FLEXURAL BEHAVIOR OF HYBRID PRECAST PRESTRESSED BRIDGE DECK PANELS UNDER STATIC AND FATIGUE LOADINGS – AN EXPERIMENTAL STUDY

Yang Yang, Graduate Student, Department of Civil, Architectural and Environmental Engineering, Missouri S&T, Rolla, MO
Lesley H. Sneed, Ph.D., P.E., Assistant Professor, Department of Civil, Architectural and Environmental Engineering, Missouri S&T, Rolla, MO
Abdeldjelil Belarbi, Ph.D. P.E., Chair and Distinguished Professor, Department of Civil and Environmental Engineering, University of Houston, Houston, TX

ABSTRACT

Partial-depth precast-prestressed concrete deck panels are popular in the US due to their ability to accelerate bridge deck construction as stay-in-place formwork for cast-in-place concrete deck. In this study, twelve 3 in. (75 mm) thick precast-prestressed concrete panels were constructed and tested, including ten hybrid panels with steel interior tendons and either epoxy-coated steel or carbon fiber reinforced polymer (CFRP) edge tendons and two control panels with steel tendons alone. Six of the panels were constructed with conventional concrete, and six were constructed with polypropylene fiber reinforced concrete. The panels were tested under static or constant amplitude fatigue loading. Test results showed that the flexural behavior of the hybrid panels was similar to that of the control panels under service level loads. Hybrid panels exhibited similar cracking load and cracking deflection, but slightly lower maximum loads and maximum deflections than the control panel. After being subjected to two million cycles of fatigue loading, the hybrid panels exhibited gradual softening, and the hybrid panel with CFRP edge tendons also exhibited a slight reduction in maximum load.

Keywords: Bridge decks, CFRP tendons, Epoxy-coated steel strands, Fiber Reinforced Concrete

INTRODUCTION

The use of precast-prestressed concrete (PPC) panels is popular in the construction of concrete bridge decks in the US. Partial-depth panels have been used in a variety of applications in slab-and-girder construction, both for deck replacement and new construction. For composite decks consisting of partial-depth PPC panels and cast-in-place (CIP) concrete topping, the panels accelerate the construction of bridge decks by serving as stay-in-place (SIP) formwork for the CIP concrete deck slab. Traditionally these panels are reinforced with mild steel temperature reinforcement in the traffic direction along with low-relaxation seven wire prestressing steel strands perpendicular to the traffic direction (along the span length of the panel). Panels are designed to remain uncracked under service level loads to maximize serviceability.

It has been observed that some bridges with the partial-depth PPC panel system have experienced rusting of embedded steel reinforcement and prestressing steel strand resulting in concrete spalling along the edges of the deck panels parallel to the prestressing strand[1,2,3]. Since the use of partial-depth PPC deck panels has proven to be a cost-effective practice for concrete bridge deck construction, it is of interest to examine ways to preclude concrete spalling and the resulting service-life reduction associated with corrosion of embedded steel prestressing strand within the panels. Hence, an investigation was initiated to study alternate design options for partial-depth PPC panels utilizing advanced corrosion-resistant materials.

This paper presents the experimental results of an investigation conducted to study the structural performance of partial-depth PPC hybrid panels under static and fatigue loading conditions. The term “hybrid panel” in this paper refers to a panel that has two tendon types: either epoxy-coated steel or carbon fiber reinforced polymer (CFRP) tendons at the panel edges, and uncoated steel tendons at the interior of the panel. The effect of adding polypropylene fibers to the concrete mixture is also explored. Structural performance is evaluated by comparing the load-displacement response of the hybrid panels to the response of the control panel under both static and cyclic or fatigue loading conditions. Stiffness degradation associated with number of cycles and strength reduction is also examined for fatigue loading conditions.

EXPERIMENTAL PROGRAM

The purpose of this experimental study was to investigate the structural behavior of PPC panels constructed with different combinations of alternative materials and compare it with that of the control panel designed using current design specifications by the Missouri Department of Transportation[4]. The control panel contained uncoated steel tendons and concrete without fibers (namely conventional concrete, termed “normal concrete” in this paper). The results of these experiments were intended to provide information including performance at service and ultimate load states, and behavior of the different panel types under static and fatigue loading conditions. Fatigue loading was included to account for the effect of the regular traffic loading on the bridge deck. This section presents and discusses the specimen design and construction, test procedure, and key test results.

