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CRACK CONTROL OF LOOP JOINT WITH HIGH STRENGTH CONCRETE

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ABSTRACT

This study discusses crack control of full-depth precast concrete decks connected by a loop joint complying with current design codes. Performance of the precast concrete joint was investigated depending on surface condition of the joint and compressive strength of closure pours. Flexural tests for the precast concrete deck specimens were conducted to evaluate the performance of flexural response and crack resistance. Test results indicated that the precast concrete decks filled with high compressive strength exhibited higher cracking moment strength and ultimate moment strength than those with normal strength concrete. Direct calculation of crack width based on stress of tensile reinforcements needs to be revised for the joints with high strength concrete due to its higher cracking load. In addition, surface at the interface between the joint and the precast concrete was analyzed using SEM (Scanning Electron Microscope) by examining acquired images.

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This study discusses crack control of full-depth precast concrete decks connected by a loop joint complying with current design codes. Performance of the precast concrete joint was investigated depending on surface condition of the joint and compressive strength of closure pours. Flexural tests for the precast concrete deck specimens were conducted to evaluate the performance of flexural response and crack resistance. Test results indicated that the precast concrete decks filled with high compressive strength exhibited higher cracking moment strength and ultimate moment strength than those with normal strength concrete. Direct calculation of crack width based on stress of tensile reinforcements needs to be revised for the joints with high strength concrete due to its higher cracking load. In addition, surface at the interface between the joint and the precast concrete was analyzed using SEM (Scanning Electron Microscope) by examining acquired images.

Introduction

The more traffic volume is increased by urban expansion, the more deterioration of infrastructures is accelerated these days. Durability of bridges is a main issue of design criteria among several limit states in current design codes [1-3]. There is a growing need for durable prefabricated structural systems which facilitate accelerated construction of on-site activities in order to minimize the impact on the environment. Precast members can provide higher quality, fast construction, and safety; however, greater offsite prefabrication of precast elements necessitates an increased reliance on the long-term performance of cast-in-place concrete connections between these components. The cast-in-place joints show often less durability than desirable overall system performance.

Bridge decks have the shortest service life among bridge members because they are heavily stressed throughout their lives by both structural and environmental loadings. For the replacement of the deck, the precast panel system as one of alternatives for this situation is received attention and used in several countries because it can guarantee quality of decks and minimizing formworks [4]. Also, it helps to reduce traffic interruption during the replacement period. On the other hand, many kinds of problems were reported, such as cracking, water leakage and corrosion of steel [5]. Since cracking at the in-situ joints can be a governing factor in design of durability and serviceability, crack control of the joint parts is the key consideration of the precast deck system.

Two kinds of precast deck system are mainly used in practices, half-depth and full-depth.

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While the half-depth precast deck system uses formwork in field, the full-depth system only uses the formwork for joint parts. Much research on various types of precast joints has been already performed [6-8]. Ultimate behavior of the precast deck with loop joints was similar to ordinary RC members without joints in scope that loop joint width is between 250 to 350 mm and loop rebar diameter is between 13 to 19 mm [8]. The performance of slabs with double loop joints was investigated similar to ordinary reference members in service load level [9].

Some researchers have performed study about influent factors of crack width such as reinforcement ratio and diameter, joint spacing and strength of concrete. Influence of reinforcement percentage is substantial in the calculation of crack width, whereas influence of the strength of concrete is less important [10]. In case of precast deck with the loop joint, crack width is influenced mainly by the diameter of reinforcing bars [8].

Generally, variables such as stress in tensile reinforcement, diameter of bars, spacing and cover depth are used for crack width control criteria in several design codes [1-3]. Crack width criterion that is based on theories in Eurocode-2 uses strain of steel and concrete as main variables and considers the direction of main reinforcements. It is adjusted by empirical adjustment using coefficients [3]. The criteria of crack control in AASHTO LRFD and ACI 318 specification based on empirical experiments are founded on steel strain, but no consideration of bonding properties [1-2].

In this paper, the precast concrete decks in this experimental program were connected by loop joint details complying with design criteria. Experimental program with flexural tests was performed to investigate the cracking behaviour at the joint part of precast members with joints using high strength concrete. Comparisons between flexural response of the experiments and theoretical calculation based on design codes which assumes RC structures for crack width control criterion were executed. Furthermore, test results were evaluated depending on different surface treatments for the joints and its compressive strength. In addition, SEM (Scanning Electron Microscope) was used to analyze surfaces at the each interface between precast member and joint in the specimens.

