EVALUATION OF FLANGE-TO-FLANGE CONNECTION OF PRECAST CONCRETE DECK USING UHPFRC

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ABSTRACT
Recently, as the social demand for accelerated construction has increased, the importance of robust connection details for precast concrete construction has been highlighted. These connections are required to secure structural performance and to ensure high durability. In this study, the structural responses of moment connections of a precast concrete deck filled with ultra-high performance, fiber-reinforced concrete (UHPFRC) are investigated. First, flexural tests of large-scale precast beam specimens connected using UHPFC are conducted. In these tests, two types of non-contact splice details, straight bars and U-bars, were applied along with various joint widths ranging from 10 to 4 in. [250 to 150 mm]. Then, the fatigue behavior of two precast concrete deck specimens with a joint width of 7 in. [180 mm] was evaluated. The test results indicate that UHPFRC had considerable bonding performance with rebars. As a result, the flexural behavior of the joint connection was not appreciably affected by the splice details, and the precast concrete deck specimens also exhibited acceptable fatigue responses.

Keywords: UHPFRC, Precast concrete, Bridge deck connection, Flange-to-flange connection

INTRODUCTION

Lately, the application of precast concrete girders with a wide flange such as a decked bulb-tee girders has been increased for rapid constructions. These precast girders are typically integrated using narrow joint connections between the girders. In the joint connection, rebars extruding from the girders are positioned to be spliced when the joint is filled with concrete or mortar. Because the procedures to construct a deck slab is simply substituted to the joint casting, a wide-flange precast girder has the advantage of reducing the construction period and cost. In this case, the joint connection is an important element to determine the structural characteristics of the bridge and the durability of the entire structure is largely dependent on this joint, which connects two precast girders using the joint casting in field. As a result, this connection is required to secure the structural performance and to ensure high durability. Accordingly, a robust connection for a flange-to-flange connection is critical. Typical desired material characteristics of the closure material are early strength achievement, anti-shrinkage properties, and low permeability. In North America, UHPFRC has been found to be an effective alternative which offers both structural performance and durability, and has been applied in recent construction projects[1].

Designers of precast concrete joints prefer the rebar details to be simplified and the joint width to be minimized for constructionability. A narrow joint width may reduce the formworks for joint casting and the quantity of the closure material can be decreased, of which the cost is higher than that of ordinary concrete. The joint width is determined based on the splice length of the rebars extruding from the girders to be spliced to transfer the stress from one bar to another. The splice length can be reduced by providing anchorage to the rebars and by enhancing the bond strength between the steel rebars and the concrete. Various details can be used to provide the anchorage in the splices[2,3,4] and U-loop bars have been effectively utilized for a joint filled with normal-strength concrete. On the other hand, superior bonding strength from UHPFRC was reported with or even without steel fiber reinforcement[5]. The UHPFRC may be characterized by high compressive strength and post-cracking behavior induced by the fiber reinforcement. The bonding strength was expected to increase because the tensile strength exerted after an onset of cracking can retain the bonding mechanism after concrete splitting. This bonding characteristic reveals the good possibility of reducing the splice length without the anchorage, but this advantage has not been utilized due to concerns with the brittleness of the high-strength concrete and a lack of relevant experimental data[6,7].

In this study, two feasible connection details, straight bars and U-loop bars, were implanted in a precast joint which was filled with UHPFRC. The flexural and fatigue behavior of precast concrete specimens were investigated. The joint width was determined based on a minimum splice length for a 180-degree standard hook specified in the ACI design specifications[8] (Eq. (1)). The minimum splice length for an uncoated deformed bar with a yield strength of 60 ksi [400 MPa] embedded in concrete with a compressive strength of 20 ksi [130 MPa] can be computed as $(0.02\times1.0\times60{,}000 \;/\; 1.0\times\sqrt{10{,}000})\ d_b = 12.0\ d_b$. It should be noted that Eq. (1) cannot reflect the advantage of the superior bonding strength of UHPFRC due to a maximum compressive strength limitation of 10 ksi instead of 20 ksi. The details of the two splice types used in this study are presented in Fig. 1.

$$\ell_{dh} = (0.02\ \psi_e\ f_y\ /\ \lambda\sqrt{f_c'})\ d_b \quad (1)$$

where ψ_e is the modification factor for the epoxy coating, f_y is the yield strength of the rebar, λ is the modification factor for lightweight concrete, f_c' is the compressive strength of concrete, and d_b is the diameter of the rebar

(a) straight bars

(b) U-loop bars

Fig. 1 Comparison of connection details (with joint width of 10 in.)