SPECIMEN DESIGN AND CONSTRUCTION

The test matrix included twelve full-scale specimens shown in Table 1. A three-term notation system was used to identify the variables of each test specimen. The first term refers to the edge tendon type (note: edge tendons refer to the two tendons at each edge): steel, epoxy-coated steel, or CFRP tendons. The second term indicates the concrete type: concrete without fibers (termed normal concrete) or concrete with polypropylene fibers (fiber reinforced concrete, FRC). The third term designates the loading type: static or fatigue loading.

Table 1 Summary of Material Properties (Note: 1 psi = 6.89x10^{-3} MPa)

Test Specimen	f'_c	Tendon f_y		Tendon f_u		
		Steel tendon	Epoxy coated steel tendon	Steel tendon	Epoxy coated steel tendon	CFRP tendon
	(psi)	(psi)	(psi)	(psi)	(psi)	(psi)
ST-NC-SL	6,360	252,000	-	274,000	-	-
ST-FRC-SL	5,580	252,000	-	274,000	-	-
ECST-NC-SL	5,900	260,000	273,000	279,000	290,000	-
ECST-FRC-SL	6,460	260,000	273,000	279,000	290,000	-
CFRPT-NC-SL	7,000	260,000	-	279,000	-	315,000
CFRPT-FRC-SL	6,390	260,000	-	279,000	-	315,000
ST-NC-FL	6,380	252,000	-	274,000	-	-
ST-FRC-FL	7,710	252,000	-	274,000	-	-
ECST-NC-FL	6,200	260,000	273,000	279,000	290,000	-
ECST-FRC-FL	7,600	260,000	273,000	279,000	290,000	-
CFRPT-NC-FL	6,040	260,000	-	279,000	-	315,000
CFRPT-FRC-FL	6,080	260,000	-	279,000	-	315,000

The control panel specimen (Panel ST-NC-SL) was designed on the basis of the full PPC-CIP composite slab system investigated[4] and was representative of panels in service. The remaining panel specimens were variations of the control panel. Panels were 3 in. (75 mm) thick, 96 in. (2440 mm) long (in the span direction), and 96 in. (2440 mm) wide (perpendicular to the span direction). Each panel contained a total of 21 tendons spaced at 4.5 in. (115 mm) (center-to-center) located at panel midheight. The specified compressive strength of the concrete mixture at 28 days was 6000 psi (41.4 MPa). Steel strand was 3/8 in. (9.5 mm) diameter, 7-wire, Grade 270 low-relaxation conforming to ASTM A416[5]. In the hybrid panel specimens, the two tendons closest to each edge were replaced with either epoxy-coated steel tendons or CFRP tendons. Epoxy-coated steel strand was 3/8 in. (9.5 mm) diameter, 7-wire, Grade 270 low-relaxation grit-impregnated conforming to ASTM A882[6]. CFRP tendons were No. 3 (9.5 mm dia.) reinforcing bar with a tensile modulus of 18,000 ksi (124 GPa). Concrete compressive strength measured at test date and measured yield and ultimate strengths of tendons are summarized in Table 1. Temperature reinforcement consisted of No. 3 (9.5 mm dia.) epoxy-coated steel reinforcing bar oriented perpendicular to the tendons. Fibers were fibrillated micro synthetic polypropylene fiber with a dosage rate of 1.5 lb/yd^3 (0.9 kg/m^3), or 0.1%.

It should be noted that one of the design objectives was to provide panels with the same number of prestressing tendons and the same prestress jacking force to keep the geometry and construction the same. Accordingly, prestress losses were expected to vary for the different tendon types,[7] although prestress losses were not measured.

