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An Investigation of the Shear Design of Bolted Shear Key Joint in Partially Filled Steel Composite Decking System

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Abstract:- The joints of the partially filled steel grid composite deck, which can be precast, may be designed as mechanical connections formed by concrete shear keys and bolts. This paper examines the results of push-out tests and of tests on the deck by eccentric loading in order to apply this mechanical joint to the partially filled steel grid composite deck. The shear resistance of the joint calculated by a design formula based upon the shear friction theory is compared to the push-out test results, and the shear performance of the joint in the deck structure as well as the design safety are examined. The analysis results reveal that the joint by mechanical connection can be designed by the LRFD design formula, and that the shear resistance of the joint is closely related to the resistance of the concrete shear key rather than to the resistance provided by the bolts. The evaluation of the static safety of the joint structure shows that the joint by mechanical connection develops sufficient shear resistance. Further studies on the structural behavior of the joint under other loading conditions are necessary to achieve its optimal design.

Keywords:- partially filled steel grid composite deck, shear key, joint, push-out test, bending test

I. INTRODUCTION

The partially filled steel grid composite deck is a composite deck made by composing a concrete slab and a steel grid (Fig. 1). The steel grid itself is formed by the combination of T-beams taking charge of the flexure-tension and cross bars disposed perpendicularly to the beams. Sometimes, longitudinal bars are installed perpendicularly to the cross bars and between the T-beams to strengthen the grid. The composition of the steel grid and concrete slab is achieved by means of shear connectors disposed at the top of the T-beams. After its introduction in 1930s, the partially filled steel grid composite deck has continuously been the subject of various studies related to the change in its structural behavior according to the sectional details and member composition, and the application of lightweight and high performance materials [1-5].

Since the partially filled steel grid composite deck can be precast, the joint structure composed of concrete shear keys and bolts (Fig. 2) was proposed recently for further application to horizontal load bearing structures like bridges or platforms [6]. This type of joint does not necessitate separate arrangement of rebar nor placing of filling concrete, which enables to assemble the whole horizontal load bearing structure using precast segments more economically and efficiently.

Push-out test and test on the deck by eccentric loading were conducted to apply such mechanical connection to the partially filled steel grid composite deck [6-8]. However, need is to consider collectively the experimental results obtained through these tests in order to apply the new type of joint. Concretely, the support conditions and the load transfer mechanism of the actual deck structure differ from those of the tests at the member level, which means that the results obtained from push-out test only are insufficient to validate the design safety and examine the shear performance of the bolted shear key joint in the actual deck structure.

Accordingly, this paper examines synthetically the results of the push-out test and of the bending test of the deck by eccentric loading so as to verify the design safety of the joint by mechanical connection, which in turn will verify its applicability to prefabricated decks like the partially filled steel grid composite deck. To that goal, the shear resistance of the deck joint is calculated using the design formula determined from the push-out test and the analysis of the corresponding results. Moreover, the load transfer behavior of the joint is investigated by analyzing the results of the test on the deck, and the shear force likely to be sustained by the joint in the actual deck is estimated. Thorough analysis of the load transfer behavior of the joint is performed with regard to the actual deck structure and not at the member level, and comparison is done between the shear resistance of the joint and the shear force that it must sustain in reality so as to be able to examine the design safety to shear and the applicability of the joint by mechanical connection.

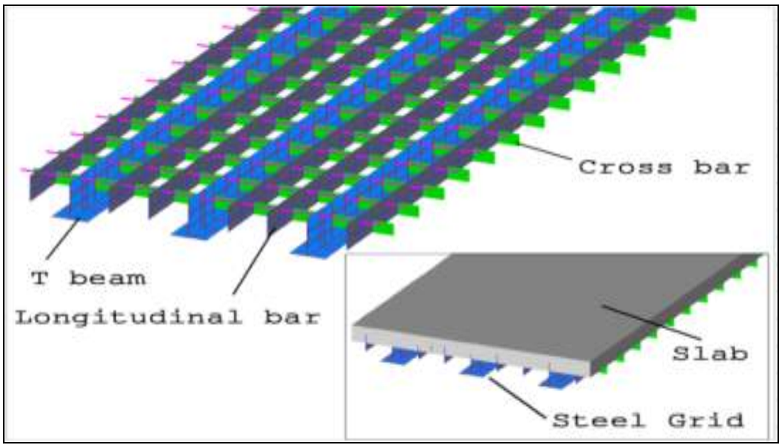


Fig. 1: Partially filled steel grid composite deck structure [6]

