Performance of CFRP posttensioned transverse grouted joints for full-depth precast concrete panels

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- In full-depth precast concrete panel systems for new bridge decks or replacing existing ones, grouted transverse joints between deck panels are among the most vulnerable elements.
- Nine specimens with grouted female-to-female transverse joints were tested monotonically to failure using an equivalent concentrated double-tire truck load.
- Low posttensioning stress provided by carbon-fiber-reinforced polymer rods in the panel joint area increased the initial joint cracking load and changed the failure mode from a bond failure between grout and concrete to a tensile failure of the grout.

The longevity of bridge decks constructed with full-depth precast concrete panels depends on several factors, one of which is the integrity of the grouted transverse joint. This paper examines the behavior of a new female-to-female grouted transverse joint under low post-tensioning stress subjected to a concentrated double-tire truck load. Posttensioning across the transverse joint was performed using carbon-fiber-reinforced polymer (CFRP) rods. A novel system consisting of an anchor and simple mechanical stressing device for CFRP rods was adopted and modified for posttensioning the transverse joint.

The focus of this paper is to examine the effect of low posttensioning of the transverse precast concrete deck joint. A performance evaluation including cracking load, peak load, and deflection capacity was conducted under a concentrated load simulating a double tire from a truck. The effectiveness of the posttensioned CFRP rods, which have a high tensile strength and excellent corrosion resistance, is evaluated.

Previous research

Many states are implementing accelerated bridge construction methods to reduce traffic delays and improve safety. One such method uses full-depth precast concrete panels for bridge decks. Full-depth precast concrete panels improve quality and durability and are used for building new bridge decks or replacing existing ones.[1] The grouted transverse joint between precast concrete panels is a vulnerable element and often affects the life and performance of the bridge.[2,3] The integrity of the grouted transverse joint is important for the longevity of bridge decks constructed with precast concrete panels. One way of improving the integrity of the joint is through posttensioning.

Researchers have studied the influence of posttensioning on the performance of transverse precast concrete deck joints using steel rods and cables.[4-9] Laboratory tests and computer models have focused on the posttensioning stress required to keep the joint in compression under truck tire loads of American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications*,[10] as well as the capacity of the joint under pure flexure and pure shear. Based on HS-20 truck loading, a minimum posttensioning stress of 200 psi (1380 kPa) was recommended for simply supported conditions and positive moment sections at the midspan of continuous bridge decks. A posttensioning stress of 450 psi (3100 kPa) was recommended at the interior support of continuous decks.[11]

Joint capacity prior to initial cracking for posttensioned and nonposttensioned transverse joints was evaluated by many researchers using finite element models and laboratory tests. Different methods to predict the peak load capacity of the joint have also been proposed. The use of principal stress equations and tensile strength of the grout to predict the cracking strength of grouted joints was compared with laboratory static shear tests.[12] The experimental strength of posttensioned concave-to-concave joints, commonly referred to as female-to-female joints, has been compared with American Concrete Institute's (ACI's) shear capacity equations in *Building Code Requirements for Structural Concrete (ACI 318-14) and Commentary (ACI 318R-14)*.[13,14] The effect of posttensioning on the negative moment region of transverse bridge deck joints as well as the bond between the grouted joint material and concrete deck have also been studied through both experiments and finite element models.[15,16] It was recommended that the maximum design tension be limited to $1.5\sqrt{f_c'}$ psi ($0.125\sqrt{f_c'}$ MPa), where f_c' is the lesser of the grout or concrete compressive strength. Finite element modeling to determine the appropriate posttensioning stress to keep the transverse joint in compression after creep and shrinkage losses has been performed.[17] It was recommended that single-span precast concrete decks be posttensioned to a stress of 200 psi (1380 kPa), while precast concrete decks with two or three spans should have higher posttensioning.

Experimental investigation

This study is concerned with the behavior of the transverse joint between precast concrete deck panels under low posttensioning stress at the panel joint area with CFRP composite rods. Such rods have advantages in this application because of their high tensile strength and corrosion resistance. In a related study, the deck of the Beaver Creek Bridge on US Route 6 near Price, Utah, was built in 2009 with full-depth precast concrete panels reinforced with glass-fiber-reinforced polymer (GFRP) bars. Posttensioning of the panels was carried out with 0.6 in. (15 mm) Grade 270 (1860 MPa) low-relaxation steel strands.[18] This motivated the authors to use CFRP rods for posttensioning the panels to develop knowledge for a future steel-free bridge deck constructed with GFRP-reinforced precast concrete panels and posttensioned with CFRP rods. The effect of low posttensioning on initial joint cracking capacity, peak load capacity, and failure mode of a grouted female-to-female transverse joint for precast concrete panels under a simulated double-tire truck load is investigated using posttensioned CFRP rods.

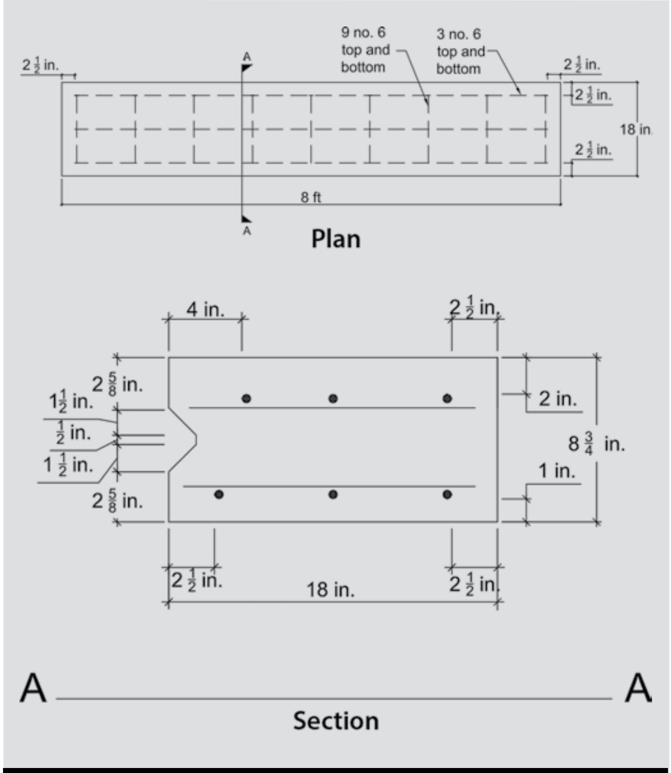


Figure 1. Single panel dimensions and steel bar details. Note: no. 6 = 19M; 1 in. = 25.4 mm; 1 ft = 0.305 m.

