during the cyclic loading period. Primary failure was in a ductile flexural mode, and there was no indication of fatigue in the reinforcement.

The only factor that caused a significant variation in behavior among the specimens was the transverse cracking of the topping slab that occurred in two of the specimens. These cracks developed approximately one week after the topping was cast and are attributed to the relatively poor curing conditions for the two. The cracks caused an increase in measured service load deflection of approximately 25 percent compared with the uncracked (better-cured) specimens. These transverse cracks closed as the specimen was loaded, and they did not appear to affect the behavior near ultimate. Whereas cracks in the cast-in-place topping slab did not significantly affect the structural performance of the test decks, such cracks could possibly have an important influence on the long-term durability and serviceability of actual bridges.

The conclusions drawn from this experimental program apply to full-span panel-form composite decks designed by the AASHTO specifications by using the effective width criteria for a one-way slab. Concrete in both the precast panels and the cast-in-place topping slab is of the type commonly classified as normal weight.

Based on the limited number of specimens tested, the following conclusions can be drawn:

1. The composite deck can withstand 2 million cycles of design load without significant loss of serviceability or strength. Adequate composite action is obtained by roughening the interface surfaces of the precast panel and by waterblasting this surface immediately prior to placing the topping slab.
2. Adequate serviceability and strength can be obtained by using flat, precast panels rather than more expensive, beveled-edge panels.
3. There is no indication that the thickness of the topping slab relative to the total thickness affects the fatigue strength of the composite deck up to 2 million cycles.
4. For HS20-44 live loads, adequate shear transfer strength is provided by a 5-in topping slab reinforced transversely with no. 4 grade 60 rebars spaced 12 in on centers. This shear transfer strength is available even when a longitudinal crack exists in the topping slab over the panel.

Specific design recommendations based on this study and on other related studies are given in the study final report (8).

ACKNOWLEDGMENT

The investigation described in this paper was sponsored by the Louisiana Department of Transportation and Development (LaDOTD) in cooperation with the Federal Highway Administration, U.S. Department of Transportation. Technical liaison with LaDOTD was provided by Sheldon Law and J.W. Porter and by Mitchell Smith, who represented the Federal Highway Administration.

The experimental work was carried out in the Structural Research Laboratories of Louisiana State University. We are indebted to Robert Taylor of the University and to student workers John Brogue, William King, and Ray Battalora for their assistance with the construction and testing of the specimens. Contributions to the typing and drafting of figures for the final report were made by Ellen Donner, Susan Sartwell, Meredith Whaley, and Carolyn Sharp.

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Publication of this paper sponsored by Committee on Concrete Bridges.

Full-Depth Modular Precast, Prestressed Bridge Decks

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Precast modular deck construction has been used successfully since 1967. It is still used in a modest but effective fashion, as exemplified by several installations. The details used to connect the panels to the supporting structures, provide composite action, permit vertical adjustment, and develop shear resistance between adjacent panels are critical. A deck protection system to prevent chemical penetration should be incorporated in the design. Construction costs were estimated for four design examples and compared with costs of conventional cast-in-place construction. In each case, the modular system proved to be more economical. Benefits of precast, prestressed decks include greater structural efficiency, reduction in the number of support elements required, less construction time, reduction in interruption to traffic for replacement decks, potential for increasing capacity of existing structures through reduction in dead load, and better quality control.

Current practice in the construction of concrete bridge decks supported by a structural framing system uses cast-in-place reinforced concrete. This is predominantly used for bridge deck replacement and for new bridge construction.

Some of the problems generated by this construction technique have been overcome through the development of new materials and improved procedures, such as concrete overlays, epoxy-coated rebars, and stay-in-place forms. However, others have not. These include the very time-consuming and labor-intensive procedures inherent in the use of cast-in-place concrete and the inefficient use of the materials that occurs when the full advantage of the compressive strength of concrete is not exploited.

One alternative to conventional cast-in-place bridge deck construction that could be more cost efficient is full-depth, precast, prestresssed bridge deck panels. This system is equally adaptable to new construction and to deck replacement projects.

STATE OF THE ART

In 1967, Purdue University, in cooperation with the Indiana State Highway Commission, initiated research to establish design criteria for a full-depth, precast, prestressed deck system (1). This study was followed by an implementation phase consisting of the replacement of decks on two structures: IN-37 over Bean Blossom Creek and IN-140 over Big Blue River. This work was completed in 1970 (2). Subsequently, the deck on a third structure in Indiana (Tonkel Road over Cedar Creek) was replaced with precast elements.

Deck panels for these bridges were cast full width in sections approximately 1.2 m (4 ft) wide. The panels were prestressed in the transverse direction of the bridge and were posttensioned in the longitudinal direction after erection. Composite action between the deck and the supporting members was not developed.

Slabs for the IN-37 bridge were match-cast with a tongue-and-groove joint assembly. Spring clips bolted to concrete inserts were used to anchor the panels to the top flange of the system. Vertical adjustments at the stringer bearing areas were achieved by welding shim plates of variable thickness to the top flange of the structure.

The replacement deck for the IN-140 bridge was constructed in a similar fashion except that the slab had a variable thickness to obtain the desired roadway crown. This was necessary since the steel framing was constructed in a level plane.

The Tonkel Road replacement deck was attached to the beams by using a "Z-clip" in lieu of the spring clip. A fiberglass expansion joint material was placed between the slab and the stringer flange. The adjacent panels were connected to each other on the top surface by a plate welded to inserts cast in the panels. An asphalt wearing surface was placed with variable thickness to provide the required roadway crown.

These structures are performing reasonably well after 12 years of service. Minor problems have developed with concrete spalling at the joints between panels and in connections used to attach the panels to the supporting members.

Since this pioneering effort, a number of agencies have designed replacement decks with precast panels. The New York Thruway has probably constructed more square footage of precast deck than any other agency. These designs did not call for prestressing but rather used mild reinforcing. This was a policy decision based primarily on concern about corrosion of the steel due to the heavy application of salts for snow and ice control.

Precast slabs used by the Thruway Authority were cast with block-outs over the supporting stringers. The slabs were placed on a thick epoxy bed applied to the stringer flange to provide uniform bearing. Studs were welded to the stringer flange through the block-outs, and the void was filled with additional epoxy mortar. This provided a positive connection between the slab and the stringer and also developed some horizontal shear capacity, although the Thruway does not rely on composite action. Keyed transverse joints between adjacent slabs were filled with epoxy. Longitudinal posttensioning was not used. A waterproof membrane and asphalt wearing surface were placed over the completed deck.

In 1979, the Pennsylvania Turnpike Authority replaced the deck on the Clark Summit Bridge near Scranton, Pennsylvania, using precast mild rein-