Experimental Program

An experimental program was designed to investigate two limit states of the precast members connected by the loop joints. Cracking load and crack width of the joint are evaluated for the serviceability limit state while ultimate strength of the connected members is for the ultimate limit state. According to Eurocode-2, performance for crack width control of specimens with suitable loop joint detail was estimated.

Test Specimens

The experiments were carried on by twelve specimens with loop joints filled with concrete with different compressive strengths. Each specimen was comprised of two precast segments and a cast-in-place joint part. To design and fabricate the experimental specimens, the requirements of development length of reinforcements in the current design code were considered. The test specimens were be grouped and named by differences of the joint spacing, strength of filling concrete in joint parts, surface treatments, and diameters of loop bars. Table 1 describes the main variables of tested precast members with loop joints. High strength concrete with 130MPa compressive strength was applied in joint parts of DD1-3, DD2-3, and DD3-3 specimens to secure structural durability and to evaluate structural performance, especially concerning serviceability for connection. All specimens were designed in order to observing the criteria of minimum depth for each bridge deck in Korea Limit State Bridge Specification [11].

Table 1. Main variables of test specimens

Specimen	Surface treatments	L (mm)	b (mm)	H (mm)	Strength of C.I.P concrete (MPa)	Diameter of loop (mm) / Yield stress (MPa)	Distance between supports (mm)	Joint spacing (mm)	Loop joint detail
DD1-3-S-A	S.B.*	2500	300	220	136	16 / 400	1200	250	Single
DD1-3-S-B	S.B.*	2500	300	220	136	19 / 400	1200	250	Single
DD2-1-S	S.B.*	2500	400	300	35.5	22 / 400	1200	300	Single
DD2-1-C	C.**	2500	400	300	35.5	22 / 400	1200	300	Single
DD2-1-B	B.C.***	2500	400	300	35.5	22 / 400	1200	300	Single
DD2-3-S-A	S.B.*	2500	300	220	136	16 / 400	1200	300	Single
DD2-3-S-B	S.B.*	2500	300	220	136	19 / 400	1200	300	Single
DD3-3-S-A	S.B.*	2500	300	220	136	16 / 400	1200	350	Single
DD3-3-S-B	S.B.*	2500	300	220	136	19 / 400	1200	350	Single
DD4-1-S	S.B.*	2500	1000	250	35.5	16 / 400	1200	500	Double
DD4-1-C	C.**	2500	1000	250	35.5	16 / 400	1200	500	Double
DD4-1-B	B.C.***	2500	1000	250	35.5	16 / 400	1200	500	Double

* Steel brushing, ** Chipping, *** Concrete bond adhesive coating

Development length makes the concrete structures to have full capacity of strength for transferring loading applied in the structures from concrete to the reinforcements. It is essential consideration for structural unity to prevent inclination of splitting of concrete sections in high stressed reinforcements. The criterion of development length for hooks in ACI 318 is used for design of loop joint details. It is calculated by production between development length of deformed bars in tension, which is in equation (1), l_{dh} , and modification factor in ACI 318-08 code. The development length of hooks, l_{dh}, shall not be less than the larger of $8d_b$ and 150 mm.

$$l_{dh} = \left[\frac{0.24\Psi_e f_y}{\lambda \sqrt{f'_c}} \right] d_b \tag{1}$$

where, l_{dh} is the development length from the tail of the hooked bar in tension, mm; Ψ_e is the epoxy coating factor; λ is the lightweight concrete factor; f_y is the yield strength of reinforcement, MPa; $\sqrt{f'_c}$ is the square root of the concrete compressive strength, MPa; d_b is the diameter of the bar, mm. Since the reinforcements which were used for test specimens were not coated and concrete was not light-weight, both factor Ψ_e and λ were 1.0. The modification factor determined as 1.0, since design condition of specimens did not accord with criterion of it.

Table 2 presents development length of loop bars for each specimen. Except for specimens using high strength concrete for joint parts, all details of the specimens satisfied criteria of development length specifying in the ACI 318 design code, since there are no requirements of high strength concrete for development length in ACI318 code.