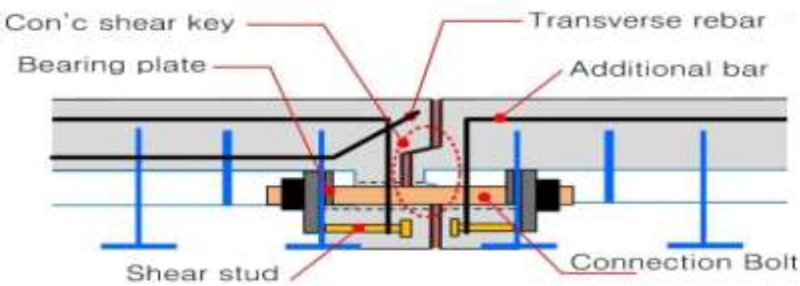


Fig. 2: Details of joint between precast decks [6]

II. EVALUATION OF SHEAR RESISTANCE BY PUSH-OUT TEST

A. Design formula for shear resistance

If crack occurs in the direction of the shear force working on the reinforced concrete member, resistance to shear is developed through the effect of the reinforcement crossing the cracks and the frictional action at the cracked surface. Such shear reinforcement is designed based upon the shear friction theory depicted in Fig. 3 and expressed in Eq. (1) [9,10].

As shown in Fig. 3, the shear resistance of the joint is calculated using the design formula and theoretical formula based on the shear friction theory, and the comparison with the results obtained from push-out test is given hereafter [6,7].

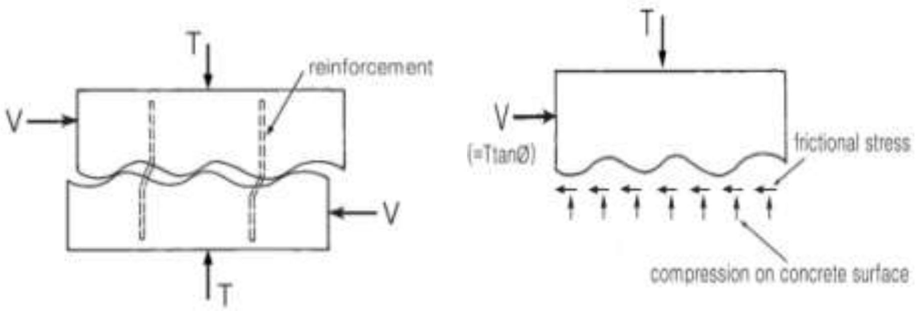


Fig. 3: Shear friction theory [11]

$$V_n = T\tan\phi = A_{vf} f_y \tan\phi = \mu A_{vf} f_y \tag{1}$$

where A_{vf} = amount of shear reinforcement; f_y = yield strength of shear reinforcement; and, tan Ø = μ = coefficient of shear friction.

Eqs. (2) to (6) express respectively the formulae computing the shear strength suggested by Birkeland [12], Mattock [13] and Walraven [14], and the design formulae proposed in ACI-318 [15] and AASHTO LRFD [16].

$$v = 2.78\sqrt{\rho f_y} \tag{2}$$

where v = shear stress; ρ = reinforcement ratio; and, f_y = yield strength of the reinforcement.

$$v = 0.467 f_c^{0.545} + 0.8\left(\rho f_y + \sigma_n\right) \leq 0.3 f_c \tag{3}$$

where f_c = compressive strength of concrete; and, σ_n = compressive stress acting normally to the shear surface.

$$v = 0.85 c_1 \left(\rho f_y\right)^{c_2},\ c_1 = f_c^{0.36},\ c_2 = 0.09 f_c^{0.46} \tag{4}$$

$$V_n = \mu A_{vf} f_y \leq \min(0.2 f_c A_c, 5.52 A_c) \tag{5}$$

$$V_n = c A_c + \mu\left(A_{vf} f_y + P_c\right) \leq \min(0.25 f_c A_c, 10.3 A_c) \tag{6}$$

where V_n = design shear force; μ = coefficient of shear friction (= 1.4 for concrete placed at once); A_{vf} = amount of reinforcement; and, A_c = cross sectional area of concrete subjected to shear. In Eq. (6), the constant c = 2.8 MPa for normal concrete; and, P_c = compressive force acting normally to the surface subjected to shear.

