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Experimental study on half-pipe shear connectors under direct shear

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ABSTRACT

Various types of steel-concrete composite decks for highway bridges have been developed in Japan. The steel-concrete composite deck holds high fatigue durability and large toughness. However, the steel-concrete composite deck is generally heavier than RC slabs or steel plate decks. The purpose of this study is to investigate mechanical characteristics of half-pipe shear connectors under direct shear force experimentally. The shear bond capacity and deformability of the half pipe shear connector are meaningfully affected by the thickness-to-diameter ratio.

1. INTRODUCTION

Recently in Japan, various types of steel-concrete composite decks (SCCDs in abbreviation) have been investigated to be applied to an abundance of super structures(JBA 2006). The steel deck plate and RC slab work together and also supplement their weakness each other. It significantly depends on the arrangement of mechanical shear connectors, for example, headed stud connectors(Ollgaard 1971) or channel's flange welded shear connectors(Slutter 1965). Perfobond strips have been also used as one of the shear connectors and their mechanical performance was investigated (Leonhardt 1987, Oguejiofor 1992). Holding high deformability and large toughness, SCCDs have been gradually constructed for recent years in Japan. However, SCCDs are generally heavier than ordinary RC slabs or steel decks for superstructures. It is necessity to reduce own weight of SCCDs because the lighter SCCD will reduce dead load and seismic action for substructures such as a pier.

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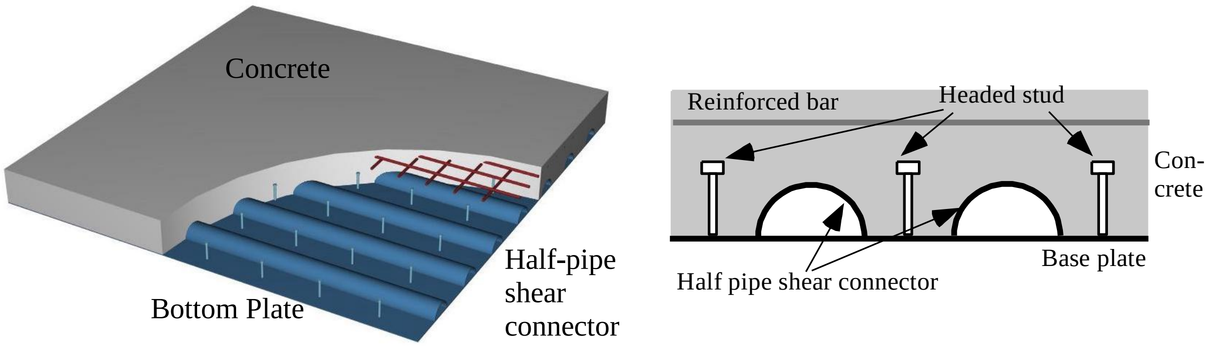


(a) Overview (b)Cross-sectional view

Fig. 1 Proposed steel-concrete composite deck

Under the above-described background, the authors have proposed the lightweight steel-concrete composite deck consisting of channel flange welded on a bottom plate with headed studs and RC slab. Furthermore, we conducted its mechanical behavior subjected to direct shear(Uenaka 2010). It is also clear that the channel holds a good deformability and shear bond capacity are meaningfully affected by the channel web's height-to-thickness ratio.

In the following, we also propose the new type of SCCD consisting of RC slab and the half-pipe shear connectors(HPSCs in abbreviation) welded on a bottom plate with headed stud connectors. Reduction of own weight of the deck due to hollow section inside HPSCs can be anticipated. For the practical application of the HPSCs as the shear connectors in a SCCD (H-SCCD in abbreviation), it is very significant that direct shear characteristics of HPSC must be clear.

This study aims to investigate direct shear capacities of eight HPSC specimens through the push-out testing method. Main testing parameters selected were thickness(t) and diameter(D) of the HPSC. Furthermore, a simple method to predict shear capacities of the HPSCs was proposed. The part of this study was reported previously in Japan(Uenaka 2009).

2. LIGHTER STEEL-CONCRETE COMPOSITE DECK

The proposed H-SCCD is shown in Fig. 1. Figures 1(a) and (b) are overview of the steel-concrete composite deck and the HPSCs arrangement, respectively. The headed studs are arranged accordingly to withstand the shear force perpendicular to the longitudinal direction.

The proposed deck is composed of RC slab and half pipes welded on a bottom steel plate with headed stud connectors. This steel-concrete composite deck is consequently lighter than ordinary SCCDs due to the hollow section inside the HPSCs. Additionally, when hot or cool water runs through the hollow section inside the HPSC, the own inside temperature of H-SCCD may be warmed up or cooled down.

