Structural and Construction Verification of an Ultra-High Durable Deck Slab to Girder Joint

超高耐久床版と鋼桁の接合部の構造的性能及び施工性の検証

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昨今，大規模更新事業の床版取替え工事が多く取り組まれているが，将来の維持管理を考慮した場合，できるだけ耐久性の高い床版構造が望まれる。そこで，鉄筋や PC 鋼材などの腐食による劣化の原因となる鋼材を一切使用しない超高耐久床版を開発した。超高耐久床版と鋼桁の接合構造は、ずれ止め孔まわりに補強筋を設置せず、床版の耐久性をさらに高めるためにずれ止め孔を床版天端まで開けずに床版下端の箱抜きとした。本接合構造について、せん断耐力の確認試験を実施した結果，一般的な PC 床版の接合構造と同等以上のせん断耐力を保有し，超高耐久床版の接合構造として使用できることを確認した。

キーワード：耐久性，無収縮モルタル，超高耐久性床版，接合部，スタッド

Considering the increased road bridge deck replacement projects in Japan recently, an ultra-high durable slab “Dura-Slab” which does not utilize any steel components had developed. A joint system between Dura-Slab and steel plate girders was proposed targeting to avoid deck penetrating openings and additional reinforcement in the deck around the joint. The usability of steel shear studs, holed steel angles and steel bolts in the joint including the effect of deck prestressing was experimentally studied. Structural behavior was evaluated against a conventional joint and the proposed joints showed a satisfactory behavior and considered to be applicable to real structures.

Key Words: Durability, No-shrinkage mortar, Dura-Slab, Joint, Stud

1. INTRODUCTION

Recently, deterioration of RC deck slabs in highway bridges has become a major problem in Japan. In numbers of cases, deck slab replacement is essential to secure the expected serviceability. A major cause of the deterioration can be identified as the corrosion of steel reinforcement which especially in snowy areas is caused by deicing agents and in coastal areas due to airborne chloride. It is well known that the reinforced concrete structures including deck slabs require a particular level of maintenance during the service life. However, the reduction of man power with aging population and increasing maintenance and renewal costs urge the promotion of ultra-durable infrastructure which will reduce the lifecycle cost as well as the overall environmental impact.

Ultra-High Durable Slab (referred as “Dura-Slab” afterwards), a new form of pre-stressed concrete bridge deck slab was introduced by the authors[1)] as a viable solution to deck slab replacement projects in plate girder bridges. It is made of fiber reinforced concrete, Aramid Fiber Reinforced Polymer (AFRP) rods as pre-stressing tendons, and does not contain any form of steel reinforcement. The conventional girder-slab connection relies on the steel reinforcement in the deck slab to counteract the concentrated stresses. Since Dura-Slab does not contain reinforcing bars, further investigation was required regarding the deck-girder joint. A new type of joint was proposed and an experimental verification was carried out to understand the constructability and the structural joint performance.

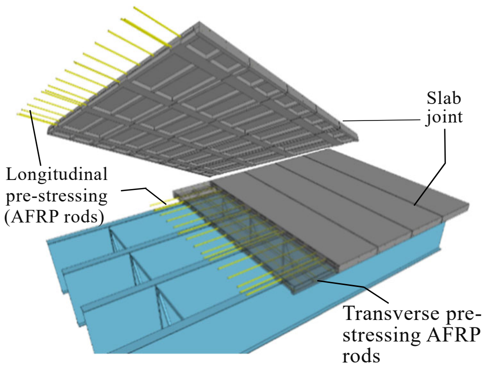
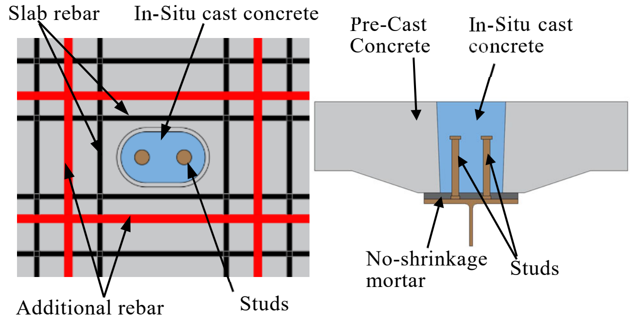