B. Push-out test results

The concrete shear key is applied in all kinds of joints in precast concrete structures. This joint is also applied for deck structures as shown in the drawing of Fig. 2. This study reexamines the shear resistance of the joint structure by mechanical connection shown in Fig. 2 and composed of bolts and the concrete shear key in the male part of the joint.

The entire structure of the push-out test specimen for the evaluation of the shear resistance of the mechanical joint composed of a concrete shear key and bolts is shown in Fig. 4. The concrete member of which core is loaded and the concrete blocks attached at both sides are fastened monolithically by means of the concrete shear key and bolts. Epoxy is applied between the male part and female part of the concrete shear key so as to fasten them tightly without gap. Moreover, the concrete interface was surface-treated by water-jet in advance to increase the bond strength of the epoxy. The design strength of concrete is 35 MPa, the diameter of the bolts is 27 mm, and the yield strength of the bolts is 640 MPa [6,7].

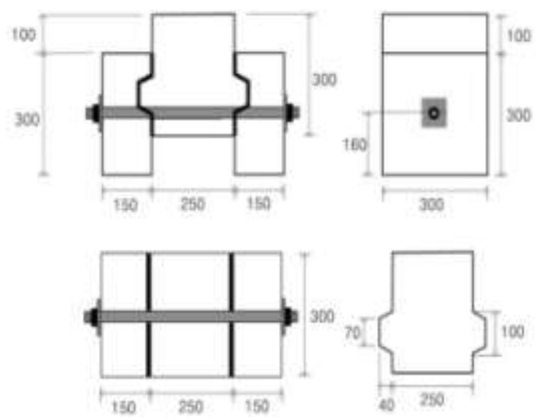


Fig. 4: push-out test specimen [6,7**]**

Fig. 5: Push-out test [6]

The test results are arranged in Table 1. In Table 1, P_{ic} is the initial crack load, P_{max} is the maximum applied load, $P_{max,\ mean}$ is the average of P_{max}, and δ_{max} is the slip(relative displacement, sliding) occurring between the core concrete member and the concrete blocks at both sides at P_{max}. Fig. 6 plots the load-slip curves measured during the push-out test.

The shear resistance of the bolted shear key joint obtained from the test is compared in Table 2 to the values given by the shear strength formulae of Birkeland [12], Mattock [13] and Walraven [14], and the values provided by the design formulae of ACI-318 [15] and AASHTO LRFD [16]. The experimental shear force V_{exp}

corresponds to half of the loading applied in the test as the force sustained by one joint. Considering that the size of the maximum shear force obtained from the test is slightly larger per specimen, V_{exp} uses the minimum value among the three values measured in the test.

Table 1: Results of push-out test.

Specimen	P_{ic} (kN)	P_{max} (kN)	$P_{max,\ mean}$ (kN)	δ_{max} (mm)
PT-BCB1	477.6	1033.4	1040.3	1.3
PT-BCB2	380.8	831.7		2.7
PT-BCB3	416.5	1255.9		0.7

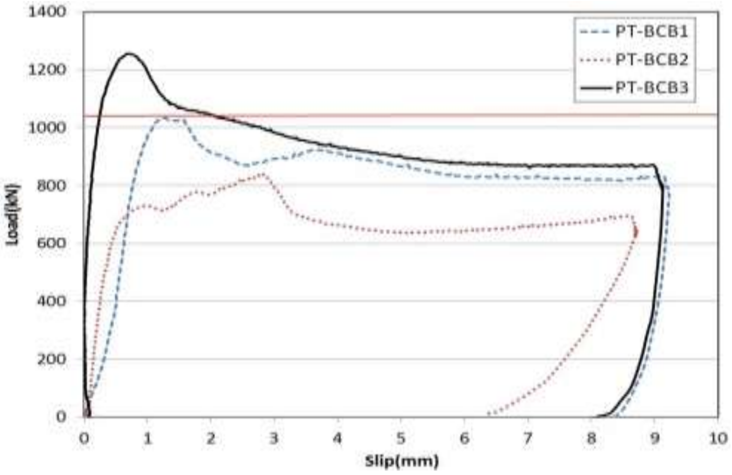


Fig. 6: Experimental load-slip curves [6,7]

Table 2: Comparison of shear resistance [6,7]