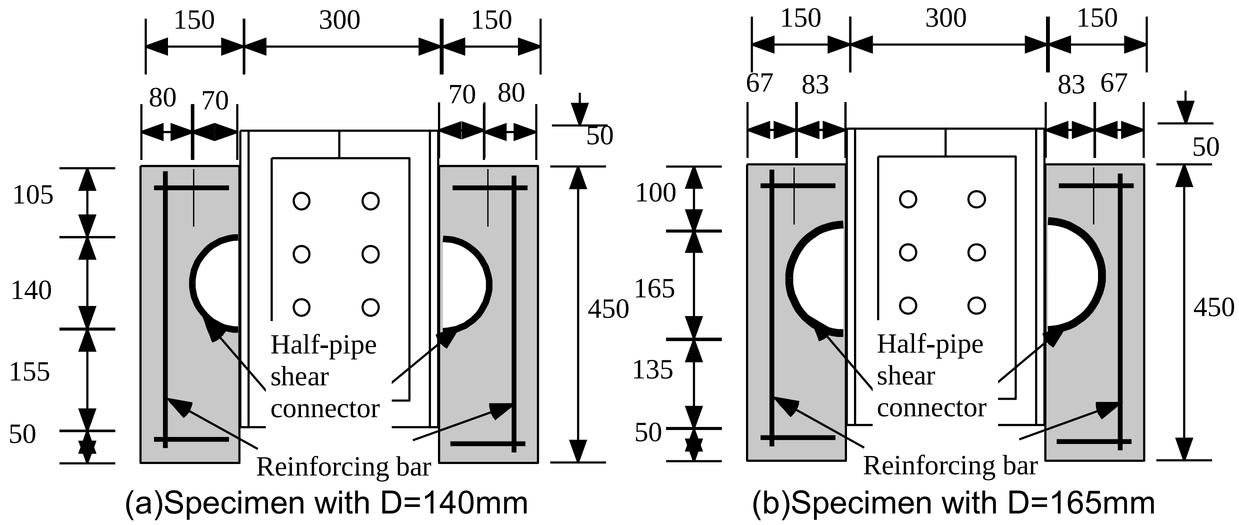


Fig. 2 Test specimens

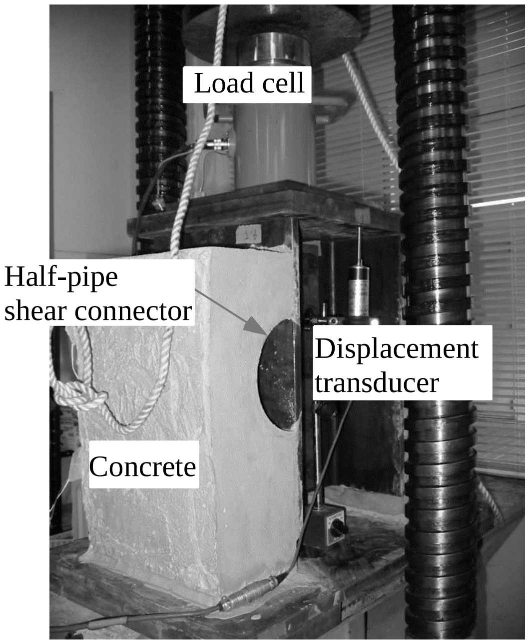


Fig. 3 Test apparatus

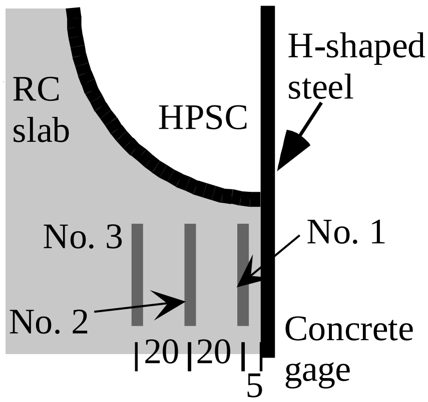


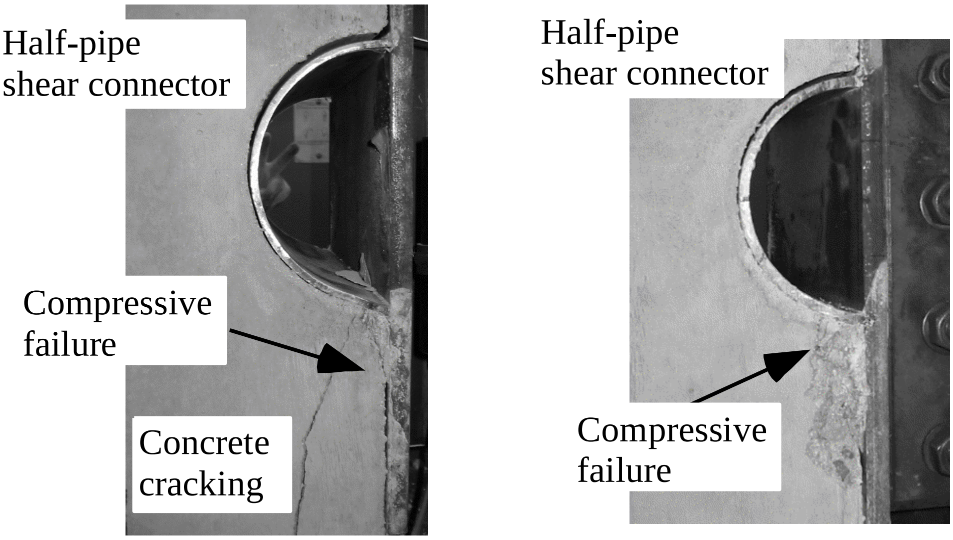
Fig. 4 Position of concrete gages

3. PUSH-OUT TEST

3.1 Test specimens

Table 1 List of the specimens

No.	Tag.	Half Pipe				Concrete
		Diameter	thickness	Ratio		
		D (mm)	t (mm)	D/t	t/D	f_c' (MPa)
1	P165-45	165	4.5	36.7	0.027	30.9
2	P165-50	165	5.0	33.0	0.030	30.9
3	P165-60	165	6.0	27.5	0.036	32.4
4	P140-23	140	2.3	60.9	0.016	30.2
5	P140-32	140	3.2	43.8	0.023	30.2
6	P140-40	140	4.0	35.0	0.029	28.4
7	P140-45	140	4.5	31.1	0.032	28.4
8	P140-60	140	6.0	23.3	0.043	28.4



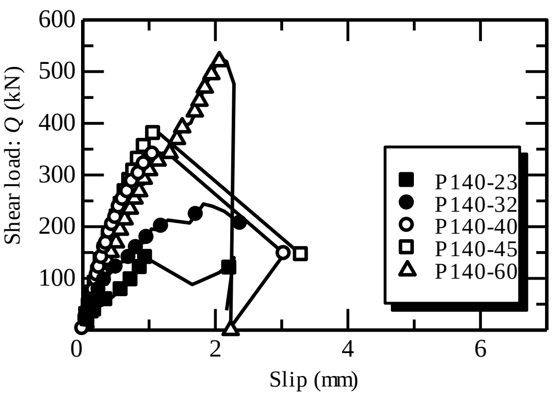
(a)165-50 (b)140-60

Fig. 5 Failure modes

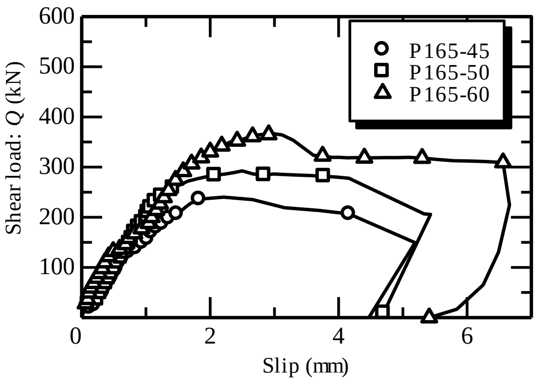
Figure 2 and Table 1 show the details of test specimens. The specimens are combinations of diameter and thickness of the HPSC. For all specimens, the RC slab commonly has 150mm in thick and 450mm in height, respectively. Four reinforcing bars were arranged to prevent unnecessary cracking of the concrete before reaching the ultimate load. Whereas, the diameter of the HPSC was varied as 165 or 140mm and thickness of that was ranging from 2.3 to 6.0mm. Therefore, the thickness-to-diameter ratios (t / D) were ranging from 1.6 to 4.3%. The tensile strength (f_u) of the HPSC and H-shaped steel were 400MPa(JIS-SS400).

Two HPSCs were welded on flanges of T-shaped steel members. The friction between the flange and the HPSC was removed by grease. The test specimens were placed on the abutment of loading machine with the cement paste as shown in Fig. 3.

Direct shear action between the HPSC and the RC slab was applied by the universal tester as shown in Fig. 3. The direct shear load was applied by cyclic loading until the ultimate failure of the specimens.