Fig. 1. Dura-Slab structure



(a) Plan-Additional reinforcement around the joint　(b) Cross section

Fig. 2. A Conventional slab-girder joint

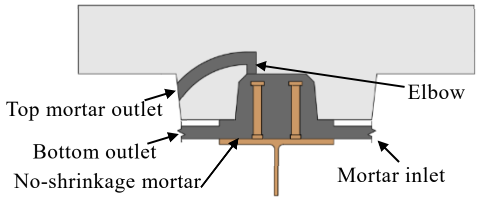


Fig. 3. Typical cross section of proposed joints

1.1 INTRODUCTION TO "Dura-Slab"

As shown in **Fig. 1**, Dura-Slab is a ribbed slab made of high strength fiber reinforced (vinyl fiber) concrete with 80N/mm^2 design compressive strength (f_{cu}). Prestressing tendons made of AFRP rods are utilized instead of steel tendons which reduces the risk of corrosion damage. To gain faster construction speeds and higher quality, it is common practice to utilize precast deck slabs in bridge deck replacement projects. Hence, Dura-Slab is made as pre-tensioned precast panels.

2. THE SLAB-GIRDER CONNECTION METHOD OF NON-COMPOSITE PLATE GIRDER BRIDGES

The most common form of the conventional connection between the steel girders and precast concrete deck slabs is constructed with shear-studs. An example of a conventional joint is shown in **Fig. 2**. Openings are provided in advance in the precast concrete deck panel in joint locations. After the deck panel is placed on the steel girder, studs are welded to the girder from the opening and the openings are sealed with no-shrinkage mortar and concrete. The slab panel is additionally reinforced as shown in **Fig. 2**, around the opening.

The water seepage through construction joints might aggravate slab deterioration by steel corrosion[2]. Since the conventional deck-girder connection includes penetrating holes which are filled by concrete, the joints formed after concrete fillings might lead to water leakage with time. Regarding the current development, it was decided to eliminate the penetrating construction joints at the slab-girder connection. The stud version of the proposed joint is shown in **Fig .3**. Two alternatives with holed steel angles and high strength bolts were experimented as well.

Non-deck penetrating openings and mortar outlet hoses for new joints are provided in advance in the precast panels. Mortar inlet hoses are installed at site. The openings will be sealed only by injecting high strength mortar instead of using concrete.

3. PERFORMANCE VERIFICATION OF NEW JOINTS

Applicability of the newly proposed joints was investigated by comparing the behavior with a conventional joint. The standard double shear pushout test was carried out according to the "Pushout Test Method for Headed Studs and the Latest Status of Research Related to Shear Studs" (In Japanese)[3] by Japanese Society of Steel Construction.

3.1 TEST CASES AND SPECIMENS

Test cases and specimen compositions are shown in **Fig. 4(a)** and **4(b)**. Case-0 represents the control specimen with a conventional joint. Case-1 to Case-6 represents the proposed joints with high strength no-shrinkage mortar and fiber reinforced concrete. Cases shown in **Fig. 4(a)** are made with shear studs where Case-1 represents the standard new studded joint. In Case-2, transverse deck prestressing is applied and in Case-3, GFRP rod reinforcement was introduced. Shown in **Fig. 4(b)**, Case-5 consists of joints made of holed angle sections while Case-6 consists of joints made by high-strength bolts connected to ceramic inserts embedded in the slab. Specimens was constructed as closely