The panel specimens were fabricated in a precast plant using local materials and standard concrete mixtures so that the test specimens would be representative of the partial-depth PPC panel components used in service by MoDOT. Each tendon was pretensioned to achieve a jacking force of 17.2 kips (76.5 kN). Standard prestressing anchorages were used for the uncoated and epoxy-coated steel tendons. For the epoxy-coated steel tendon, the epoxy was removed before anchoring the tendon. To tension the CFRP tendons, a special system designed by the CFRP tendon manufacturer was used to splice the two CFRP edge tendons to a single 0.6 in. (15.24 mm) steel tendon that was pretensioned using normal procedures.

TEST PROCEDURES

The panel specimens were subjected to a uniformly distributed line load on simply-supported boundary conditions as shown in Figure 1 to investigate the flexural capacity. Supports were comprised of 6 in. (150 mm) wide continuous steel plates that were supported on continuous steel rods on the top flanges of W12×96 structural steel beams. Since the predominant internal force in the panel in service loading conditions is bending moment, testing the punching shear capacity was not considered in this study, nor was it observed. It should be noted, however, that punching shear can be critical for SIP PPC bridge deck panels during construction.

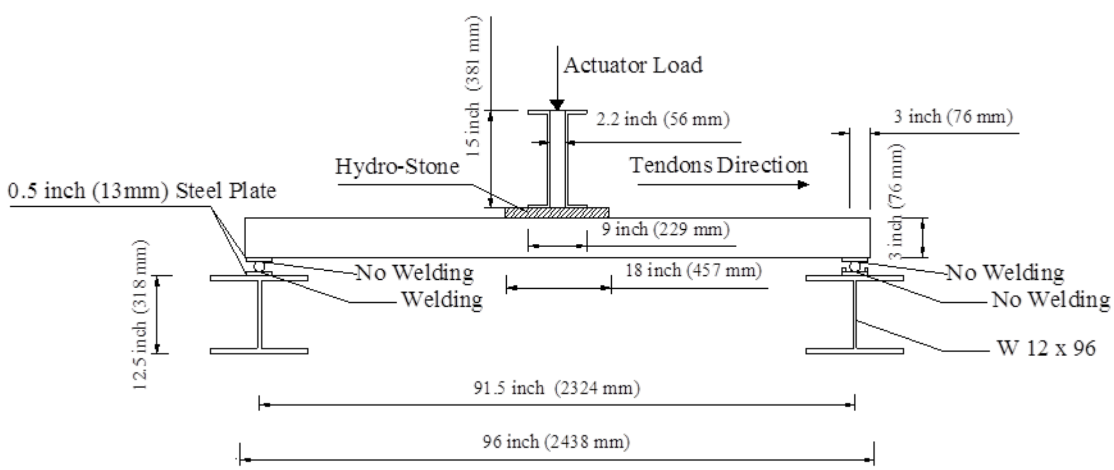


Figure 1 Panel Test Set-Up

The load was applied to the test specimens by a single closed-loop servo hydraulic actuator with a capacity of 110 kips (490 kN) that was suspended from the loading frame located at midspan. A steel spreader beam was attached to the actuator and was used to distribute the load across the panel width. The footprint of the spreader beam was approximately 18 in. (460 mm) wide in the panel span direction, and the spreader beam length was longer than the width of the panel specimen. The spreader beam was secured to the test specimen with a continuous layer of 0.5-1.0 in. (15-25 mm) thick Hydrostone cast in place between the spreader beam and the specimen to increase the uniformity of the loading and to level the spreader beam on the panel surface.

Six panels were subjected to static loading conditions, and six were subjected to fatigue loading conditions. Panels tested under monotonic (static) loading conditions were loaded under displacement control until failure. The loading process was temporarily paused and held constant at key load intervals to observe cracks and take photographs during testing. Panels tested under fatigue loading conditions were subjected to constant amplitude cyclic displacements of the actuator without reversals. A total of two million cycles were applied at a rate of 4 Hz with a range of 0.05 to 0.25 in. (1.25 to 6.25 mm). The target displacement range was determined based on a target applied load range of 2 to 12 kips (9 to 53 kN). The minimum load (and corresponding displacement) selected corresponds to the self-weight of the CIP concrete topping in the composite deck system with no live load. The maximum load (and corresponding displacement) was selected to be 60% of the average maximum load measured during the monotonic static loading tests, which approximates the full service load. The maximum load and corresponding displacement was less than the cracking load and deflection at cracking based on results of the static load tests; thus, macro-cracking was not expected to occur during the fatigue loading. Monotonic tests were conducted initially on each fatigue specimen and after every 500,000 cycles to a displacement of 0.25 in. (6.25 mm). After two million cycles were applied, panels were tested under monotonically increasing displacement to failure using the protocol described previously for the static loading panels.