(a)D=140mm



(b)D=165mm

Fig. 6 Load-slip curve

3.2 Measurement

Two displacement transducers were used to measure the relative slip between the HPSC and the RC slab. Whereas, in two of the specimens, namely P140-23 and P140-32, three axial strain gages were attached on the surface of the RC slab beneath the HPSC to obtain their strain distributions as shown in Fig. 4.

4. RESULTS AND DISCUSSION

4.1 Failure modes

Observed failure modes were concrete cracking and compressive failure of the RC slab under the HPSC as shown in Fig. 5. The specimen, D165-50, both compressive failure and cracking of the concrete slab under the HPSC can be observed as shown in Fig. 5(a). Furthermore, the specimen, D140-60, showed compressive failure near the welded joint of the HPSC as shown in Fig. 5(b). This is because the direct shear action concentrated into the welded joint between the HPSC and the flange of H-shaped steel. No cracking of a fillet welding of the HPSC was found.

4.2 Shear load-slip curve

Figure 6 shows the relationship between the shear load(Q = P/2, P: applied load) and the relative slip between the HPSC and the RC slab. In the specimens of D=140mm with t/D being larger than 0.029, the deformation curves increase gradually up to the peak shear load. On the other hand, the deformation curves of the other specimens draw flexibly as same as a stud shear connector as shown in Fig. 6(b). It can be also found that the deformability increases as t / D increases.

Table 2 Test results

No.	Tag.	Strength		
		P_u (kN)	Q_u (kN)	M_u (kN m)
1	P165-45	480.2	240.1	13.2
2	P165-50	585.1	292.5	16.1
3	P165-60	733.0	366.5	20.2
4	P140-23	280.3	140.1	6.5
5	P140-32	488.0	244.0	11.4
6	P140-40	688.9	344.5	16.1
7	P140-45	772.2	386.1	18.0
8	P140-60	1047.6	523.8	24.4

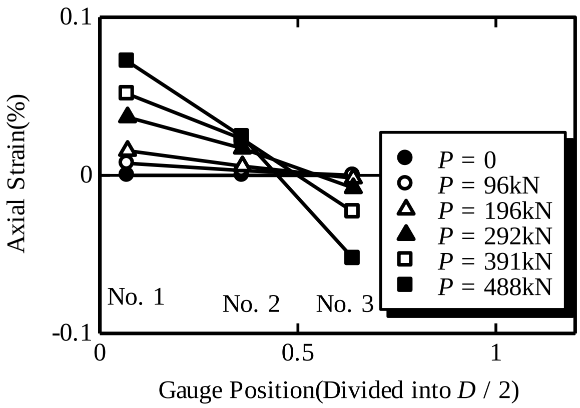


Fig. 7 Strain distribution(140-32)

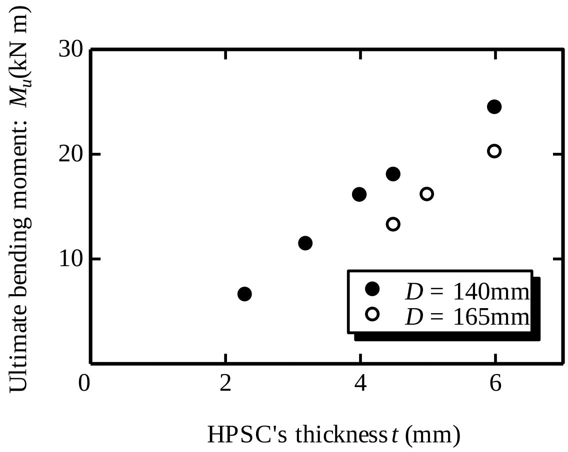


Fig. 8 Local bending moment and thickness

4.3 Strain Distribution

Figure 7 shows the relationship between the gages' position divided by the HPSC's height (r = D / 2) and the concrete strain measured on the RC slab's surface. Compressive strain is taken as positive. This fact indicates that local bending moment occurred in the HPSC. In the specimen of P165-50, both cracking and compressive failure under the HPSC was observed as described in *4.1 Failure modes*. The upper the HPSC was in tension and the lower that was in compression, respectively.

4.4 Ultimate strength

According to *4.3 Strain distributions*, the local bending moment was distributed to the HPSC. In this study, triangular stress distribution at the ultimate loading is assumed to calculate the local bending moment as below.