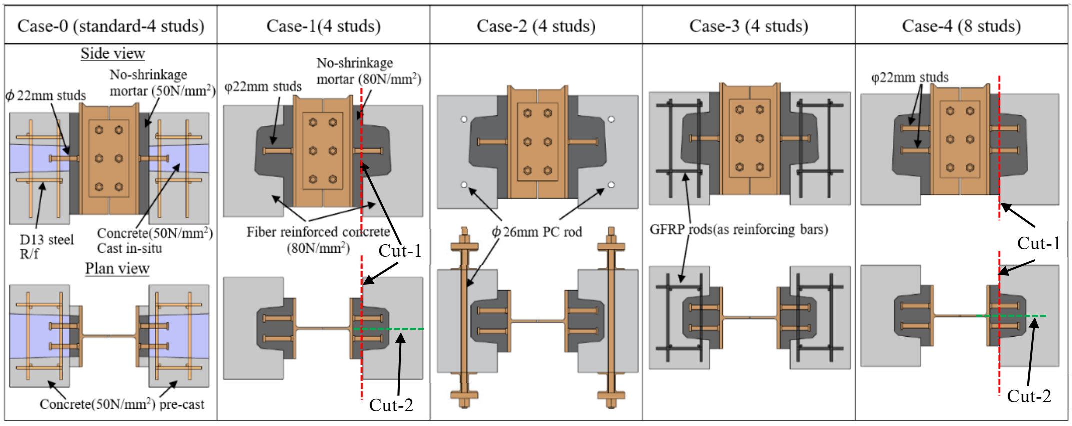


Fig. 4. (a) Specimens with shear studs

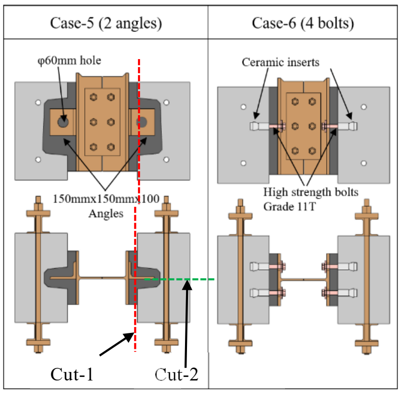


Fig. 4. (b) Specimens with angle and bolts

(a) precast concrete block (b) block placing

Fig. 5. Specimen making (Case-4)

as possible to a real structure. Pre-cast concrete block of a Case-4 specimen and placing it on the loading girder are shown in **Fig. 5**.

Each test case consisted of 3 specimens (named as 1, 2, 3 e.g. first specimen of Case-0 is named as "0-1"). Two specimens (1, 2) were loaded monotonically while one specimen (3) was loaded with incremental cyclic loading (load controlled with 40kN steps until a relative displacement of 1mm and then displacement controlled in 0.5mm steps until 4mm).

3.2 EXPERIMENTAL SETUP AND MEASUREMENTS

Typical specimen dimensions and loading method are shown in **Fig. 6**. Steel plate supports were installed to simulate the continuity of mortar layer in real structures. Supports were not installed in Case-1-1, Case-2 and Case-3. The major measurement was the load-relative displacement curve between the concrete block and the loading girder at the level of the studs.

3.3 CONSTRUCTION QUALITY INVESTIGATION

All of the proposed joint variations require mortar injection below the deck. The mortar injection quality is not possible to be visually inspected in real construction. Hence, the injection quality of specimens was investigated to verify the suitability of the proposed construction method. A single side of several selected specimens, 1-1, 1-3, 4-3, 5-3, were cut along the lines shown in **Fig.4** for inspection. The objective of the Cut-1 was to inspect the deformed shape of the studs while Cut-2 was made to observe the mortar injection quality.

4. EXPERIMENTAL RESULTS

Load-displacement curves were normalized based on the number of studs in a specimen as recommended in the standard testing procedure[3)]. Similarly, in Case-5 and 6 the load was normalized to the number of angles or bolts in a specimen. The normalized curves under monotonic loading for stud joints are shown in **Fig. 7** (Cases with lower load capacity) and important experimental results, maximum load, yield load and displacement coefficient, which represents the joint stiffness, per stud of each specimen are shown in **Table 1**. The latter two parameters were calculated based on the guidelines[3)].