Each specimen was instrumented with a combination of sensors to measure force, displacement, and strain at key locations. All devices were connected to a data acquisition system controlled by a personal computer. The applied load was measured by a load cell within the hydraulic actuator. Displacement was measured at nine locations on each panel using eight DC voltage transformers (DCVTs) and one linear variable differential transformer (LVDT). Displacement measurements were taken at mid-span and quarter-span of the panel length in the span direction at three different locations with respect to specimen width. Eighteen uniaxial electric resistance strain gages were applied to the prestressing tendons at various locations of each panel. Tendons were tensioned in the prestressing beds prior to the application of strain gages to ensure the bond between strain gages and tendons would remain intact during the fabrication process. Measured strain was used to determine the relative contribution of the edge and interior tendons to the panel resistance as well as to determine whether the steel yielded.

TEST RESULTS

Failure Mode and Cracking Behavior

All six panels subjected to fatigue loading were able to withstand two million cycles without failure. No cracks were observed during the fatigue load cycles or the monotonic test that was conducted at each 500,000 cycle increment. All twelve panels (static and fatigue loading specimens) were tested to failure under monotonically increasing displacements following previous loading cycles (where applicable). All panels failed in flexure by concrete crushing in the compression zone (the top surface of the panel). The typical failure mode is shown in Figure 2. No tendons yielded or ruptured at the maximum load. Cracking load and maximum load and the corresponding displacements for each panel are summarized in Table 2.

Due to the nature of the test setup, the bottom surface of the test specimens was not visible during testing. Thus, cracks were observed and marked on the two side surfaces during testing. The cracking loads reported in Table 2 were determined by the change in load-displacement and load- tendon strain behavior. Cracks were visible on the side surfaces at loads slightly higher than these values. The initial cracks were flexural cracks that appeared beneath the spreader beam location. As the applied displacement (and applied load) was increased, more flexural cracks appeared, and existing cracks propagated upward. At the maximum load, crushing occurred in the compression zone near the edge of the spreader beam location.

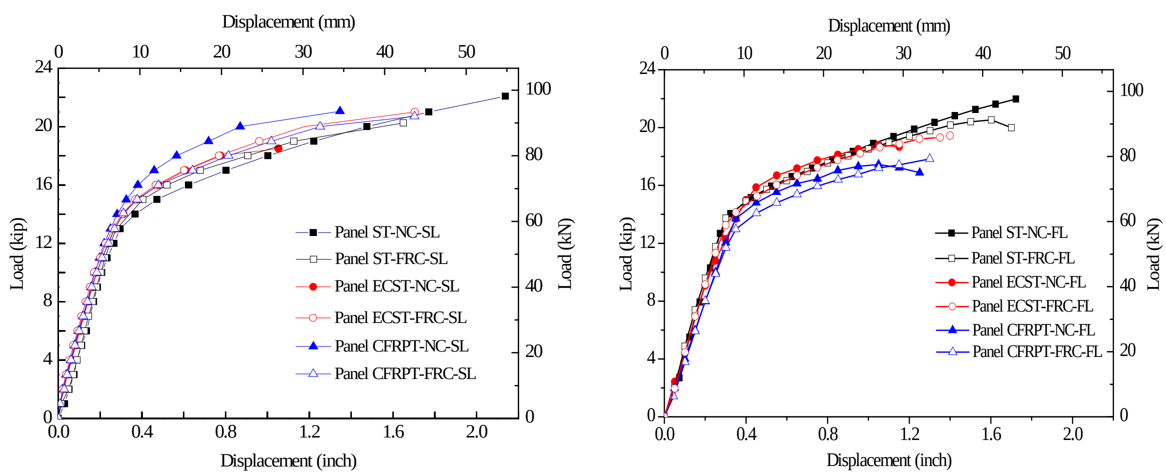
Figure 2 Typical Flexural Failure at Midspan by Concrete Crushing of Simply Supported Panels (Panel ST-NC-SL shown)