MATERIAL CHARACTERISTICS OF UHPFRC

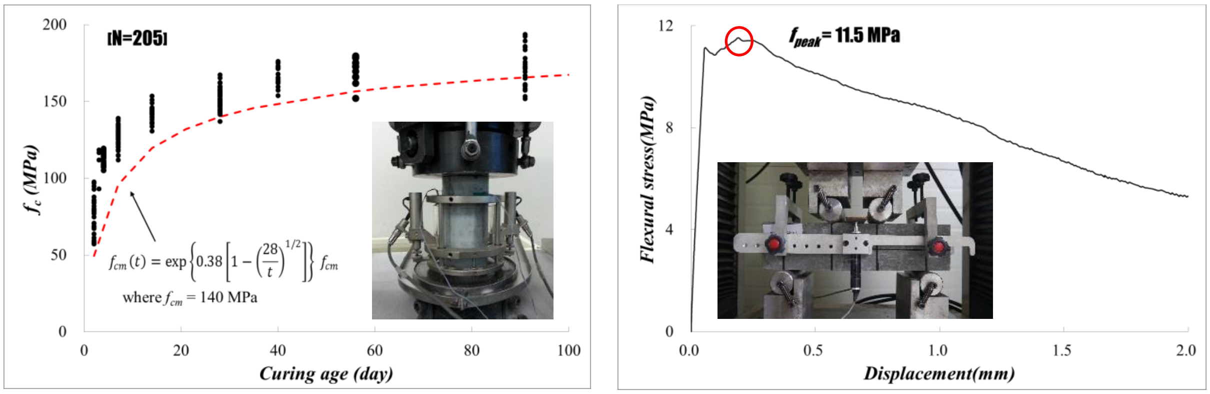
The UHPFRC is newly developed and used in the experiment. UHPFRC consists of ordinary Portland cement along with binder supplements of silica fume and blast furnace slag. In comparison to commercially available UHPFRC, it includes gravel aggregates by 867 kg/m^3. The constituents of UHPFRC were mixed with potable water by 0.17 *w/cm* and a superplasticizer. Steel fiber reinforcement, 1.6 % by volume, was also incorporated. The standard mix design is given in Table 1.

Table 1 Standard mix proportion of UHPFRC

w/cm (%)	S/a (%)	Unit weight (kg/m^3)							
		W	OPC	ZSF*	BFS**	Gy***	Sand	Gravel	SF****
17	37	150	573	106	176	27	571	867	126

*ZSF: Silica fume, **BFS: Blast furnace slag, ***Gy: Gypsum, ****SF: Steel fiber (V_f=1.6%)

The compressive strength gain depending on the curing age was tested using approximately 200 cylinder specimens over 90 days in the laboratory. The recorded compressive strength of the UHPFRC mostly exceeded 21 ksi (140 MPa) at 28 days, and a considerable portion of the compressive strength over 14.5 ksi [100 MPa] was achieved by 7 days (Fig. 2(a)). In this graph, the predicted compressive strength depending on the number of curing days proposed in Eurocode 2[9] is also plotted for a comparison. The flexural behavior of UHPFRC was tested in accordance with ASTM C 1609[10]. A specimen 4×4×15.7 in. [100×100×400 mm] in size was subjected to a four-point loading condition to produce a constant moment region. In the test, a crack initiated at the midpoint of the specimens and progressed. The load and displacement relationship acquired from` the test is presented in Fig. 2(b). After cracking, the UHPFRC specimen still exerted tensile strength and exhibited gradual tensile strain-softening response. The test results show that the brittleness of high-strength concrete was compensated by the steel fiber reinforcement.