Specimen	**Shear resistance (kN)**					
	Experiment	**Empirical**			**Design**	
	V_{exp} 1)	V_B 2)	V_M 3)	V_W 4)	V_{nA} 5)	V_{nL} 6)
PT-BCB	415.9	233.4 (1.78) 7)	252.0 (1.65)	233.4 (1.78)	132.5 (3.14)	210.0 (1.98)

1) Shear resistance from test ($V_{exp} = 0.5P_{max}$)
2)~4) Shear resistance equation by Birkeland (Eq. 2), Mattock (Eq. 3) and Walraven (Eq. 4)
5)~6) Nominal shear force by ACI-318 (Eq. 5) and LRFD (Eq. 6)
7) V_{exp}/V_B

Compared to the design formula of ACI-318, the experimental shear resistance appears to be about 3.14 times larger than the experimental value. This indicates that ACI-318 underestimates the shear resistance of the joint than it is in reality. Besides, Birkeland, Mattock, Walraven and LRFD underestimate the shear resistance by approximately 1.8 times. These results reveal that the equation of LFRD is more appropriate than that of ACI-318 as design formula for the shear resistance of the joint considered in this study. Accordingly, the design formula of LRFD is used for the safety check of the joint with regard to the test of the deck.

III. EVALUATION OF THE STRUCTURAL BEHAVIOUR OF THE JOINT BY TEST ON DECK

A. Background and summary of test

The shear resistance of the deck joint is closely related to the performance in the transverse transfer of the live load applied through the external force. The joint shall be designed to develop sufficient resistance for the transverse transfer of the load until the occurrence of the bending failure or punching failure of the deck itself.

The shear performance of the deck joint can be predicted to some extent through the analysis of the push-out test results. However, the support conditions and load transfer mechanism in the push-out test differ from those of the actual deck structure. This means that the results obtained from push-out test only are insufficient to validate the design safety and examine the shear performance of the joint in the actual deck structure. The following presents the results of the bending test of the deck by eccentric loading [6,8] and the analysis of these results.

B. Bending test of deck by eccentric loading

(1) Test method

Eccentric load was applied on a simply supported deck with span length of 2.5 m and total width of 2.0 m. The joint structure installed in the deck is shown in Fig. 2 and Fig. 7. Table 3 arranges the designation of the specimens of which test variable is the number of bolts installed in the joint. Specimen JD9B has 9 bolts installed at spacing of 30 cm and specimen JD4B has 4 bolts installed at spacing of 60 cm. In addition, specimen UMD is the reference unit deck with span length of 2.5 m and width of 1.0 m for the comparison of the transverse load transfer [6].

Strain gages were attached at the bottom of the T-beams as shown in Fig. 8 to measure the transverse distribution of the strain at mid-span of the deck. Displacement sensors(LVDT) were also installed at the quarters and at mid-span of the deck to obtain the deflection of the deck. Loading was applied as a concentrated load on one side of the deck to examine the load transfer behavior and shear performance of the joint.

Table 3: Designation and characteristics of specimens

Specimen	Joint type	Test no.	Loading method
UMD	–	1	3-point
JD9B-E	Concrete shear key + 9 bolts (@300)	1	Eccentric
JD4B-E	Concrete shear key + 4 bolts (@600)	1	Eccentric

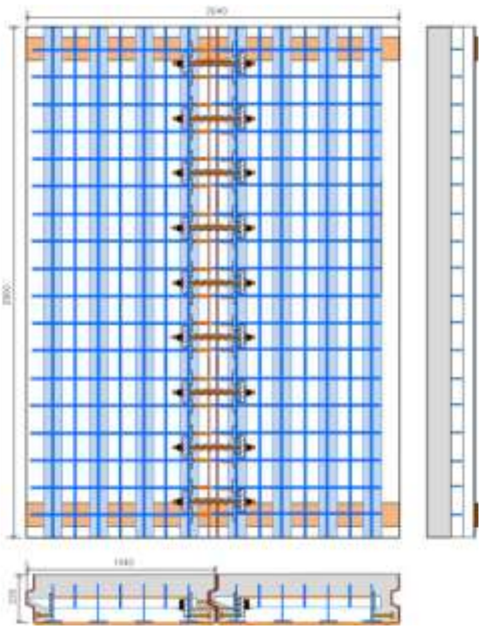


Fig. 7: Structure of the deck (JD9B-E) [6]