Table 2 Summary of Applied Load and Displacement at Cracking and Failure (Note: 1 in. = 25.4 mm, 1 kip = 4.448 kN)

Test Specimen	Cracking Load (kips)	Cracking Deflection at Midspan (in.)	Maximum Load (kips)	Deflection Corresponding to Maximum Load at Midspan (in.)
ST-NC-SL	13.92	0.35	22.08	2.14
ST-FRC-SL	13.79	0.35	20.25	1.65
ECST-NC-SL	14.19	0.35	18.49	1.05
ECST-FRC-SL	14.25	0.35	21.10	1.70
CFRPT-NC-SL	14.99	0.35	21.05	1.35
CFRPT-FRC-SL	14.06	0.35	20.70	1.70
ST-NC-FL	13.98	0.32	22.10	1.76
ST-FRC-FL	14.00	0.33	20.70	1.63
ECST-NC-FL	15.50	0.42	19.00	1.12
ECST-FRC-FL	13.25	0.30	19.49	1.40
CFRPT-NC-FL	13.70	0.35	17.49	1.02
CFRPT-FRC-FL	13.00	0.35	17.95	1.33

Load-Displacement Behavior

Figure 3 compares the applied load-displacement relationship at the center (midspan and midwidth) of each panel during the monotonic test to failure. An approximately bilinear load-displacement relationship was observed for all panels as expected in prestressed concrete members. All panels were expected to have similar flexural stiffness before cracking because they had the same geometry and similar compressive (and tensile) strengths of concrete. Compressive strength of concrete measured at test date is reported in Table 1. For the hybrid panels with CFRP edge tendons, it should be noted that the tensile modulus of the CFRP tendons is lower than that of the steel strand [28,500 ksi [196 GPa)]; however, the tensile modulus of the tendons has much less of an effect on the cross section moment of inertia before concrete cracking than after concrete cracking. Further, since only 4 edge strands of 21 total strands were replaced with CFRP, the effect on the moment of inertia is small before cracking. Figure 3 shows that the flexural stiffness of all panels is approximately the same until cracking; however, panels exhibited different stiffnesses after cracking. Midspan displacements measured at cracking and maximum loads are summarized in Table 2. Deflection measured at different locations along the width of the panel at each load level were similar (within 4-10%), which confirms that the load applied by the actuator was uniformly distributed along width of the panel by the spreader beam.



a. Static loading panels

b. Fatigue loading panels

Figure 3 Applied Load-displacement Relationships at Midspan – Final Test to Failure

The effect of the different tendon types on panel behavior is examined by comparing the load-displacement behavior of panels having the same type of concrete (normal concrete or FRC). For the panels subjected to monotonic loading only (Figure 3a), all panels exhibited a decrease in stiffness in the post-cracking load phase. Comparing the behavior of panels with normal concrete (indicated by solid markers in the figure), differences were observed in terms of post-cracking stiffness, maximum load, and displacement ductility (defined here as the ratio of the displacement at ultimate to the deflection at cracking). Panel ST-NC-SL had a nearly constant post-cracking stiffness until the maximum load was achieved, while Panels ECST-NC-SL and CFRPT-NC-SL showed gradual softening until the maximum load was achieved. Of the panels with normal concrete, Panel ST-NC-SL had the greatest displacement ductility, while Panel ECST-NC-SL had the lowest displacement ductility. For panels with FRC (indicated by hollow markers in the figure), however, the post-cracking stiffness, peak load, and displacement ductility were approximately the same. This may be attributed to the addition of fibers that restrains deep propagation of cracks, as well as the increased ultimate compressive strain and improved and softened post-peak behavior of FRC[8,9].

All six panels that had been previously subjected to fatigue loading exhibited a decrease in stiffness with applied load greater than the cracking load (Figure 3b). Panels with steel tendons only had the greatest maximum load and displacement ductility, while hybrid panels had lower maximum load and displacement ductility. This may be due to the decreased bond strength between concrete and epoxy-coated or CFRP tendons because of micro cracks induced by the fatigue loading. Panels with CFRP edge tendons had the lowest maximum load and displacement ductility of all panels.