The bridge deck of the Beaver Creek Bridge was made with full-depth precast concrete panels. The panels were 44 ft 5 in. × 6 ft 10 in. × 9 1/4 in. (13.4 m × 2.08 m × 0.23 m) and were placed in the transverse direction. The panels were connected through female-to-female grouted transverse joints placed at 6 ft 10 in. (2.08 m). The panels were lightly posttensioned with 11 grouted low-relaxation steel tendons spaced at 3 ft 9 ½ in. (1.16 m) in the span direction. The girders for the 88 ft 2 in. (26.9 m) single-span bridge were AASHTO Type IV prestressed girders placed at 7 ft 7 in. (2.31 m) center to center. The bridge deck panel details of the Beaver Creek Bridge were used to determine test specimen dimensions.

In the present research, single panels were 18 in. (460 m) wide, 8 ft (2.4 m) long, and 8 ¾ in. (0.22 m) thick (**Fig. 1**). Two 18 in. single panels were grouted together at the transverse joint to construct a 3 ft 1 in. (0.94 m) wide specimen (**Fig. 2**). The 8 ft length correlates to the 7 ft 7 in. (2.3 m) girder spacing found at the bridge. Two ⅜ in. (10 mm) diameter CFRP rods spaced 4 ft (1.2 m) apart were used to posttension the grouted joints. The 4 ft spacing between the posttensioning rods corresponds to the 3 ft 9 ½ in. (1.16 m) spacing used at the bridge. A system consisting of an anchor and simple mechanical stressing device for CFRP rods was adopted and modified for posttensioning the transverse joint.[19,20]

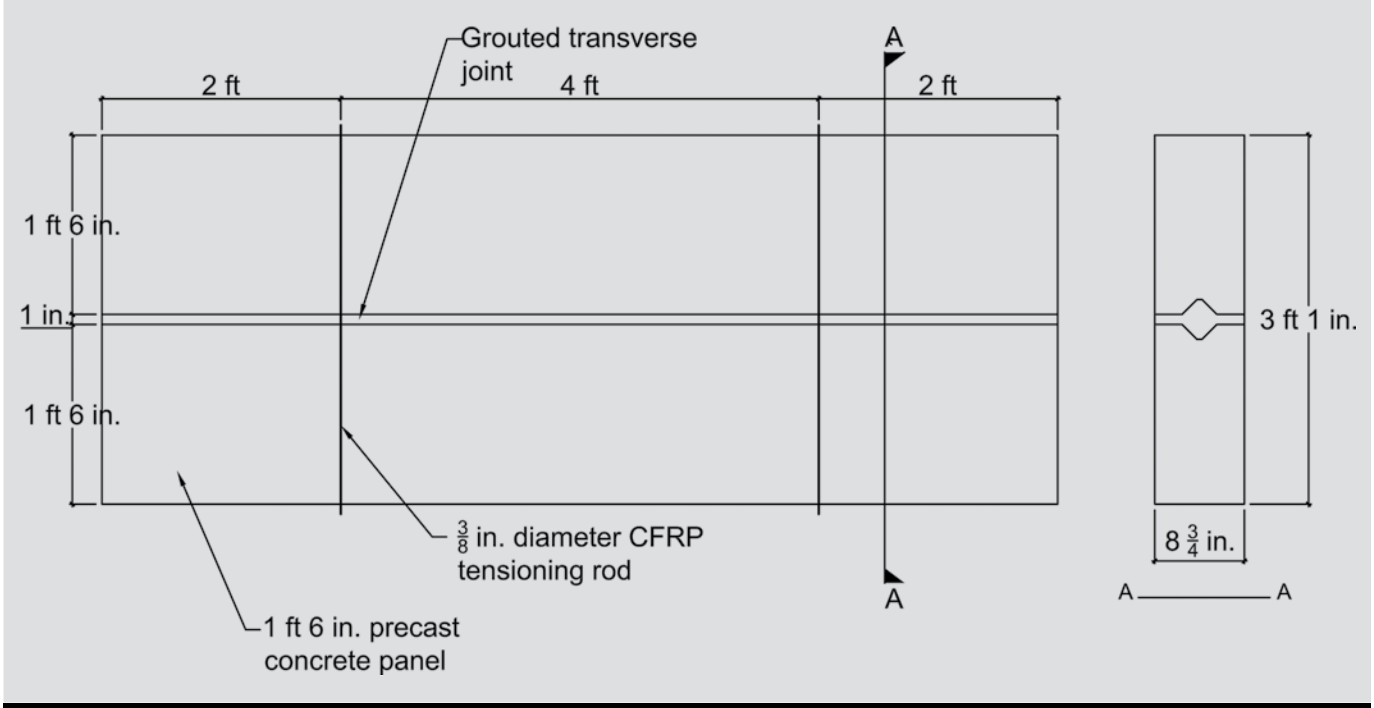


Figure 2. Plan view of jointed panel specimens. Note: CFRP = carbon-fiber-reinforced polymer. 1 in. = 25.4 mm; 1 ft = 0.305 m.

A total of 11 tests were conducted. Nine of the tests were composed of two panels grouted together with the female-to-female grouted joints. The grouted panel tests consisted of three nonposttensioned specimens (0A, 0B, and 0C), three specimens with a posttensioning force equal to 50% of the design tensile capacity of the CFRP rod or 35 psi (240 kPa) stress on the joint area (35A, 35B, and 35C), and three specimens with a posttensioning force equal to 70% of the design tensile capacity of the CFRP rod or 48 psi (330 kPa) stress on the joint area (48A, 48B, and 48C). Two single precast concrete panels (SA and SB) were also tested for comparison. Low posttensioning stresses were used to study the failure modes of the transverse joint.