(a) compressive strength (b) flexural toughness

Fig. 2 Material characteristics of UHPFRC (Note: 1 ksi = 6.89 MPa, 1 in. = 25.4 mm)

FLEXURAL EVALAUTION OF A PRECAST CONNECTION WITH UHPFRC

Precast beam specimens were fabricated to investigate the flexural behavior of a connection joint filled with UHPFRC. The geometry of the specimens was 31.5×8.7×118.1 in. [780 × 220 × 3,000 mm]. The depth of the specimens was determined based on a minimum depth requirement in accordance with the domestic highway design code[11]. Two segments for each specimen were cast separately, positioned in parallel, and then connected by pouring UHPFRC into the joint. No. 5 steel rebars (D16 mm) were arranged by distance of 6.0 in. [150 mm] in each segment and staggered by distance 3.0 in. [75 mm] in the joint as shown in Fig. 3. The joint width was varied from 10 to 6 in. [250 to 150 mm] and the splice length decreased as the joint width decreased from 6.3 to 4.3 in. [160 to 110 mm] or from 10 to 7 d_b. A monolithic concrete specimen without a joint connection was also fabricated as a control specimen to compare the flexural responses. Details of test specimens are summarized in Table 2.

Table 2. Details of precast beam specimens (Note: 1 in. = 25.4 mm, 1ksi = 6.89 ksi)

Specimen	Joint width, mm(in.)	f_c' *, MPa(ksi)	Splice length ,mm(in.)	Remark
P-N-Inf-S	none	none	none	control
P-250-160-S	250, (10)	135,(19.6)	160, (6.3)	straight bar
P-200-140-S	200, (8)	124,(18.0)	140, (5.5)	
P-150-110-S	150, (6)	137,(19.9)	110, (4.3)	
P-250-160-U	250, (10)	135,(19.6)	160, (6.3)	U-loop bar
P-200-140-U	200, (8)	124,(18.0)	140, (5.5)	
P-150-110-U	150, (6)	137,(19.9)	110, (4.3)	

*compressive strength of UHPFRC at testing

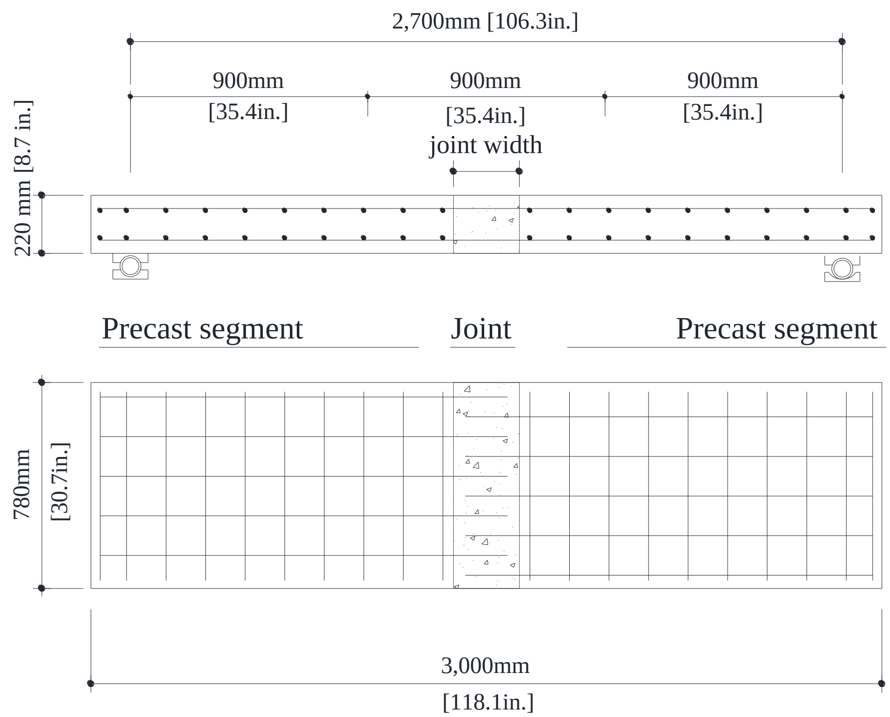


Fig. 3 Geometry of the precast beam specimens (Note: 1 in. = 25.4 mm)

The specimens were subjected to flexural bending until they collapsed and the joint connection was located in the constant moment region at middle. As the load increased, the displacement of the specimen was recorded using two displacement transducers. The load and displacement relationships are plotted in Fig. 4. The flexural responses of the specimens were similar with those of a typical beam in that the linearized curve became non-linear as a flexural crack occurred first, after which the slope of the curve sharply decreased as tension steel yielded. Slight variations among these curves were mostly induced by differences in the compressive strength of concrete. These flexural responses were not affected by types of splice details. Furthermore, no appreciable difference in the load-to-displacement curves was observed between the monolithic beam and the precast beams. The load and displacement curve obtained from the monolithic beam specimen is plotted in a thick black line.