To study the effect of the presence of fibers (panels with normal concrete or FRC) on panel behavior, the load-displacement behavior of panels with the same edge tendon types (steel, epoxy-coated steel, or CFRP) is compared. For the panels subjected to monotonic loading only (Figure 3a), the pre-cracking and post-cracking stiffness was similar for panels with the same tendon types, irrespective of concrete type. Panels with FRC had larger displacement ductility than panels with normal concrete for those panels with hybrid tendons. For panels that had been previously subjected to fatigue loading (Figure 3b), the response of panels with the same tendons and different concrete types was similar in terms of pre-cracking and post-cracking stiffness, maximum load, and displacement ductility.

Stiffness Degradation and Strength Decay

The effect of fatigue loading was evaluated in terms of the degradation in flexural stiffness (softening) with increasing numbers of cycles and the reduction in maximum load compared to the comparable panel with no cycles. Stiffness determined after each 500,000 cycles is summarized in Table 3 for each panel. Stiffness values reported in the table were determined using linear regression of the load-displacement relationships measured during the monotonic test conducted after the associated number of cycles.

Table 3 Panel Stiffness and Reduction in Maximum Load Measured after Different Cycles

Test Specimen	**0 Cycles**	**500,000 Cycles**	**1,000,000 Cycles**	**1,500,000 Cycles**	**2,000,000 Cycles**	
	Stiffness (kips/in.)	**Stiffness (kips/in.)**	**Stiffness (kips/in.)**	**Stiffness (kips/in.)**	**Stiffness (kips/in.)**	**Reduction in Maximum Load (%)**
ST-NC-FL	(Note 1)	(Note 1)	52.4	51.4	50.0	0
ST-FRC-FL	51.3	49.8	49.6	48.7	47.3	0
ECST-NC-FL	59.8	50.9	50.0	49.7	47.7	0
ECST-FRC-FL	49.2	48.0	46.3	46.4	45.7	8
CFRPT-NC-FL	51.3	48.0	46.9	46.2	42.8	17
CFRPT-FRC-FL	42.2	41.6	43.4	42.8	41.0	13

Note 1. Measurement unavailable

Figure 4 shows the stiffness degradation of each test specimen after each 500,000 cycle increment. Values on the vertical axis correspond to the percent difference in stiffness relative to the stiffness determined at zero cycles with the exception of panel ST-NC-FL, which was conducted at one million cycles. In general, all specimens show a decreasing trend in the stiffness with increasing number of cycles except panel CFRPT-FRC-FL. This phenomenon can be attributed to the accumulated damage due to cyclic loading, which could be due to microcracking that was not visually observed, bond-slip, or a combination of both. Gradual softening was also observed by Dolan et al.[7] and Saiedi et al.[10] in CFRP-prestressed beams subjected to fatigue loading. In general, the stiffness of panels with FRC degraded less than panels with normal concrete. The maximum stiffness degradation was approximately 20% after two million cycles.

Reduction in panel maximum load was evaluated by computing the percent difference in maximum load achieved during the monotonic test to failure after two million cycles relative to the maximum load of the corresponding panel with no cycles (i.e., the corresponding static panel). Maximum loads for all panels are reported in Table 2, and the reduction in maximum load is presented in Table 3. Panels with steel tendons did not exhibit a reduction in maximum load (strength) after being subjected to two million cycles. Panels with hybrid tendons, however, did exhibit a reduction in maximum load after two million cycles, which again can attributed to the accumulated damage due to repeated loading. The reduction in maximum load was most significant in the hybrid panels with CFRP tendons (13-17%). Strength reduction of CFRP-prestressed beams subjected to fatigue loading was also reported by Saiedi et al.[10], which the authors attributed to weakening of bond between the CFRP bars and concrete. A study conducted by Dolan et al.[7], however, showed no loss of strength in CFRP-prestressed beams subjected to fatigue loading conditions.