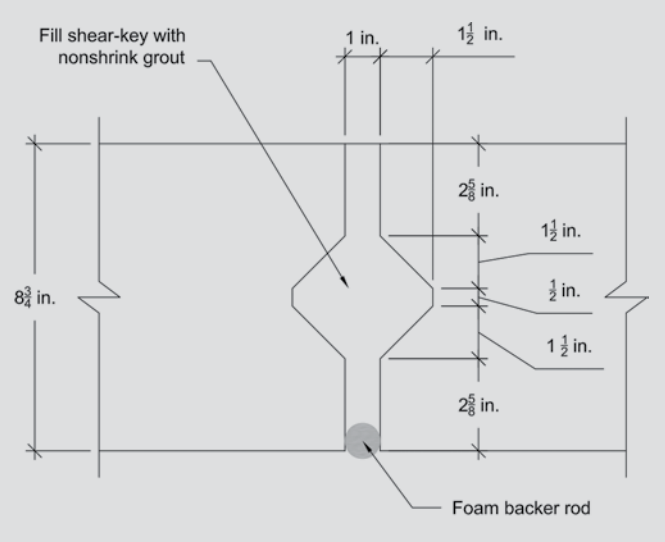


Figure 3. Grouted joint details. Note: 1 in. = 25.4 mm.

Precast concrete specimens and CFRP rods

Precast concrete panels were cast for this research with a 28-day concrete compressive strength of 11,000 psi (76 MPa). The specimens were grouted with the female-to-female joint configuration of **Fig. 3**; this is a new de-

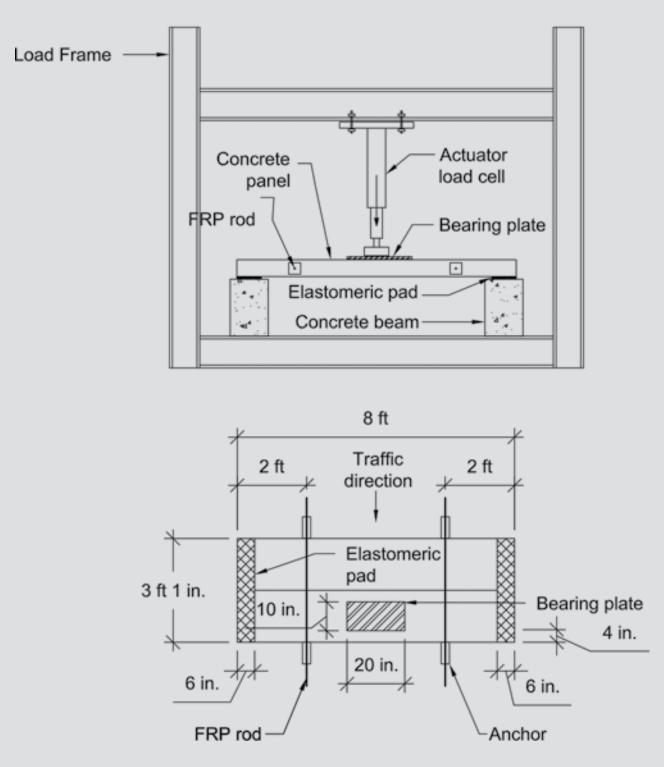
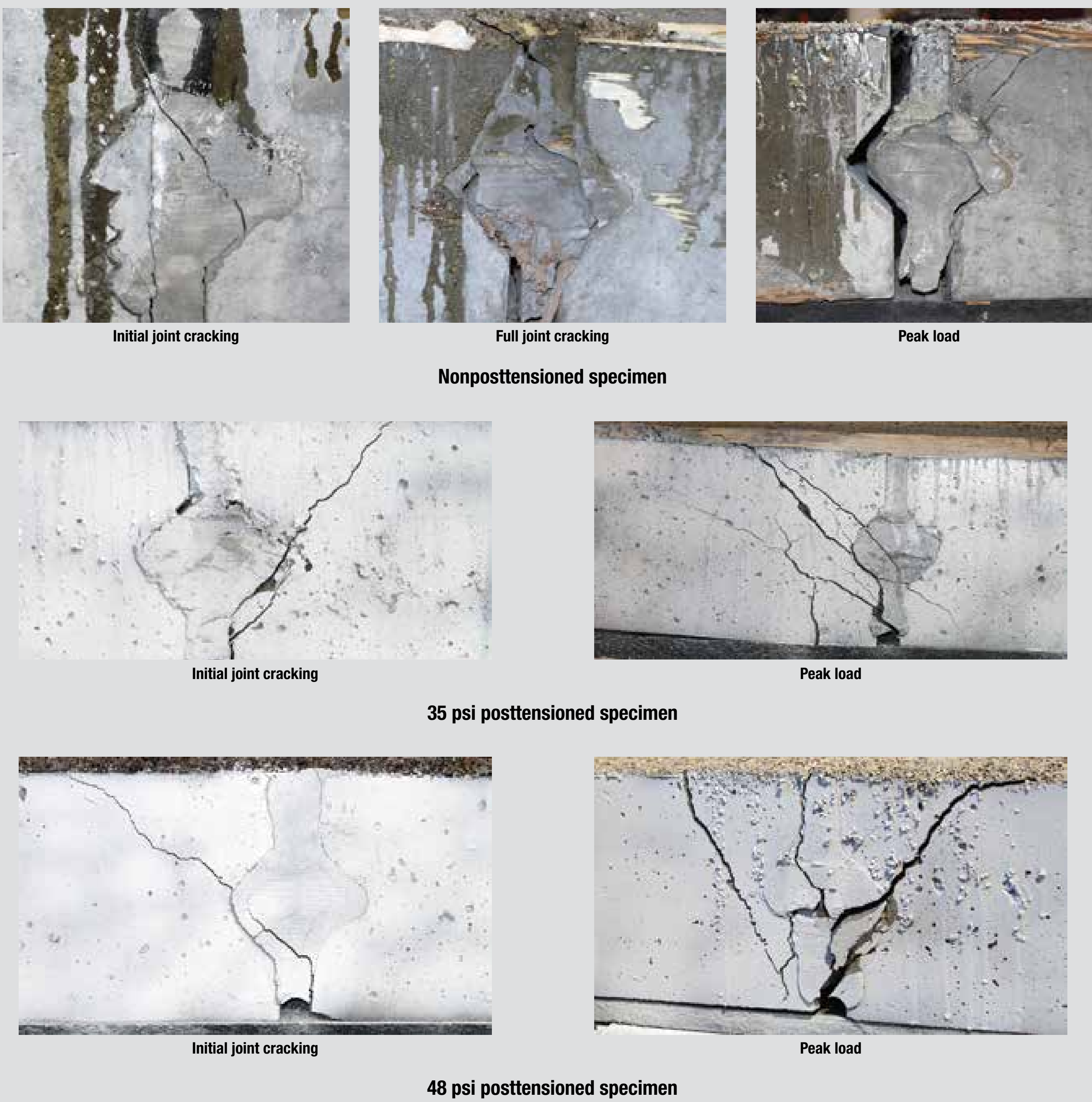