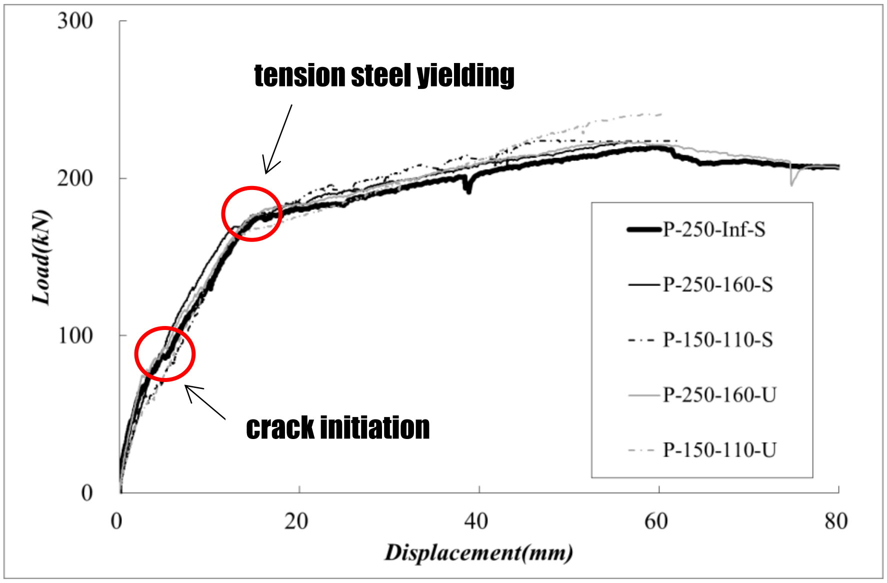


Fig. 4 Load and displacement relationship (Note: 1kip = 4.45 kN, 1in. = 25.4 mm)

All of the precast specimens collapsed due to a compressive failure of the concrete in the precast concrete segment adjacent to the joint. A failure was not observed from the joint connection because it had higher cross-sectional moment strength than the precast segments due to the higher compressive strength of the concrete and the steel fiber reinforcement. In the flexural test, a flexural crack tended to initiate along the interface of the joint. However, as the amplitude of the load increased, the flexural cracks became distributed over the length, and the crack width and depth along the joint interface were similar to those of other flexural cracks. Upon completion, the flexural cracks were well distributed over the length. A representative distribution of the flexural cracks is presented in Fig. 5.

Fig. 5 Failure of specimens upon completion (P-250-160-U)

The test results indicated that the flexural responses of all specimens were reasonably equivalent. The flexural behaviors were in good agreement even with the specimen without the joint connection. Furthermore, the flexural behavior was not affected by the splice details. The specimens with straight bars exhibited equivalent flexural responses to those of the U-loop bars. The splice length for the straight bars or lap splices was as short as 4.3 in. [110 mm] or 7 d_b. These experimental results indicate that the bonding between the UHPFRC and the rebars was considerable enough for short lap splices to perform equivalently to hooked splices.

FATIGUE EVALAUTION OF A PRECAST DECK CONNECTION WITH UHPFRC

The fatigue behavior of the precast deck connection with UHPFRC was evaluated using two large-scale precast deck specimens. The specimens had a geometry of 85×8.7×98.4 in. [2,160×220×2,500 mm]. As with the precast beam specimens, the two precast segments are prefabricated and then connected using a joint connection with UHPFRC (Fig. 6). The joint width was determined to be 7.1 in. [180 mm] and the geometry of the joint was determined diamond. The splice length was 6.3 in. [160 mm] or 10 d_b. Details of the specimens are summarized in Table. 3. Details of the precast deck specimens are presented in Fig. 7 and the geometric characteristics of the two joint connections are shown in Fig. 8.

(a) UHPFRC casting (b) completion of the fabrication

Fig. 6 Fabrication of precast deck specimens

Table 3. Details of the precast deck specimens (Note: 1 ksi = 6.89 ksi, 1 in. = 25.4 mm)

Specimen	f_c'*, MPa (ksi)		Splice length, mm (in.)	Bar details
	Deck	Joint		
SYM-160-S	50 (7.3)	140 (20.3)	160 (6.3)	straight
SYM-160-U	54 (7.8)	140 (20.3)	160 (6.3)	U-loop

*compressive strength of concrete at testing

A series of the flexural tests verified the flexural strength of the joint connection filled with UHPFRC. These types of fatigue tests aim to investigate the serviceability of the joint connection. The serviceability is related to the progress of concrete cracks and the fatigue failure of steel rebars induced by a service load. A cyclic load was applied using a hydraulic actuator. The actuator was positioned adjacent to the joint connection to induce the maximum amount of flexural stress at the interface of the connection on the bottom surface.