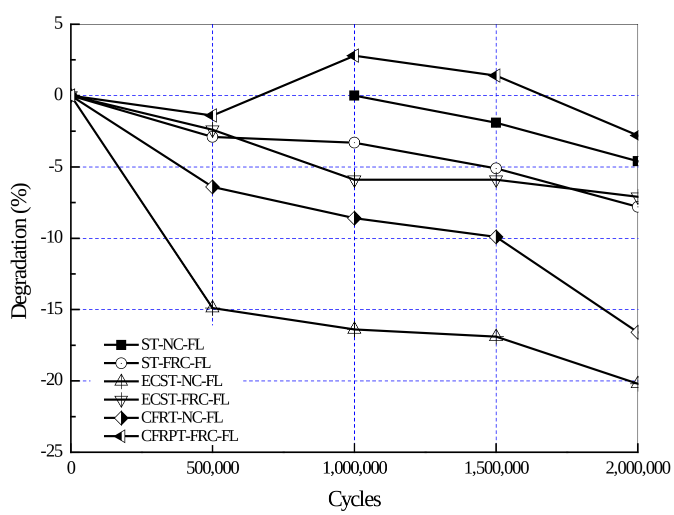


Figure 4 Stiffness degradation with increasing number of cycles

DISCUSSION OF FINDINGS – COMPARISON TO CONTROL PANELS

As discussed previously, the purpose of this experimental study was to investigate the structural behavior of PPC panels constructed with different combinations of alternative materials relative to that of the control panel. The following sections summarize and discuss the general behavior of the hybrid panels and panels with FRC relative to the control panel at service and ultimate load conditions. For the sake of discussion, service load was taken to be 60% of the average maximum load measured during the monotonic tests on the static panels. The authors postulate that differences in tendon prestress loss as well as transfer and development lengths influenced the results, although they were not measured explicitly and require further study.

HYBRID PANELS WITH EPOXY-COATED STEEL EDGE TENDONS AND NORMAL CONCRETE

The structural performance under service load conditions of the hybrid panel with epoxy-coated steel edge tendons tested under monotonic loading only was comparable to the control panel. When tested to failure, the failure mode was the same as that of the control specimen (concrete crushing). The panel with epoxy-coated steel edge tendons, however, showed a marked reduction in maximum load (16%) and maximum displacement (51%).

The fatigue structural performance under service load conditions was comparable to the control panel. Similar to the control panel, the hybrid panel with epoxy-coated steel edge tendons was able to withstand the prescribed two million cycles without failure and without noticeable cracking. When tested to failure under monotonic loading, the failure mode was the same as the control panel (concrete crushing). Hybrid panels with epoxy-coated steel edge tendons showed no significant differences before and after being subjected to two million cycles with respect to cracking load and maximum load, similar to the control panels. The maximum load and maximum displacement of the panel with epoxy-coated steel edge tendons subjected to fatigue loading, however, were 14% and 36% lower than corresponding values of the control panel subjected to fatigue loading.

HYBRID PANELS WITH CFRP EDGE TENDONS AND NORMAL CONCRETE

The structural performance under service load conditions of the hybrid panel with CFRP edge tendons tested under monotonic loading only was comparable to the control panel. When tested to failure, the failure mode was the same as that of the control specimen (concrete crushing). The panels with CFRP edge tendons, however, showed a reduction in maximum load (5%) and maximum displacement (37%) compared to the control panel.

The fatigue structural performance under service load conditions was comparable to the control panel. Similar to the control panel, the hybrid panel with CFRP edge tendons was able to withstand the prescribed two million cycles without failure and without noticeable cracking. When tested to failure under monotonic loading, the failure mode was the same as that of the control specimen (concrete crushing). Panels with CFRP edge tendons showed marked reduction in maximum load after being subjected to two million cycles (17%). A reduction in maximum displacement (22%) was also observed after the panel had been subjected to two million cycles. This phenomenon is attributed to the accumulated damage in the fatigue test specimen, which could be due to microcracking that was not visually observed, bond-slip, or a combination of both. Such reductions were not observed in the control panels. Additionally, the maximum load and maximum displacement of the panel with CFRP edge tendons subjected to fatigue loading were 16% and 37% lower than corresponding values of the control panel subjected to fatigue loading.