Figure 4. Test setup. Note: FRP = fiber-reinforced polymer. 1 in. = 25.4 mm; 1 ft = 0.305 m.

tail developed in this research for ease of construction. The panels were reinforced with nine no. 6 (19M), Grade 60 (420 MPa) mild steel bars. Six specimens were posttensioned before testing with two 3/8 in. (10 mm) diameter CFRP rods with a modulus of elasticity of 22,500 ksi (155 GPa) and a design tensile strength of 27.5 kip (122 kN) (**Fig. 4**). Posttensioning was applied by tightening a four-bolt plated anchoring system to create tensile strains in the rods.[20] Posttensioning strains were measured using strain gauges attached to the CFRP rods. The grout had an average compressive strength of 4500 psi (31 MPa) with a maximum strength of 5000 psi (34 MPa).

Test setup

A load frame with a 500 kip (2200 kN) hydraulic actuator applied a monotonic displacement at midspan of the simply supported slab, which was supported on elastomeric pads (Fig. 4). The actuator was operated using displacement control. An electronic data acquisition system recorded displacements and strains in the reinforcing bars and CFRP rods. Loading was applied over a 10 × 20 in. (250 × 510 mm) area to simulate a double-tire truck load on a bridge deck, as specified in the AASHTO LRFD specifications. The load on the specimens was applied at the center of one of the panel halves (Fig. 4). The speci-



Initial joint cracking | Full joint cracking | Peak load

Nonposttensioned specimen

Initial joint cracking | Peak load

35 psi posttensioned specimen

Initial joint cracking | Peak load

48 psi posttensioned specimen

Figure 5. Behavior of grouted joint. Note: 1 psi = 6.895 kPa.

mens were posttensioned with ⅜ in. (10 mm) CFRP rods before loading. Figure 4 shows the typical loading condition. The anchor locations for the two posttensioned CFRP rods are also shown. The CFRP anchors and original stressing device[19,20] were modified for this application.

Instrumentation

Linear variable differential transformers (LVDTs) were placed on the bottoms of the panels to measure vertical displacements along the centerline of each specimen half. Three LVDTs per panel were used: one at midspan and two at the quarter points for the loaded and unloaded sides. Strain gauges were attached at the center of the top and bottom longitudinal steel bars adjacent to the grouted transverse joint and on the adjacent transverse bars. Strain gauges were attached to the CFRP rods to measure strain in the rods during stressing and under loading. Three gauges were placed on each rod: two at the quarter points and one at the middle.

Experimental results

All specimens experienced an initial significant crack at the joint and a final crack on the loaded panel. **Figure 5** compares joint behavior for the three specimen types at different stages. The nonposttensioned specimens had two characteristic cracks: an initial joint crack at the bond interface with bond failure between the grout and concrete at the joint and the propagation and opening of the second crack until specimen ultimate failure. The posttensioned specimens had one major crack: an initial diagonal tension crack at the joint during monolithic behavior and the propagation of the initial joint crack until failure. Posttensioning changed the failure mechanism from bond failure to joint shear failure, indicating that the posttensioning stress was sufficient to overcome bond failure between grout and concrete. Bond capacity and the load at which the first crack occurred depended on the posttensioning stress. Ultimate failure was a diagonal tension failure, which occurred on the loaded panel side and was similar for all specimens.

Specimen data analysis

Figure 6 shows the force versus displacement performance of the nine grouted panel specimens and two single panels. The grouted panel specimens had a well-defined initial cracking and peak load. The initial cracking load corresponds to the first significant joint crack with an initial drop in capacity. This is an important benchmark at which chloride intrusion begins and deck integrity is compromised. Initial joint cracking is shown in Fig. 5. At ultimate, the specimens and panels experienced diagonal tension failure of the loaded panel.

Table 1 summarizes the initial cracking and peak load capacity of the nine grouted panel specimens and two single panels. Single panels had an average peak load of 51.5 kip (229 kN). The three nonposttensioned grouted panel specimens (0A, 0B, and 0C) had an initial drop in capacity at displacements of 0.3 to 0.44 in. (8 to 11 mm) corresponding to an initial cracking load of 37.7 to 54.5 kip (168 to 242 kN). The peak load ranged from 60.5 to 64.3 kip (269 to 286 kN). Posttensioned specimens 35A, 35B, and 35C experienced initial cracking at 0.6 to 0.7 in.

Table 1. Tested joint capacity

Specimen	Posttensioning force, kip	Load at initial significant crack, kip	Displacement at initial crack, in.	Peak load, kip
SA	n/a	n/a	n/a	51.0
SB	n/a	n/a	n/a	52.0
0A	0	54.5	0.5	60.5
0B	0	43.7	0.3	61.1
0C	0	37.7	0.3	64.3
35A	27.2	69.8	0.7	79.3
35B	28.0	72.0	0.6	89.3
35C	29.1	73.0	0.7	81.4
48A	37.7	78.1	0.9	78.1
48B	37.7	68.0	0.6	80.1
48C	38.0	78.4	0.9	80.0

Note: n/a = not applicable. 1 in. = 25.4 mm; 1 kip = 4.448 kN.