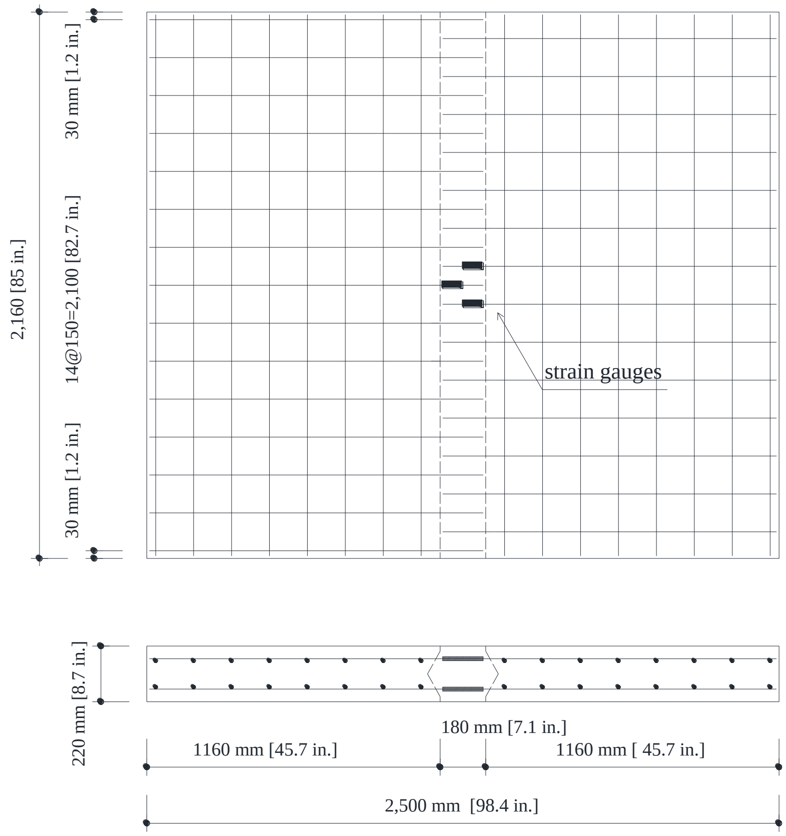
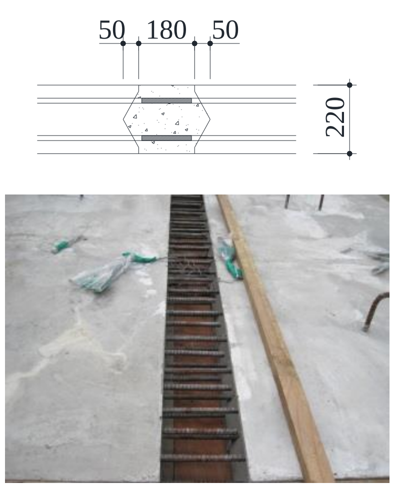
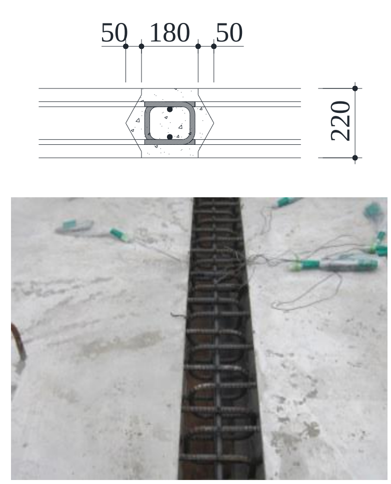


Fig. 7 Geometry of the precast deck specimens (Note: 1 in. = 25.4 mm)



(a) SYM-160-S



(b) SYM-160-U

Fig. 8 Geometry of the precast deck joint and bar details (Note: 1 in. = 25.4 mm)