PANELS WITH FRC

The static and fatigue performance of panels with FRC and steel tendons only was comparable to the control panel with steel tendons and normal concrete. ACI Committee 544 noted that the addition of polypropylene fibers generally has little effect on the compressive strength of concrete[8]. Because the failure mode was associated with concrete crushing in the compression zone, the addition of fibers did not improve the maximum load relative to the control condition.

The addition of fibers was found to lessen the influence of edge tendon type. All panels with FRC tested under monotonic loading only exhibited similar performance before and after cracking, despite different edge tendon types. In contrast, the post-cracking performance of panels with normal concrete varied with edge tendon type after cracking. As mentioned previously, this may be because fibers restrain deep propagation of cracks, increase the ultimate compressive strain, and soften the post-peak behavior of concrete[8,9]. The addition of fibers also lessened the stiffness degradation after two million cycles and reduction in maximum load for the hybrid panels, especially those with CFRP edge tendons.

CONCLUSIONS

This paper described results of an experimental investigation aimed to study the structural response of hybrid SIP PPC deck panels that included multiple tendon types, with and without the addition of polypropylene fibers to the concrete mixture. Panels were subjected to static and fatigue loading conditions. Service and ultimate states were explored. Based on the results of this study, the following conclusions are made:

1. Hybrid panels with epoxy-coated steel or CFRP edge tendons performed similar to the control panel under service load conditions in terms of stiffness and ability to withstand two million cycles of loading without cracking. Hybrid panels exhibited more significant stiffness degradation and reduction in maximum load with respect to the control panel after being subjected to two million cycles.
2. The addition of fibers to the control panel did not improve the service or ultimate performance. The addition of fibers to the hybrid panels, however, improved the static performance in terms of maximum load, post-peak stiffness, and maximum displacement. The addition of fibers also improved the fatigue performance of hybrid panels in terms of stiffness degradation and strength reduction.
3. Differences in tendon prestress losses as well as transfer and development lengths in hybrid panels require further study.

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REFERENCES

1. Wenzlick, J.D., "Inspection of deterioration of precast prestressed panels on bridges in Missouri," Report No. RI05-024B, Missouri Department of Transportation, Jefferson City, MO, 2008.
2. Sneed, L. Belarbi, A., and You, Y-M., "Spalling Solution of Precast-Prestressed Bridge Deck Panels." Report OR11-005, Missouri Department of Transportation, Jefferson City, MO, 2010.
3. Spraggs, K., Sneed, L., Belarbi, A., and Anderson, N., "Field Investigation of Spalling in Partial-Depth Precast Concrete Bridge Decks Using NDT." *PCI Journal*, V. 57, No. 2, 2012.
4. Missouri Department of Transportation (MoDOT), "2004 Missouri Standard Specification Book for Highway Construction," Missouri Department of Transportation, Jefferson City, MO, 2004.
5. ASTM A 416, "Standard Specification for Steel Strand, Uncoated Seven-Wire for Prestressed Concrete," ASTM, West Conshohocken, PA, 2010, 5 pp.
6. ASTM A 882, "Standard Specification for Filled Epoxy-Coated Seven-Wire Prestressing Steel Strand," ASTM, West Conshohocken, PA, 2010, 5 pp.
7. Dolan, C. W., Hamilton, H. R., Bakis, C. E., and Nanni, A., "Design Recommendations for Concrete Structures Prestressed with FRP Tendons." Report No. DTFH61-96-C-00019, Federal Highway Administration, Washington, D.C., 2000.
8. ACI Committee 544, "State-of-the-Art Report on Fiber Reinforced Concrete (ACI 544.1R)," American Concrete Institute, Farmington Hills, MI, 1996, 66 pp.
9. Yang, J. M., Min, K. H., Shin, H. O., Yoon, Y. S., "Effect of Steel and Synthetic Fibers on Flexural Behavior of High-strength Concrete Beams Reinforced with FRP Bars," *Composites: Part B*, V. 43, 2012, pp. 1077-1086.
10. Saiedi, R., Fam, A., Green, M. F., "Behavior of CFRP-Prestressed Concrete Beams under High-Cycle Fatigue at Low Temperature," *Journal of Composites for Construction*, ASCE, V. 15, No. 4, 2011, pp. 482-489.