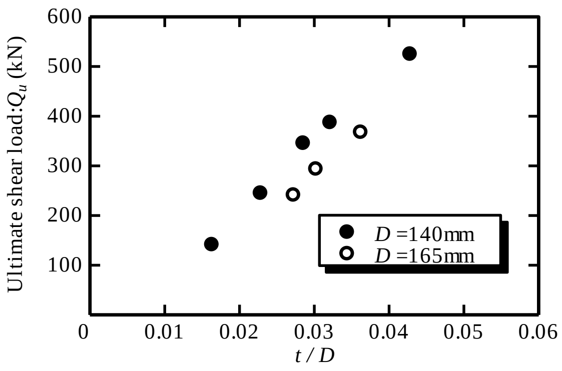


Fig. 9 Ultimate shear force and *t/D*

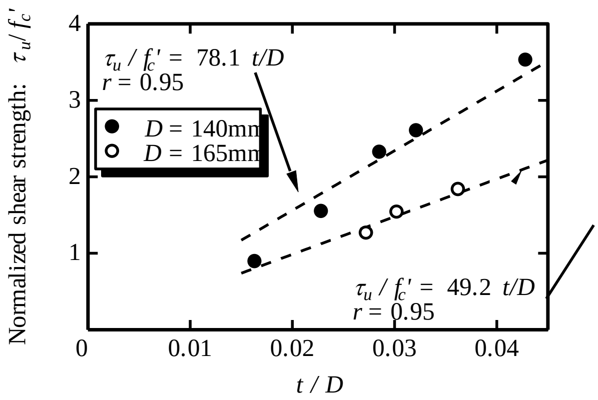


Fig. 10 Normalized shear strength and *t/D*

$$M_u = \frac{2}{3} Q_u r \tag{1}$$

where Q_u is the ultimate shear force(=P_u/2, P_u: maximum applied load) and r is the radius of the HPSC(=D/2), respectively. Obtained three testing strengths, P_u, Q_u and M_u, are summarized in Table 2.

The relationship between the ultimate bending moment(M_u) and the thickness of HPSC(t) is shown in Fig. 8. It can be found that M_u gradually increases as the thickness of the HPSC increases.

The relationship between the ultimate shear force(Q_u) and the HPSC's thickness-to-diameter ratio(t / D) is shown in Fig. 9. It can be found that the ultimate shear force is strongly affected by t / D. Furthermore, Fig. 10 shows the relationship between the shear strength(τ_u) and the thickness-to-diameter ratio(t / D). The shear strength assumed such as in Fig. 7 can be calculated as below.

$$\tau_u = \frac{4Q_u}{rB} \tag{2}$$

From Fig. 10, the linear relationship between the normalized shear strength(τ_u/ f_c') and t / D. The method of least squares being applied, the normalized shear strength of the HPSC under the parameter of t / D ranging from 0.02 to 0.04 can be expressed as below.

$$\frac{\tau_u}{f_c'} = 78.2 \frac{t}{D} \quad \text{(for } D\text{140 specimens)} \tag{3a}$$

$$\frac{\tau_u}{f_c'} = 49.2\frac{t}{D} \quad \text{(for } D\text{165 specimens)} \tag{3b}$$

Therefore, the simply estimated shear capacity $Q_{u\text{-}est}$ of the HPSC is proposed as below.

$$Q_{u-est} = \frac{f_c'}{4} rB\left(78.2\frac{t}{D}\right) \quad \text{(for D140 specimens)} \tag{4a}$$

$$Q_{u-est} = \frac{f_c'}{4} rB\left(49.2\frac{t}{D}\right) \quad \text{(for D165 specimens)} \tag{4b}$$

It can be noted that the simply estimated shear capacity obtained from Eq. (4) is applicable for t / D ranging from 0.02 to 0.04.

5. CONCLUSIONS

The proposed new type of steel-concrete composite deck consists of the RC slab and the HPSC welded on a bottom steel plate with headed studs. Under the parameters t / D, the mechanical behavior of the HPSC subjected to direct shear was investigated experimentally. From the results, the following remarks can be drawn.

1. Observed failure modes were concrete cracking and compressive failure under the HPSC. In the specimen of D165-50, the failure mode observed was both concrete cracking and compressive failure of the RC slab under the HPSC. This is because the directly shear load triangularly distributed.
2. In the D=140mm specimens with t / D being larger than 0.029, the deformation curve increases linearly up to the peak load. On the other hand, the other specimens deformed gradually.
3. Triangular strain distributions in the RC slab under the HPSC were found. This fact indicated that the local bending moment occurred in the HPSC being induced by the RC slab. The local bending moment was occurred owing to the concentrated shear force.
4. The ultimate shear force(Q_u) increased as t / D increased. This is because the ultimate local bending moment (M_u) gradually increased as the HPSC's thickness increased.
5. A simple method to predict the direct shear capacity of the HPSC subjected to the direct shear was proposed with respect to t/D.

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