According to **Fig. 7**, in the cases without the steel plate supports, there was a sudden load drop after reaching a maximum value. Accordingly, there was a reduction in maximum and yield loads as shown in **Table 1**. However, in

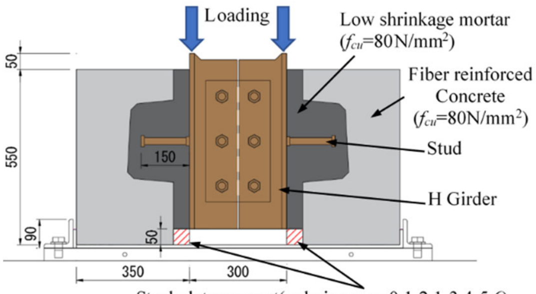


Fig. 6. Experimental setup

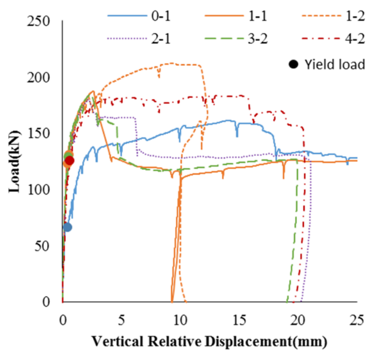


Fig. 7. Load-Displacement behavior (Load per stud)

Table 1. Experimental results

Specimen	Provison of steel plate support	Maximum load Q_{max} (kN)	Yield load Q_y (kN)	Displacement coefficient K_{st}° (kN/mm)
0-1	Yes	161.0	66.8	327.3
0-2	Yes	172.9	58.1	312.5
0-3	Yes	161.5	65.4	327.3
1-1	No	187.3	123.5	495.6
1-2	Yes	212.3	131.3	467.7
1-3	Yes	219.1	139.5	255.3
2-1	No	180.2	126.3	460.9
2-2	No	163.3	133.8	492.5
2-3	No	158.3	129.7	459.5
3-1	No	184.1	125.3	387.0
3-2	No	184.6	118.3	527.8
3-3	No	151.0	129.0	530.5
4-1	Yes	191.9	125.0	299.8
4-2	Yes	183.0	125.5	308.0
4-3	Yes	189.6	130.1	248.2
5-1	Yes	813.8	435.0	831.2
5-2	Yes	844.9	483.5	1166.3
5-3	Yes	788.7	444.2	1012.4
6-1	Yes	274.7	120.8	61.8
6-2	Yes	282.7	114.6	91.1
6-3	Yes	298.5	117.1	67.0

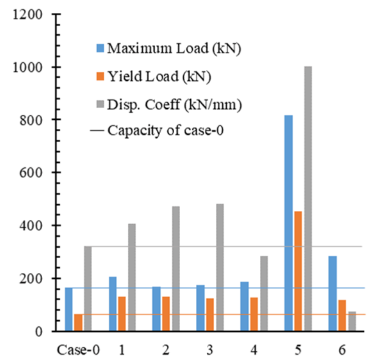


Fig. 8. Experimental results – Average value of each case

Case 1-1 without the support, load-displacement behavior up to the sudden load drop was not significantly different from Case1-2 with the support. Additionally, considering the fact that in a real structure the mortar layer is continuous, the cases with the support may have grasped the real structural behavior. Hence, the specimens without supports may underestimate the real load capacity leading to a conservative design. Therefore, the difference of provision and non-provision of the steel plate supports was considered to be negligible for the purpose of the current study.

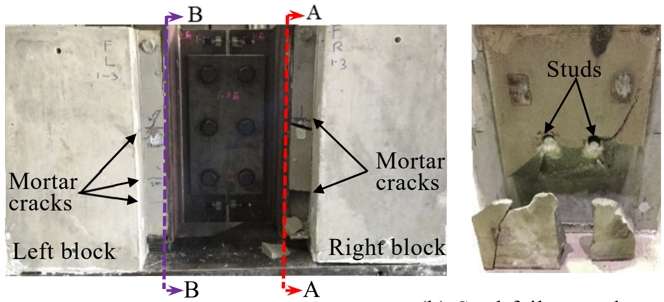
Third specimen of each case was loaded with incremental repetitive loading. According to the **Table 1**, it was observed that the behavior did not drastically change based on the loading method. In several cases such as 1-3, 3-1, 4-1 and 5-1 the displacement coefficient was relatively low compared to the other specimens in each case. This was due to the slight difference in time of crack appearance and displacement due to cracking.