Table 2. Development length of test specimens

Specimen	**Loop reinforcement diameter (mm)**	**Designed concrete compressive strength for joint (MPa)**	**Calculated development length (mm)**	**Development length of each specimen (mm)**
DD1-3-S-A	16	130	135 (Out of range)	235
DD1-3-S-B	19	130	160 (Out of range)	237
DD2-1-S	22	40	334	250
DD2-1-C	22	40	334	250
DD2-1-B	22	40	334	250
DD2-3-S-A	16	130	135 (Out of range)	260
DD2-3-S-B	19	130	160 (Out of range)	262
DD3-3-S-A	16	130	135 (Out of range)	285
DD3-3-S-B	19	130	160 (Out of range)	287
DD4-1-S	16	40	243	220
DD4-1-C	16	40	243	220
DD4-1-B	16	40	243	220

Three kinds of surface condition to enhance bond strength of the interface were considered in the experiments shown as Fig. 1. Minimum requirement of the interface is to remove laitance of the surfaces by any means. The ACI stipulates that existing concrete should be moistened thoroughly before placement of the fresh concrete so the parent and the new concrete can achieve full monolithic behavior [12].

The first treatment is manual steel brushing. After removing formworks of the precast members, the surface of the members, which will meet cast-in-place concrete, was manually brushed using a steel brush. The second treatment is chipping. Chipping which is to chop or cut off the piece of surface was executed until exposure of course aggregates. After chipping, steel brushing was also done for cleaning. The concrete bond adhesive was coated on the surfaces of joints for some specimens after treatment of steel brushing. For the adhesive coating, it needs careful treatment to prevent any coating to reinforcing bars.

(a) Steel brushing (b) Chipping (c) Coating with bond adhesive

Figure 1. Surface condition for interface of specimens

Loading and Measurement

Static loading tests with three points loading onto the specimens were performed. In the loading tests, a concentrated load was applied to the center of the span by an actuator of 2000 kN capacity. The applied load was controlled by load control method (0.5kN/sec) from the beginning of loading until observation of first crack. After occurrence of initial crack, applied load was unloaded and two omega gauges were installed to measure crack width of initial crack and flexural crack at bottom surface of mid-span, and then displacement control method (1mm/min) was used until failure of specimens were reached.

The objectives of the measurement plan were to estimate whether loop joint details have sufficient flexural strength and to check the ability of crack control by collected data such as load, strain, and crack width. LVDT (linear variable differential transformer), strain gauges for concrete and steel, crack gauges were installed at each specimen in order to collect the data. A LVDT was placed at bottom-center of each specimen to measure the displacement. Strain gauges for reinforcements were installed at critical parts which may have cracking. Cracking in each specimens were carefully observed during the tests by measuring cracks at sections of a joint and a center. Width of initial cracks was measured using two omega-type gauges which measure crack width after initial crack was generated.

Test Results

Flexural Strength and Development Length of the joints

The specimens had been designed with smaller amount of reinforcements than balanced reinforcement ratio to induce flexural failure with ductile deformation. Test results of all specimens were compared with predicted strengths that were calculated by the ACI 318 code in Table 3 and Table 4. Based on failure mode and location that failures were occurred, theoretical nominal flexural strength and cracking strength were calculated. Specifically, in case of the specimens with joints that were poured by 136 MPa high strength concrete, the theoretical values were computed at two points, one was mid-span and the another one was interface between precast segments and the joint with cast-in-place concrete.

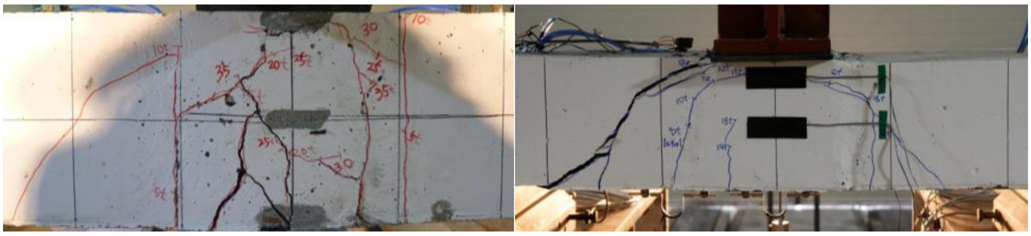
Test results showed that the calculated flexural strengths at each interface showed more exact estimation than the value at the mid-span section. Also, without the value of DD1-3-S-B, ultimate strengths of the all specimens computed at the interfaces were larger than theoretical predictions. It means that through enough development length of loop bars, sufficient flexural performances of the specimens were secured. Though the DD1-3-S-B specimen with a high strength concrete joint had a shear failure as shown in Fig. 2, except for it, failure modes of all specimens were flexural failures as shown in Fig. 3.

The effects of the interface treatment were not clear for concrete bond adhesive but the chipping gave greater strength than that of the specimen with steel brushing. From this observation, the minimum requirement of the interface treatment was recommended to expose coarse aggregates by sand or water blasting after precasting.