The cyclic load was computed to produce service-level fatigue loading on the joint connection. The produced moment load per unit width caused by a live load was approximated using Eq. (2)[9] as (2.45+0.6) × 78 (1+0.15) / 9.6 = 28.5 kN-m. The standard wheel load for the fatigue and distance between the spans (L) was 17.5 kip [78 kN] including dynamic load allowances (IM) of 0.15 and 8 ft [2.45 m], respectively. The joint connection was subjected to cyclic loading with amplitudes of 21.0 to 24.3 kip-ft [28.5 to 33.0 kN-m] from left to right. The cyclic load was applied with a frequency of 5 Hz over 2 million cycles.

$$M_T = (L + 0.6)\ P\ (1 + IM)/9.6 \quad (kN - m/m) \tag{2}$$

where L is the distance between spans and P is the fatigue axial-wheel load.

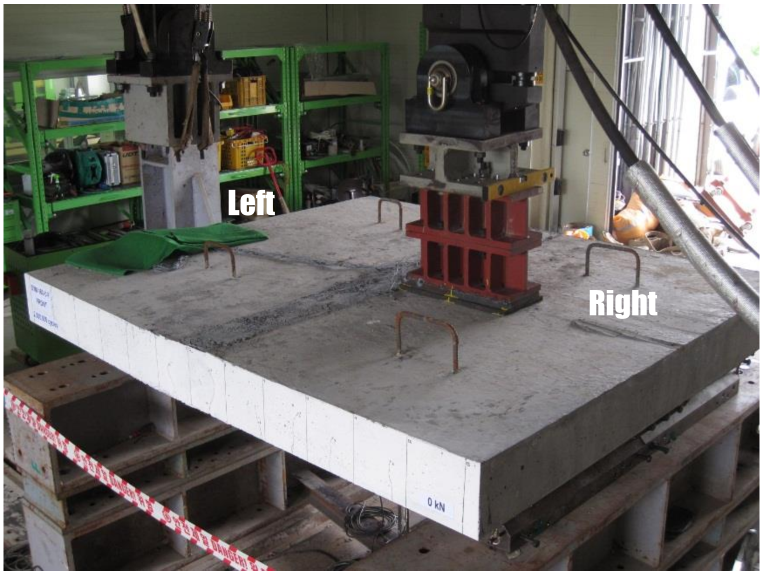


Fig. 9 Overview of fatigue test set-up

The stresses were computed by multiplying an elastic modulus of 29,000 ksi [200 GPa] with recorded strains. Strain gauges were positioned within the precast joint connection (Fig. 7). The stress range (Δf) in the rebars was evaluated using the characteristic fatigue strength curve specified in Eurocode-2[7]. The characteristic fatigue strength curve provides the resisting stress range ($\Delta\sigma_{Rsk}$) can be considered as the allowable stress range at N cycles. The stress ranges caused by the cyclic load over 2 million cycles are plotted in Fig. 10. During the fatigue test, no significant increment in the stress range was observed, and the stress range was within the resisting stress range. These test results suggest that the rebars were unlikely to be damaged by fatigue loading.

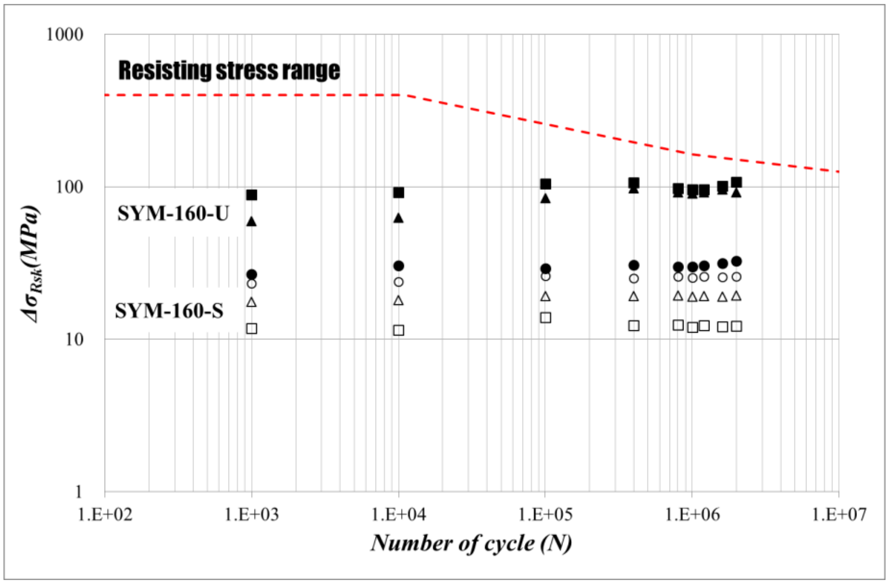
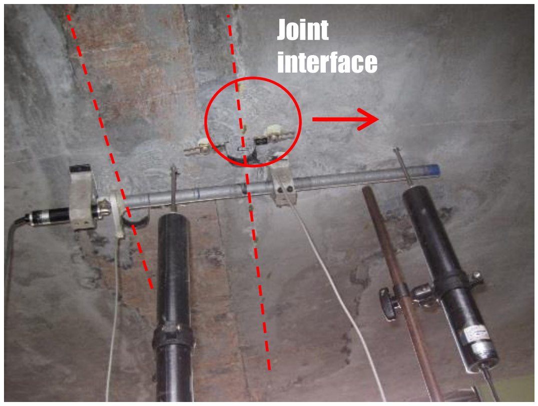