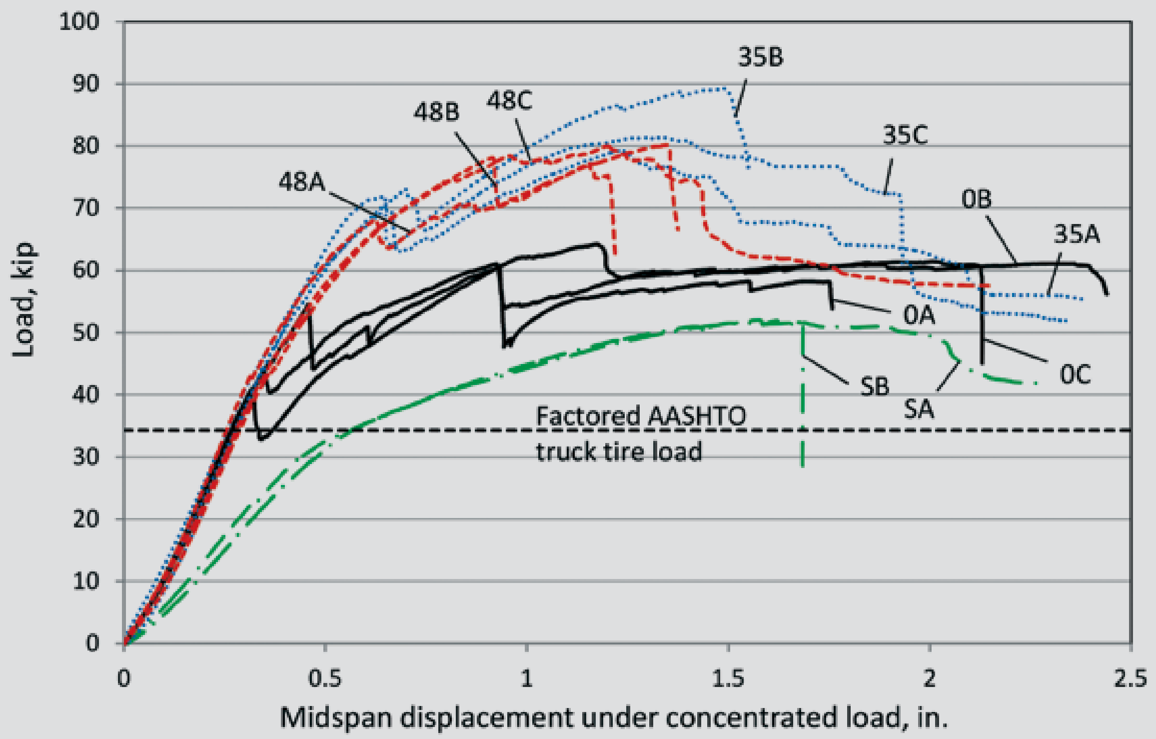


Figure 6. Applied load versus midspan displacement under concentrated load. Note: AASHTO = American Association of State Highway and Transportation Officials. 1 in. = 2.54 mm; 1 kip = 4.448 kN.

(15 to 18 mm), corresponding to an initial joint cracking load of 69.8 to 73 kip (311 to 325 kN). The peak load ranged from 79.3 to 89.3 kip (353 to 397 kN). Posttensioned specimens 48A, 48B, and 48C had an initial drop in capacity at 0.6 to 0.9 in. (15 to 23 mm), corresponding to an initial cracking load of 68.0 to 78.4 kip (302 to 349 kN). The peak load ranged from 78.1 to 80.0 kip (347 to 356 kN).

The nine grouted panel specimens experienced failure on the side of the loaded panel. They achieved a greater peak load than the single 18 in. (460 mm) wide panels by 13 to 39 kip (58 to 170 kN). The peak load was higher than the initial cracking load. This indicates that after the initial significant crack occurred there was additional load transfer across the joint due to friction.

Strain gauge data analysis

Figure 7 shows the measured strain at the center of the bottom longitudinal reinforcement on the loaded side adjacent to the joint. The strain in the bottom steel reinforcement prior to the initial cracking load was similar for nonposttensioned specimens (OB) and posttensioned specimens (35C, 48C), indicating that the grouted panels behave monolithically before the initial joint crack. A strain higher than 2000 $\mu\varepsilon$ was measured in the bottom longitudinal bars prior to initial cracking of the posttensioned specimens, indicating that the bottom reinforcement had already yielded.

Figure 8 compares strains in the reinforcing steel for specimen 0C without posttensioning and posttensioned specimen 35A. Figure 8 shows the bottom longitudinal steel strains for specimen 0C. Prior to initial cracking, the strains were similar for the loaded and nonloaded panels. After initial cracking, the bottom steel for the nonloaded panel had a drop in measured strain, while the strain in the loaded panel kept increasing past yield. The strain in the bottom longitudinal steel bars of the loaded panel reached up to four times the strain of the same bars on the nonloaded panel after the initial cracking load. This indicates a change of deflection behavior across the joint after initial joint cracking. Figure 8 shows the strains for the top and bottom longitudinal steel reinforcement of posttensioned specimen 35A. Prior to initial cracking, the bottom steel reinforcement on the loaded panel and nonloaded panel of the specimen had similar strains, indicating monolithic behavior. The strain increased for both the top and bottom steel on the loaded side after the initial cracking load. The strain in the bottom longitudinal steel bars of the loaded panel reached only two times the strain of the same bars on the nonloaded panel after the initial cracking load. This indicates that posttensioning was beneficial in load sharing between the two panels across the joint after initial joint cracking.

Comparing the graphs in Fig. 8, the bottom steel on the loaded panel of posttensioned specimen 35A achieved three times the strain of the bottom steel on the loaded panel of nonposttensioned specimen 0C. This shows that