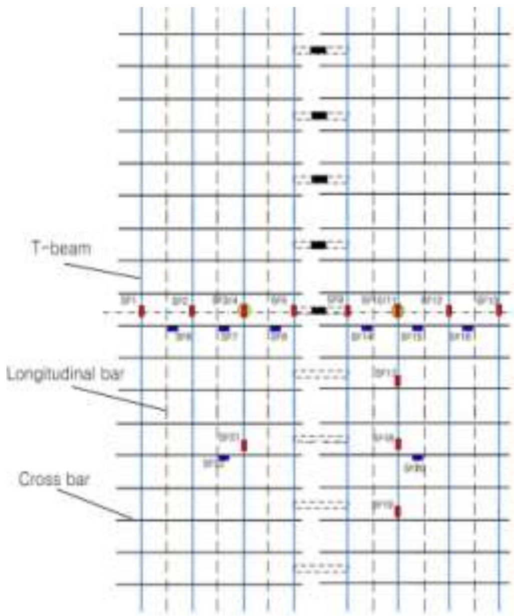


Fig. 8: Layout of strain gages [6]

(2) Test results

Table 4 summarizes the major results of the bending test by eccentric loading. In Table 4, P_{pc} is the load measured when punching crack could be clearly observed at the loaded location, P_m is the maximum load, and $P_{m,UMD}$ is the maximum load measured in specimen UMD. $\varepsilon_{b,pc}$ and $\varepsilon_{b,m}$ are respectively the strains measured in the joint bolt at the punching failure load and at the maximum load. The strains indicated in Table 4 correspond to the values measured at the bolt located at mid-span as the largest strains measured in the deck. Fig. 9 plots the load-deflection curves measured during the bending test and correspond to the values measured at the location of maximum displacement.

Table 4: esults of bending test by eccentric loading [6,8]

Specimen	Load (kN)		P_{pc} / $P_{m,UMD}$	P_m / $P_{m,UMD}$	Strain in joint bolt ($\times 10^{-6}$)	
	P_{pc}	P_m			$\varepsilon_{b,pc}$	$\varepsilon_{b,m}$
UMD	–	551.3	–	1.00	–	–
JD9B-E	673.4	762.2	1.22	1.38	227	764
JD4B-E	678.3	734.4	1.23	1.33	216	732

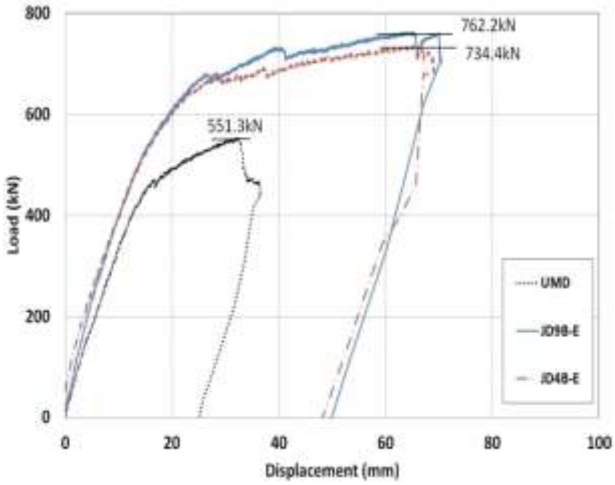


Fig. 9: Load-deflection curves [6,8]

In view of the load-deflection curves, specimen JD9B experienced rapid loss of its flexural rigidity with respect to the load of 500 kN and showed clear change in the slope of the load-deflection curve at about 600 kN corresponding to the yielding of most of the T-beams around the loaded point. The examination of the major cracks revealed the occurrence of inclined cracks on the top of the slab at load of approximately 365 kN. For load beyond 560 kN, cracks propagated from the lateral faces of the slab to meet those at the top of the slab and numerous crack developed around the loaded point (Figs. 10 and 11). The maximum load was measured at 762.2 kN [6].

For specimen JD4B-E with 4 bolts installed in the joint, the maximum load was measured at 734.4 kN, which is comparable to that of specimen JD9B-E with a difference within approximately 6%. The cracking and failure behaviors were seen to be similar to those of specimen JD9B-E.

The comparison of the maximum load of specimens JD9B-E and JD4B-E did not show noticeable difference. Compared to the reference specimen UMD, the specimens appeared to sustain loading larger by about 36% on the average.