The average (of three specimens per case) experimental results are shown in **Fig. 8**. Average maximum loads and the yield loads of Case-1 to Case-6 were more than in Case-0. Case-5 with the angle showed a significantly higher maximum and yield load due to larger dowel size and in Case-6 with bolts higher maximum load was due to higher bolt failure strength (1,040N/mm² against 440N/mm² in studs).

The displacement coefficient value in Case-4 was lower than the Case-1. The reduction may be due to the behavior interaction of closely placed studs. Due to the smaller displacement coefficient in Case-6, bolted joint was considered as not suitable for real applications without further improvements. The reduction may have occurred due to the extra space in bolt holes in flanges and between the ceramic insert wall and stud.

The behavior of the specimens in Cases-1, 2, 3 was similar irrespective of the provision of pre-stressing in Case-2 and reinforcement in Case-3. The reason is explained by the failure mode. The failure mode of specimen 1-3 is shown in **Fig. 9**. In all the specimens in Cases-1, 2 and 3, severe cracking was observed in the injected mortar without cracking in the concrete blocks. As the failure is governed by mortar failure, the applied pre-stressing and reinforcement may not have significantly contributed to the load-deformation behavior. Whereas, concrete cracks were observed in several specimens of Cases-4, 5 and 6.

The Cut-1 is shown as section B-B in **Fig. 9** and the



(a) Crack pattern (b) Stud failure and mortar failure (section A-A)

Fig. 9. Failure mode of specimen 1-3

(a) specimen 0-2 (b) specimen 1-3

Fig. 10. Deformation of studs

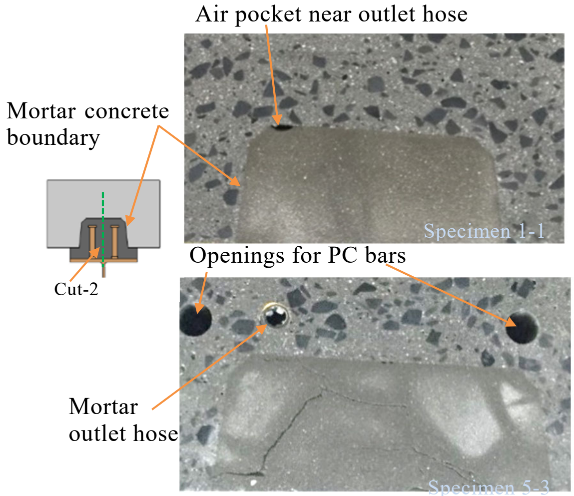


Fig.11. Cutting face of specimen 1-1, 5-3

observed stud deformation is shown in **Fig. 10**. Based on the stud deformation pattern, it was observed that the proposed stud joints behaved shear dominantly while the conventional joint showed a tensile dominant behavior. This phenomena may have caused the lower displacement of Case-1 at maximum load as shown in **Fig. 7** .

Based on the experimental results, the stud joints was considered to be the most desirable because the angled joint might require more construction effort due to significantly increased weld lengths. Bolted joint was considered to require further improvements to be applied in a real structure.

4.1 INVESTIGATION OF THE CONSTRUCTION QUALITY

Specimens, 1-1, 1-3, 4-3, 5-3, were cut to inspect the mortar injection quality. A single side of the observed surfaces after executing the Cut-2 in specimen 1-1 and 5-3 are shown in **Fig. 11**. It was observed that small air pockets were remained near the top mortar outlet hose in specimen 1-1 and 4-3. In the other two specimens this condition was not observed. However, in specimen 5-3 it was observed that air has stuck on a portion of the mortar outlet hose.

Since not all specimens had air pockets near the top outlet hose, it was considered that formation of this air pocket can be prevented by careful construction and properly pressurizing mortar before sealing the outlet hose. The air trapping in the outlet hose might be able to control by slightly angling the outlet hose inside concrete instead of using the elbow as shown in **Fig. 3**.

5. JOINT SAFETY FACTOR AGAINST DESIGN GUIDELINES

Design load capacity of the studded joints was calculated according to Specification for Highway Bridges [4)] by Japan Road Association(JARA) guidelines and Standard Specification for Composite Structures[5)] by Japanese Society of Civil Engineers(JSCE). The factor of safety of the joint specimens was evaluated as the ratio between the experimental load capacity and the calculated design load capacity.