Table 3. Comparison between and experimental and theoretical ultimate strengths

Specimen	Ultimate strength			Failure mode
	Experimental (kN)	**Theoretical (Location) (kN)**	**Experimental/ Theoretical**	
DD1-3-S-A	228	132 (M) / 162 (I)	1.73 (M) /1.41 (I)	Flexural
DD1-3-S-B	227	189 (M) / 228 (I)	1.20 (M) / 0.99 (I)	Shear/Flexural
DD2-1-S	361	341 (I)	1.05 (I)	Flexural
DD2-1-C	390	341 (I)	1.14 (I)	Flexural
DD2-1-B	343	341 (I)	1.00 (I)	Flexural
DD2-3-S-A	245	132 (M) / 171 (I)	1.85 (M) / 1.43 (I)	Flexural
DD2-3-S-B	346	189 (M) / 241 (I)	1.83 (M) / 1.44 (I)	Flexural
DD3-3-S-A	218	132 (M) / 181 (I)	1.65 (M) / 1.20 (I)	Flexural
DD3-3-S-B	339	189 (M) / 255 (I)	1.79 (M) / 1.33 (I)	Flexural
DD4-1-S	368	270 (M)	1.36 (M)	Flexural
DD4-1-C	429	270 (M)	1.59 (M)	Flexural
DD4-1-B	395	270 (M)	1.46 (M)	Flexural

* (M) : Calculated at mid-span (I) : Calculated at interface



(a) Flexural failure (DD2-1-S specimen) (b) Shear failure (DD1-3-S-B specimen)

Figure 2. Failure modes of specimens

Cracking

The first cracking strengths were compared with theoretical cracking strengths in Table 4.

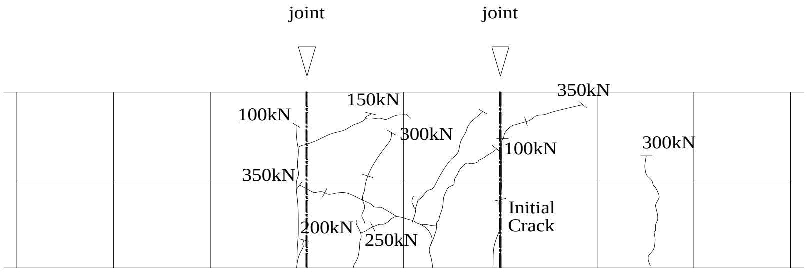
Calculated cracking strengths were computed based on interfaces between sections of precast member and cast-in-place concrete, because the initial cracks of all specimen always were occurred near the interface sections.

Without DD2-1-S and DD5-2-S specimen, specimens with normal strength concrete joints had an initial crack prior to the experiments during delivery and erection processes. It shows difficulty of quality control of interface between precast concrete and cast-in-place concrete. Moreover, initial crack of DD2-1-s specimen was occurred by 35 kN applied load, which was is 44% of theoretical value of 101 kN. In cases of specimens adjusting high strength concrete for joint parts, the initial cracks were observed between 75 kN and 95 kN loading. These values are almost over 1.5 times bigger than predicted values by calculations.

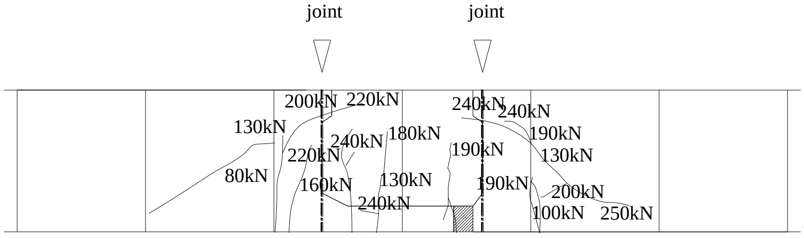
Table 4. Comparison between and experimental and theoretical cracking strengths

Specimen	Cracking strength		
	Experimental (kN)	**Theoretical (kN)**	**Experimental/ Theoretical**
DD1-3-S-A	82	47	1.74
DD1-3-S-B	78	47	1.66
DD2-1-S	35	101	0.44
DD2-1-C	Existing	101	-
DD2-1-B	Existing	101	-
DD2-3-S-A	95	50	1.9
DD2-3-S-B	80	50	1.6
DD3-3-S-A	105	53	1.98
DD3-3-S-B	105	53	1.98
DD4-1-S	Existing	226	-
DD4-1-C	Existing	226	-
DD4-1-B	Existing	226	-

Cracking patterns in precast member is important for estimation of overall performance of the structure. Vertical flexural crack were observed near interfaces of all specimens and some diagonal cracks were also distributed as shown in Fig. 3. Though the first crack was occurred at the joint, all the specimens did not show concentrated cracking at the joint part.