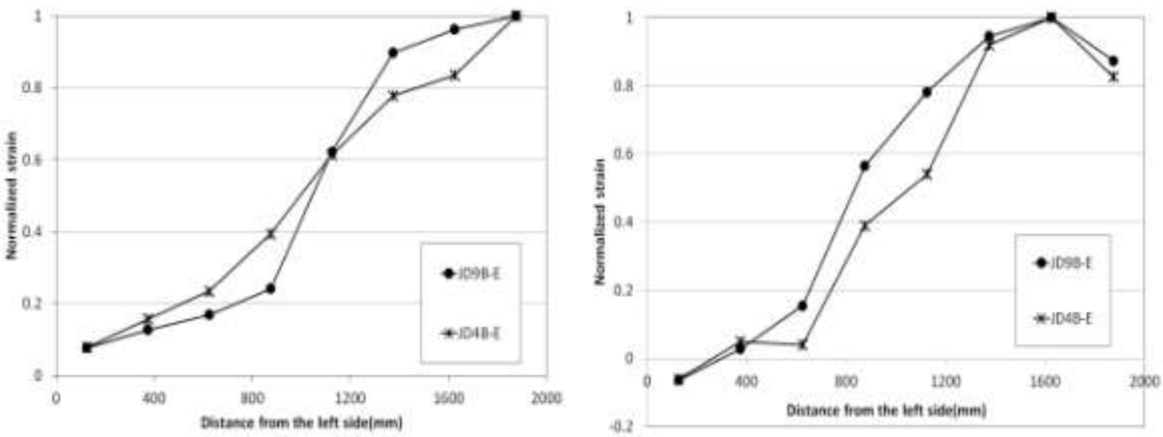
Fig. 10: Bending test [6]

Fig. 11: Punching failure [6]

Larger transverse stiffness of the deck enables the deck to transfer the load more effectively in the transverse direction. Accordingly, the parabolic distribution of the stress developed in the section of the deck happens to exhibit smoother slope [3,17]. In order to examine the load transfer behavior in the transverse direction according to the number of bolts installed in the joint, the transverse distributions of the strain measured by the strain gages disposed at the bottom of the T-beam flanges of the deck were compared.

Fig. 12 compares the transverse distribution of the strain measured in specimens JD9B-E and JD4B-E at the initial elastic state and at the maximum loading state [6,8]. There is no clue about which of the joints is more favorable to the transverse load transfer in the elastic state since the distributions are very similar to each other. Besides, the comparison of the graphs in the maximum loading state shows that specimen JD9B-E exhibits larger and smoother distribution but there is no noticeable difference.

Accordingly, these results indicate that no particular difference could be observed in the load transfer performance in the transverse direction with respect to the number of bolts installed in the bolted shear key joint. As shown in Table 4, the fact that the strain values of the joint bolts measured at maximum load did also not exhibit noticeable difference demonstrates that the number of bolts installed in the joint has not particular effect on the load transfer performance of the joint.



(a) Elastic state **(b) Maximum loading state**

Fig. 12: Normalized strain distribution for eccentric loading

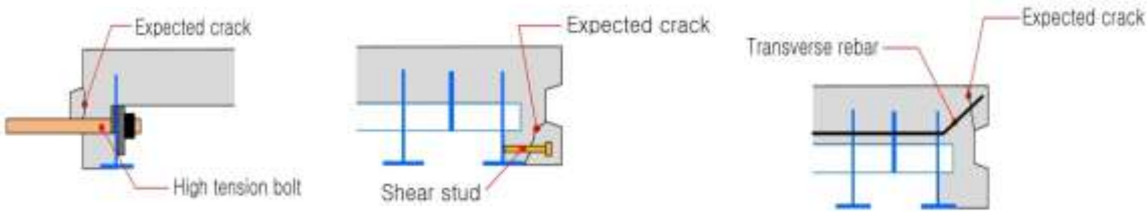
In view of the results of the discussion, the number of bolts installed in the joint has no sensitive effect on the shear performance of the joint and, in turn, appears to have no particular influence on the flexural performance of the whole deck structure. It seems that the flexural performance of the deck system is subordinated to the punching failure of the slab around the loaded point.

Here, the point to be considered is that the results are restricted to the eccentric concentrated loading. This means that other results are likely to happen in occurrence of bending moment in the joint under loading applied at the center of the deck or in absence of punching failure like under distributed loading. Additional tests are thus necessary to further the analysis.