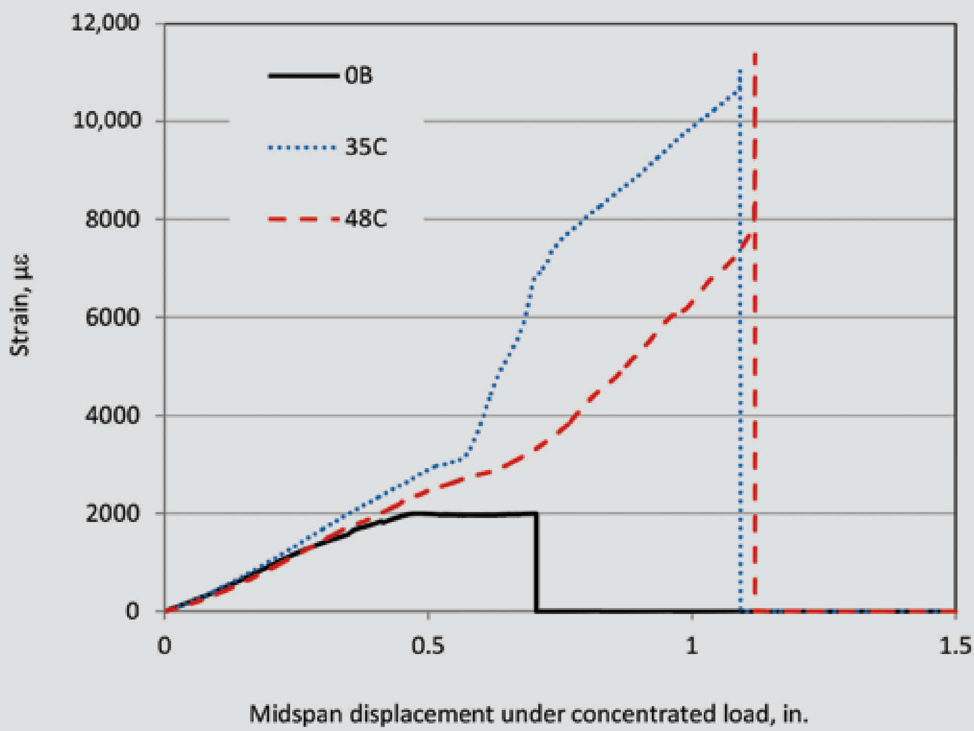


Figure 7. Strain versus displacement for bottom longitudinal bars on loaded side. Note: 1 in. = 2.54 mm.

posttensioning permitted the two panels of specimen 35A to behave in a more composite manner, thus achieving a higher initial cracking load and higher peak load compared with specimen 0C without posttensioning. Similar results were obtained for the 48 psi (330 kPa) posttensioned specimens.

The use of posttensioning increased the initial significant joint cracking load by 58% to 65% and the peak load by 28% to 34%, compared with the joint with no posttensioning. The increase of average posttensioning stress from 35 to 48 psi (240 to 330 kPa) did not affect the peak load but had a slight increase of 9.7 kip (43 kN) in the initial cracking load at the joint. This is because the panels and grout behave as a weak-link system. The weakest portion of the nonposttensioned specimen was the bonded interface between grout and concrete. The posttensioning was sufficient to overcome bond failure, leaving the tensile strength of the grout as the weakest component of the joint. **Figure 9** shows the average strain in the CFRP rods for posttensioned specimen 35A. Left (L tendon) and right

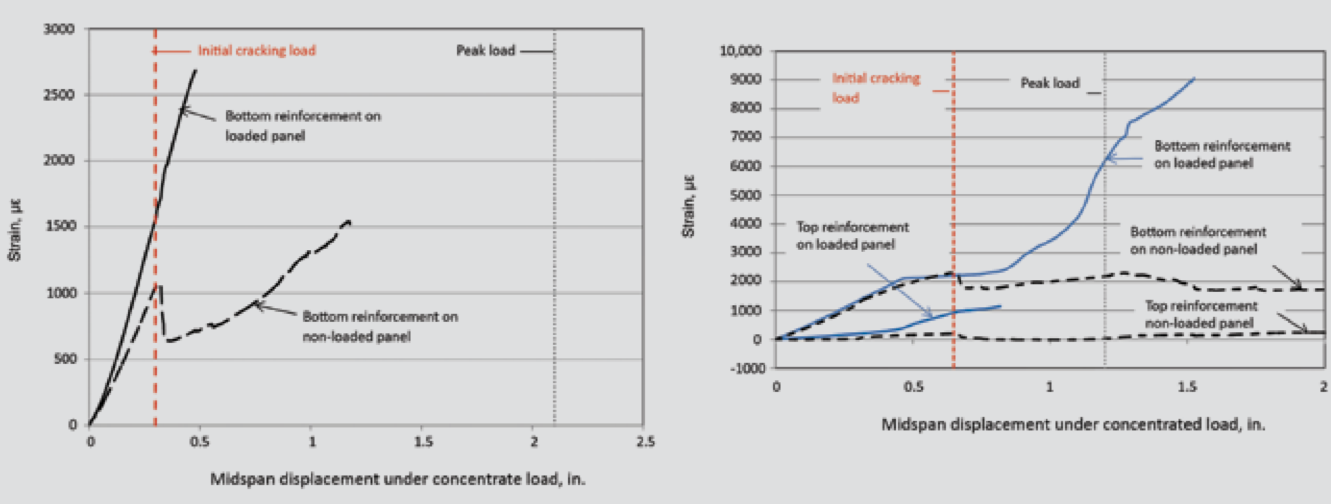


Figure 8. Longitudinal reinforcing bar strain. Note: 1 in. = 2.54 mm.