(a) DD2-1-B specimen

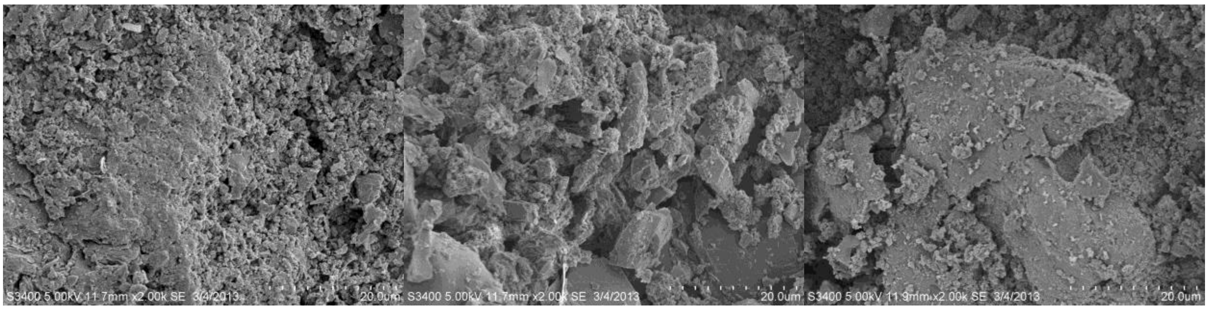


(b) DD3-3-S-A specimen

Figure 3. Cracking patterns of specimens

Eurocode-2 suggests provision of minimum reinforcements to restrain crack width by 0.3 mm in service limit state, and this value is 1.5 times greater than average crack width [13]. At the point of the initial crack, crack width was reached to 0.2 mm earlier than mid-span which is the maximum moment area. This clearly indicates that the flexural crack was developed in higher load level than the joint crack. Current design codes on direct crack width control rely on steel stress of the reinforcement. For the precast members with loop joints, the crack width should be estimated at the joint location instead of center span.

Three figures in Fig. 4 were captured after cracking of the joints by SEM (Scanning electron microscope) that produced images by scanning with 50,000 times magnification. Fig. 4 (a) and (b) show the surfaces of specimens with treatment of steel brushing and chipping, respectively. It shows that the surface treatments such as chipping and steel brushing can help to increase the bonding surface between old and new concrete. The particles of surface concrete by brushing are smaller than chipping in the images of same scale. Fig. 4 (c) shows the surface of concrete with coating by bonding adhesive, it presents the cohesion between large and small particles of surface concrete. From this observation, it is important to remove weak phase of the interface to reduce premature cracking of the joints in precast structures.



(a) Steel brushing (b) Chipping (c) Coating with bond adhesive

Figure 4. Microscopic images of interfaces by SEM

Conclusion

The flexural behaviour of precast members with loop joints with two types of pouring concrete and different surface conditions was investigated by static tests. Comparisons between theoretical calculation and test results based on the measured data were executed to verify the applicability of current design codes to precast members with loop joints. Through the experimental program, the following conclusions were derived.

1. For the strength of the precast members connected by loop joints, current design requirements on development length of the reinforcement gave enough flexural strength.
2. Failure modes of all specimens with high-strength concrete in the joints showed flexural failure. The loop joint using high strength concrete showed possibility to reduce the development length requirement to achieve the same flexural strength as that of a precast deck.
3. The initial cracks of specimens used normal concrete joints were observed before the tests. It showed that the cracking can be occurred in low loading condition when low strength joint material was utilized.
4. Every initial crack of tested specimens was occurred at interface section. In addition, crack widths measured at location of initial crack were reached to 0.2 mm crack width earlier than flexural cracks at the mid-span. Thus, performance of specimen for crack width control in precast structures with joints was governed by joint cracks.

5. Tested cracking loads of specimens using high strength concrete for cast-in-pace concrete were almost 1.5 times bigger than the computations of cracking strength.
6. These results indicated that initial crack control can be achieved when the high-strength concrete was used for joint parts, whereas it was difficult when the normal strength concrete was used.
7. The effects of the interface treatment were not clear for concrete bond adhesive but the chipping gave greater strength than that of the specimen with steel brushing. Therefore, it is recommended to expose coarse aggregates by sand or water blasting after precasting.
8. For the serviceability check of the precast structures, the loop joints with required development length and surface treatments provided conservative cracking loads.
9. Structural performance of the joint in precast concrete structures is crucial for the durability. Therefore, it is recommended to remove weak phase of the interface to reduce premature cracking of the joints in precast structures.

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