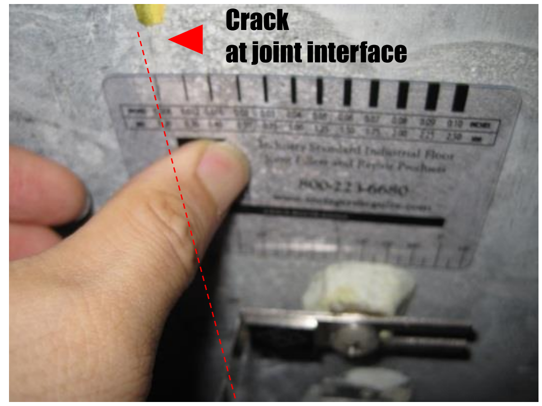
Fig. 10 Characteristic fatigue strength curve (Note: 1 ksi = 6.89 MPa)

As the cyclic load was applied, the crack at the interface was initiated at an early stage. The cracks were then distributed in the longitudinal direction. The distribution of cracks was inspected after each of 0.4 million cycles. At each time, the cyclic load was interrupted and cracks were observed. Then, the cracks observed from the first inspection at 0.4 million cycles did not exhibit substantial progress until the fatigue test terminated at 2 million cycles (Fig. 11). At first, the interface of the joint connection was considered to be susceptible to cyclic loading. However, no appreciable progress of the crack which developed at an early stage was observed during the test. The crack width along the interface was less than 0.008 in. [0.2 mm] upon the completion of the test. The distribution of the cracks upon the completion of the fatigue tests from the bottom surface is presented in Fig. 12.

Fig. 11. Distribution of the crack over the length upon the completion of the fatigue test (SYM-160-S)



(a) cracks on the bottom surface



(b) interfacial crack width

Fig. 12 Residual crack at the joint interface upon the completion of the fatigue test

Fatigue damage in concrete structures may be separated into a failure of the steel rebar and the progress of micro-cracks in the concrete. In a fatigue evaluation, experimental results showed that no appreciable crack progress was observed at the joint connection over a 2 million cyclic load and that the steel rebars were not susceptible to fatigue failure. These observations indicate that the precast deck connection filled with UHPFRC exhibits allowable structural responses to resist service-level fatigue loads.

CONCLUSIONS

The experimental results demonstrated structural performance of the joint connection filled with UHPFRC. The bonding performance of the UHPFRC with the steel rebars was remarkable enough for short lap splice of 4.3 in. [110 mm] to transfer applied tension until the specimens collapsed. The flexural behavior was not influenced by either the splice length or the splice type. The specimens with the straight bars exhibit the structural responses equivalent to those from the specimens with the U-loop bars. It indicates that sufficient bonding strength to transfer applied tension was exerted by the splices consisting of the straight bars and the additional anchorage from the U-loop bars did not appreciably contribute to the bonding strength.

In the fatigue evaluation, large-scale precast deck specimens did not exhibit appreciable damage after 2-million cyclic loads. Flexural cracks were induced by the applied load at an early stage, but they did not progress as the number of cyclic loads increased. The stress range of the rebars caused by the cyclic load was also less than the allowable range. These observations indicate the precast deck specimens connected using the UHPFRC has sufficient fatigue resistance capabilities. These experimental results suggest that simplified splice details are applicable along with remarkably short joint widths when the UHPFRC is used as a closure material in precast concrete structures.

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