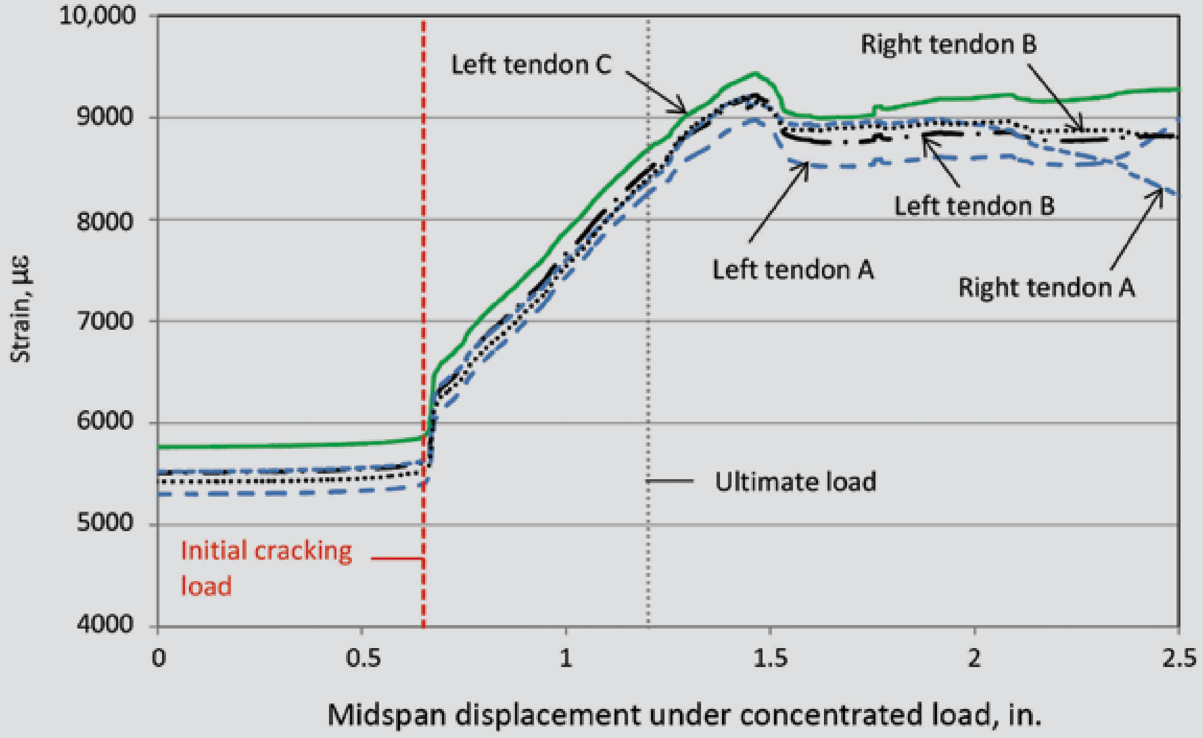


Figure 9. Carbon-fiber-reinforced polymer tendon strain versus displacement for posttensioned specimen 35A. Note: 1 in. = 2.54 mm.

(R tendon) correspond to CFRP rods at the left and right side of the specimen (Fig. 4). Tendon B refers to the strain gauge located at the center point of the CFRP rod, and tendons A and C refer to strain gauges at the quarter points of the CFRP rod. Prior to initial cracking, the strain in the CFRP rods was constant. An increase in CFRP strain occurred after initial joint cracking, indicating that a different deflection behavior across the joint had occurred. Several CFRP rods fractured at or below their ultimate tensile strain due to vertical and rotational deformations between the panel end faces. **Figure 10** shows the condition of a posttensioning CFRP rod at fracture, which typically occurred near the anchor at a displacement much higher than that corresponding to the initial joint cracking load.

Comparison with AASHTO HL-93 double-tire truck loading

The AASHTO HL-93 double-tire truck load[10] of 16 kip (71 kN) was adjusted to determine the required capacity of the joint under a concentrated tire truck load. A load combination factor γ_p of 1.75 was applied for Strength I limit state load combination. An importance factor η_I of 1.05 was applied for the worst case condition. A ductility factor η_D of 1.05 was applied due to the brittle nature of the failure modes, with a redundancy factor η_R of 1.0. A factor ϕ of 0.9 was applied due to shear failure of the grout and concrete at the joint. Thus, the required factored load capacity of the joint under test conditions was 34.3 kip (153 kN).

The average initial joint cracking load of the nonposttensioned specimens was 1.32 times the factored AASHTO design HL-93 double-tire truck load of 34.3 kip (153 kN). The cracking load was 2.09 and 2.18 times the factored AASHTO design HL-93 double-tire truck load for the 35 psi (240 kPa) and 48 psi (330 kPa) posttensioned specimens, respectively. The specimens have a capacity higher than the AASHTO required capacity, indicating an acceptable joint configuration for the HL-93 double-tire truck concentrated loading.

Figure 10. Fracture of carbon-fiber-reinforced polymer posttensioned rod.

Joint posttensioning stress, psi	Initial joint crack load, kip	Factored AASHTO HL-93 static load, kip	Test/AASHTO static	Factored AASHTO HL-93 dynamic load, kip	Test/AASHTO dynamic
0	45.3	34.3	1.32	60.0	0.76
35	71.6	34.3	2.09	60.0	1.19
48	74.8	34.3	2.18	60.0	1.25

Table 2. Average tested and required AASHTO values

Note: AASHTO = American Association of State Highway and Transportation Officials. 1 in. = 25.4 mm; 1 kip = 4.448 kN.

To account for an equivalent dynamic or impact load due to uneven deck surfaces for the design static capacity, the HL-93 double-tire truck load of 34.3 kip (153 kN) is multiplied by a dynamic load factor of 1.75. This gives an equivalent impact load of 60 kip (270 kN). The nonposttensioned specimens had an average initial joint cracking load that was only 0.76 times this equivalent impact load (**Table 2**). Low-level posttensioning at the panel joint area was sufficient to exceed the equivalent impact load by a factor of 1.19 and 1.25 for the 35 psi (240 kPa) and 48 psi (330 kPa) posttensioned specimens, respectively.

Conclusion

Nine precast concrete specimens were tested to determine the effect of low-level posttensioning at the panel joint area using CFRP rods on a new female-to-female grouted transverse joint. The following conclusions can be drawn from this research:

- When the compressive strength of the grout at the transverse joint between precast concrete bridge deck panels is lower than that of the precast concrete panel, the initial significant crack occurs at the joint in the grout or at the bonded surface between the grout and the concrete.

- There are two different modes of grouted transverse joint failure: bond failure between the grout and precast concrete panel and tensile failure of the grout. The nonposttensioned specimens failed in the bond-failure mode. A low posttensioning stress was sufficient to overcome bond failure between the grout and precast concrete panel. The use of low posttensioning from 35 to 48 psi (240 to 330 kPa) increased the initial joint cracking load by 58% to 65% and the peak load by 28% to 34% compared with the nonposttensioned specimens.

- Jointed panel specimens behaved monolithically prior to initial joint cracking. All specimens had an initial joint cracking capacity larger than the required factored AASHTO HL-93 static double-tire truck load. The initial cracking load of the nonposttensioned, 35 psi (240 kPa) posttensioned, and 48 psi (330 kPa) posttensioned specimens was 1.32, 2.09, and 2.18 times the required factored AASHTO load, respectively. However, when impact was considered, the nonposttensioned specimens had an average capacity that was 0.76 times the equivalent impact double-tire truck load. A low posttensioning stress was sufficient to overcome the required equivalent impact load by a factor of 1.19 and 1.25 for the 35 psi (240 kPa) and 48 psi (330 kPa) posttensioned specimens, respectively.

- There is additional load transfer across the joint after the initial significant crack. The peak load was up to 70% higher than the initial joint cracking load for the nonposttensioned specimens, up to 24% higher for the 35 psi (240 kPa) posttensioned specimens, and up to 18% higher for the 48 psi (330 kPa) posttensioned specimens.

- The use of CFRP rods was beneficial for posttensioning across the transverse joint of precast concrete deck panels at low posttensioning at the panel joint area. Additional research should be conducted for higher posttensioning stresses than the ones used in this research. This could be achieved by using more CFRP rods posttensioned to similar stresses. The results of such research could lead to steel-free bridge decks that are constructed with GFRP reinforced full-depth precast concrete panels and posttensioned with CFRP rods.