5.1 DESIGN LOAD CALCULATION

Allowable design load on a stud (Q_a) can be calculated according to equation (1) when height to diameter ratio exceeds 5.5 (6.8 in current experiment) based on JARA guidelines. Stud diameter is denoted by d and σ_{ck} is the design compressive strength of concrete (80N/mm^2).

$$Q_a = 9.4d^2\sqrt{\sigma_{ck}} \tag{1}$$

$$V_{ssud} = 31A_{ss}\sqrt{(h_{ss}/d_{ss})f'_{cd}} + 10000 \tag{2}$$

$$V_{ssud} = A_{ss}f_{ssud} \tag{3}$$

Ultimate load capacity of a stud (V_{ssud}) is given by the least value defined by equations (2) and (3) according to JSCE standards. The design load capacity is obtained as $0.5V_{ssud}$. The quantities defined by A_{ss}, h_{ss}, d_{ss}, f'_{cd}, f_{ssud}, are

Table 2. Factor of safety

Case	Experimental		JARA			JSCE	
	Qy	Qu	Qd	$FOS_{(QY)}$	$FOS_{(Qu)}$	Qd	$FOS_{(Qu)}$
C-0	63.4	165.1	32.2	2.0	5.1	76.0	2.2
C-1	131.4	206.2	40.7	3.2	5.1	76.0	2.7
C-2	129.9	167.3	40.7	3.2	4.1	76.0	2.2
C-3	124.2	173.2	40.7	3.1	4.3	76.0	2.3
C-4	126.9	188.2	40.7	3.1	4.6	76.0	2.5

Qy, Qu, Qd are in kN, all values are given per stud

stud cross sectional area, stud height, stud diameter, compressive strength of concrete and stud strength (400N/mm^2). Compressive strength of concrete was used as 50N/mm^2 based on the scope of equation (2). The design joint load capacity was governed by equation (3) for all the specimens irrespective of used compressive strength value.

5.2 CALCULATION RESULTS

Factor of safety calculation results are shown in **Table 2** where Qy, Qu, and Qd denote experimental average yield load, experimental average ultimate load and calculated design load. $FOS_{(Qy)}$ and $FOS_{(Qu)}$ respectively denote the factor of safety against experimental yield load and factor of safety against experimental ultimate load.

The design load capacity calculated from JSCE equations results in ultimate load capacity. Therefore, the factor of safety against yield load was not evaluated.

The $FOS_{(Qy)}$ of proposed joints according to JARA standards was above 3.0 and was greater than the standard specimen. However, $FOS_{(Qu)}$ was smaller than the standard test case in Cases-2,3 due to the lack of continuity due to absence of steel plate supports which might increase to same level as Case-1 with continuous mortar layers in real structures. In Case-4, reduction might be due to the group action of studs. According to JSCE calculation method, all the proposed joints showed a higher safety factor than the standard specimen. JARA standards may be used to estimate the design strength of proposed stud joints when the joints consists of only two studs based on results of Case-1.

6. CONCLUSIONS

A new type of joint was proposed between ultra-high durable slab, Dura-Slab, and steel girders targeting to remove the deck penetrating holes in slab. An experimental study was carried out to investigate the structural performance of the new joint system and suitability of using shear studs, holed angle steel sections and bolts as well as the requirement of additional deck reinforcement near the joint and effect of deck transverse pre-stressing on the joints were investigated based on joint load capacity, yield load and displacement coefficient.

The performance of newly proposed joints except of the bolted joint showed a superior behavior with respect to a conventional joint. The failure mode of the new joint was not governed by the concrete failure, hence the provision of additional reinforcement deemed unnecessary while the effect of transverse prestressing may not affect the joint load capacity. The construction quality of the joint was investigated by cutting several selected specimens and the quality was considered to be at an acceptable level with some room for improvement in later stages.

Factor of safety of the joints was evaluated based on JARA and JSCE design specifications and showed an ample factor of safety against design load. Considering the margin of safety, JARA standards may be used to calculate the design capacity of newly proposed stud joints with only two studs.

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