IV. INVESTIGATION OF DESIGN SAFETY OF JOINT STRUCTURE TO SHEAR

In view of the analysis of the push-out test results, the design formula proposed in LRFD appears to be the most appropriate for the shear design of the joint considered in this study. Safety check to shear was conducted using the design formula of LRFD for each of the detailed structures shown in Fig. 13.

Table 5 presents the cross sectional areas of steel and concrete used for the calculation of the shear resistance and Table 6 lists the calculated shear resistances. Here, the external load applied on one side of the deck is assumed to be transferred to the other side of the deck by means of the joint located within about 1.0 m from the center of the loaded point.



(a) Male shear key + bolt (St1) **(b) Female shear key at bottom + stud (St2)** **(b) Female shear key at top + transverse rebar (St3)**

Fig. 13: Checked part of joint for shear resistance

Table 5: Cross section for the estimation of shear resistance

Joint type	Area of shear-resistant cross section	
	Steel (A_{vf})	Concrete (A_c)
St1	M30 (561 mm^2) × 3	80 mm × 1000 mm
St2	Φ16 (201 mm^2) × 6	
St3	Φ16 (198.6 mm^2) × 7	

Table 6: Comparison of calculated shear resistance and experimental values

Joint type	V_n (kN)			$0.80V_n$ (kN)
	$cA_c + \mu(A_{vf}f_y + P_c)$	$0.25f_cA_c$	$10.3A_c$	
St1	1732.0	700.0	824.0	560.0
St2	899.4			
St3	1002.5			

Since the maximum load that could be sustained by one 1.0-m wide side of the deck was 551.3 kN (Fig. 9), it was assumed that the whole structural system of the deck could sustain the doubled load of 1,102.6 kN. Under this assumption, the joint should be able to transfer a load of 551.3 kN. Accordingly, the shear resistance of about 560.0 kN provided for the joint by the design formula of LRFD indicates that the joint could transfer the load of 551.3 kN. However, this simplistic analysis neglects the fact that the maximum load is determined by the punching failure of the deck slab. In reality, since the maximum load is mostly determined by the punching failure in single-span decks subjected to concentrated loading, the actual maximum load is significantly smaller than 1,102.6 kN and the load to be sustained by the joint is smaller than 551.3 kN. This means that the safety factor of the joint to shear would be noticeably higher if the shear force to be actually sustained is considered.

In view of the bending test results of real decks, the maximum load relevant to the punching failure of the deck concrete at the loaded location reached 762.2 kN for JD9B-E and 734.4 kN for JD4B-E. Since the maximum load that can be sustained by one side of the deck is 551.3 kN, it can be assumed through simple arithmetic that a force of only 200 kN was transferred to the other side of the deck through the joint. This implies that the shear resistance of about 560.0 kN of the joint is 2.8 times larger than the actually working shear force. Moreover, considering that the strain of 764×10^{-6} developed in the joint bolt at the maximum load of the deck remains below the yield strain of the bolt (Table 4), this indicates that the joint composed of the concrete shear key and bolts secures sufficient safety with regard to the shear performance.

V. CONCLUSIONS

The joints of the partially filled steel grid composite deck, which can be precast, may be designed as mechanical connections formed by concrete shear keys and bolts. The analysis of the results of push-out test and bending test of the deck by eccentric loading revealed that the bolted shear key joint could be designed using the formula of LRFD and that the shear resistance of the joint was more closely related to the resistance of the concrete shear key rather than to the resistance of the bolts. The analysis of the static safety of the joint structure showed that the joint formed by the concrete shear key and the bolts secured sufficient shear resistance.

The test of the deck showed that there was practically no change in the overall behavior of the deck and load transfer performance of the joint even when the number of bolts installed in the joint was reduced from 9 to 4. This indicated that the shear behavior and resistance of the joint were more influenced by the concrete shear key than the number of bolts installed in the joint. Accordingly, the number of bolts to be installed in the joint shall be reduced to achieve optimal design of the joint structure. However, one should recall that the test of the deck was limited to the application of an eccentric loading and was purposed to the evaluation of the static shear performance of the joint. This means that the joint would exhibit different behavioral characteristics in case of fatigue loading or in case where bending moment becomes larger than shear under the direct application of external load on the top of the joint. Considering this fact, additional studies are necessary to evaluate the behavior of the joint under other loading conditions. The optimal design of the joint would be achieved by reflecting synthetically its behavioral characteristics under various conditions.

ACKNOWLEDGMENT

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