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References

1. Federal Highway Administration. "Prefabricated Bridge Elements and Systems." Last modified May 18, 2012. http://www.fhwa.dot.gov/bridge/abc/prefab_def.cfm.

2. Issa, M. A., A.-T. Idriss, I. I. Kaspar, and S. Y. Khayyat. 1995. "Full Depth Precast and Precast, Prestressed Concrete Bridge Deck Panels." *PCI Journal* 40 (1): 59–80.

3. Hanna, K. E., G. Morcous, and M. K. Tadros. 2010. *Second Generation Precast Deck Panel (NUDECK) System*. Lincoln, NE: Nebraska Department of Roads.

4. Biswas, M. 1986. "Precast Bridge Deck Design Systems." *PCI Journal* 31 (2): 40–94.

5. Yamane, T., M. K. Tadros, S. S. Badie, and M. C. Baishya. 1998. "Full Depth Precast, Prestressed Bridge Deck System." *PCI Journal* 43 (3): 104–120.

6. Issa, M. A., R. Anderson, T. Domagalski, S. Asfour, and M. S. Islam. 2007. "Full-Scale Testing of Prefabricated Full-Depth Precast Concrete Bridge Deck Panel System." *ACI Structural Journal* 104 (3): 324–332.

7. Carter III, J. W., T. Pilgrim, F. K. Hubbard, T. Poehnelt, and M. Oliva. 2007. "Wisconsin's Use of Full-Depth Precast Concrete Deck Panels Keeps Interstate 90 Open to Traffic." *PCI Journal* 52 (1): 16–30.

8. Badie, S. S., and M. K. Tadros. 2008. "Full-Depth Precast Concrete Bridge Deck Panel Systems." NCHRP (National Cooperative Highway Research Project) report 584, ed. M. K. Tadros, V. 110. Washington, DC: Transportation Research Board.

9. Graybeal, B. A. 2010. "Behavior of Field-Cast Ultra-High-Performance Concrete Bridge Deck Connections under Cyclic and Static Structural Loading." FHWAHRT- 11-023. Springfield, VA: National Technical Information Service.

10. AASHTO (American Association of State Highway and Transportation Officials). 2012. *AASHTO LRFD Bridge Design Specifications*. Washington DC: AASHTO.

11. Issa, M. A., A. A. Yousif, M. A. Issa, I. I. Kaspar, and S. Y. Khayyat. 1998. "Analysis of Full Depth Precast Concrete Bridge Deck Panels." *PCI Journal* 43 (1): 74–85.

12. Kim, Y.-C., S. Shin, and J.-J. Park. 2003. "Shear and Fatigue Strength of Grout-Type Transverse Joints." *Canadian Journal of Civil Engineering* 30 (4): 607–614.

13. ACI (American Concrete Institute) Committee 318. 2014. *Building Code Requirements for Structural Concrete (ACI 318-14) and Commentary (ACI 318R-14)*. Farmington Hills, MI: ACI.

14. Roberts, K. S. 2011. "Performance of Transverse Post-Tensioned Joints Subjected to Negative Bending and Shear Stresses of Full Scale, Full Depth, Precast Concrete Bridge Deck Systems." Master's thesis, Utah State University.

15. Swenty, M. K. 2009. "The Investigation of Transverse Joints and Grouts on Full Depth Concrete Bridge Deck Panels." PhD diss., Virginia Polytechnic Institute and State University, Blacksburg, VA.

16. Swenty, M. K., C. L. Roberts-Wollmann, and T. E. Cousins. 2014. "Transverse Panel-to-Panel Connections for Full-Depth Precast Concrete Bridge Deck Panels on Continuous Steel Girder Bridges: Part 1, Experimental." *PCI Journal* 59 (2): 62–77.

17. Bowers, S. E. 2007. "Recommendations for Longitudinal Post-Tensioning in Full-Depth Precast Concrete Bridge Deck Panels." Master's thesis, Virginia Polytechnic Institute and State University, Blacksburg, VA.

18. Pantelides, C. P., J. Ries, and R. Nix. 2013. "Construction and Monitoring of a Single-Span Bridge with Precast Concrete Glass-Fiber-Reinforced Polymer Reinforced Deck Panels." *PCI Journal* 58 (1): 78–95.

19. Burningham, C. A., C. P. Pantelides, and L. D. Reaveley. 2014. "New Unibody Clamp Anchors for Post-tensioning Carbon-Fiber-Reinforced Polymer Rods." *PCI Journal* 59 (1): 103–113.

20. Burningham, C. A., C. P. Pantelides, and L. D. Reaveley, 2014. "Repair of Prestressed Concrete Beams with Damaged Steel Tendons using Post-tensioned Carbon Fiber-Reinforced Polymer Rods." *ACI Structural Journal* 111 (2): 387–395.

Notation

f_c' = concrete compressive strength

γ_p = load combination factor

η_D = ductility factor

η_I = importance factor

η_R = redundancy factor

ϕ = factor

About the authors

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Abstract

One method employed in accelerated bridge construction is the use of full-depth precast concrete panels to construct new bridge decks or replace existing ones. The grouted transverse joint between precast concrete deck panels is one of the most vulnerable elements of the bridge deck. To extend the life of precast concrete bridge decks, it is imperative to improve the integrity of the grouted transverse joint. This paper compares the effect of low posttensioning stress on the initial joint cracking load, the peak load, and the overall behavior of precast concrete panels at a new female-to-female grouted transverse deck joint. Nine specimens with grouted transverse joints were tested monotonically to failure using an equivalent concentrated double-tire truck load. Three of the nine specimens had no posttensioning, three specimens were subjected to a 35 psi (240 kPa) posttensioning stress, and three were subjected to a 48 psi (330 kPa) posttensioning stress using carbon-fiber-reinforced polymer rods. Low posttensioning stress in the panel joint area increased both the initial joint cracking load and peak load capacity of the specimens and changed the failure mode from a bond failure of grout to concrete for the specimens without posttensioning to a tensile failure of the grout for the post-tensioned specimens.

Keywords

Accelerated bridge construction, bridge, carbon-fiber-reinforced polymer, cracking load, deck, joint, posttensioning.

Review policy

This paper was reviewed in accordance with the Precast/Prestressed Concrete Institute's peer-review process.

Reader comments

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