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Shear Bond Characteristics of Embossed Steel Plates for an Open Sandwich Type Composite Beam

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SYNOPSIS: RILEM/CEB/FIP has proposed a standard shear bond test method of deformed steel bars with R/C beam. We developed herein a similar test of embossed steel plate for open sandwich beams. As the results of 30 specimens, the obtained failure modes and bond strengths were almost same as those from the direct pull-out test method that we previously proposed.

Keywords: Composite Construction, Embossed Steel Plate, Shear Bond Characteristics and Beam

1. INTRODUCTION

In Japan, there have been developed various embossed steel products for composite constructions, for instance, checkered or ribbed plates and tubes with inner or outer embossed surface in order to improve shear bond characteristics between steel and concrete in composite members[1]. In particular, it should be noted that these embossments are automatically made through a heat rolling process at steelworks so as to make their mass production possible. Their shear bond characteristics, however, is not clear enough to establish the practical design method for composite constructions when they are adapted. We, therefore, previously conducted a systematic study on the characteristics of embossed steel plates by using a direct shear test method, also called pull-out test method, with a controlled confinement stress and proposed the empirical equation to estimate their shear bond strengths[2]-[4]. To examine an applicability of the equation in practice, it should be required to carry out a composite beam test under shear incidental to bending moment.

The purpose of this study is, therefore, to evaluate the shear bond characteristics of the embossed steel plates 30 open sandwich beam specimens. The test method was developed here as an alternation of the bond test method for deformed steel bars in R/C beam proposed by RILEM/CEB/FIP[5]. Checkered plates and ribbed plates were used for the bottom plates of the specimens. These plates had various bond lengths and thickness, and headed stud connectors for some of them. The obtained shear bond strengths of the plates were compared with the predicted values based on the direct shear tests with a confinement stress[2]-[4]. The validity of a simple accumulative strength of the plates and the studs was also examined.

2. EXPERIMENT

2.1 Specimens

The specimens used herein was designed based on the beam test method proposed by RILEM/CEB/FIP[5](see **Fig. 1**). The specimens are shown in **Fig. 2** and **Fig. 3**. Difference between the two type of the specimen was that one of **Fig. 2** consisted of a steel plate and two concrete blocks, while the other of **Fig. 3** consisted of a steel plate and one concrete block. To easily compare the result from these tests with the previous results from the direct shear tests, we adapted a web support system with 2 round bars of 50mm in. diameter as shown in **Fig. 2** and **Fig. 3**, in which confinement stress between steel plate and concrete set free. The adaptation made the condition of bond severer than the practical beam member in which confinement stress could be expected somewhat near a support or loading portion.(see **Photo 1**). Those two types of embossments used are shown in **Fig. 4** and **Fig. 5**. Some of the specimens had a headed stud connector, which was welded at the mid point of the bonded

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Table. 1 Test Specimens

					Plate							Reinforcement ratio	
					Embossments								
No.	Tag.	Width B (mm)	Effect. height d (mm)	Type	sort	Height h_e (mm)	Number n_e	Bond Length L_b (mm)	Bear. Ratio m	Thick t (mm)	Stud	Plate p' (%)	Rebar p (%)
1	SP-6M-1	200	297.0	1	Plain	0.0	0	400	0.0000	6	$\phi16\times80$	2.02	0.00
2	SR4-6M-1	200	297.0	1	Rib	2.5	4	400	0.0250	6	$\phi16\times80$	2.02	0.00
3	DR4-6M-1	200	297.0	1	Rib	2.5	4	400	0.0250	6	Dummy	2.02	0.00
4	NR4-6M-1	200	297.0	1	Rib	2.5	4	400	0.0250	6	Non	2.02	0.00
5	SR8-6M-1	200	297.0	1	Rib	2.5	8	400	0.0500	6	$\phi16\times80$	2.02	0.00
6	DR8-6M-1	200	297.0	1	Rib	2.5	8	400	0.0500	6	Dummy	2.02	0.00
7	NR8-6M-1	200	297.0	1	Rib	2.5	8	400	0.0500	6	Non	2.02	0.00
8	SC-6M-1	200	297.0	1	Check.	2.5	-	400	0.0696	6	$\phi16\times80$	2.02	0.00
9	DC-6M-1	200	297.0	1	Check.	2.5	-	400	0.0696	6	Dummy	2.02	0.00
10	NC-6M-1	200	297.0	1	Check.	2.5	-	400	0.0696	6	Non	2.02	0.00
11	SP-6M-2	200	297.0	2	Plain	0.0	0	400	0.0000	6	$\phi16\times80$	2.02	0.95
12	SR4-6M-2	200	297.0	2	Rib	2.5	4	400	0.0250	6	$\phi16\times80$	2.02	0.95
13	DR4-6M-2	200	297.0	2	Rib	2.5	4	400	0.0250	6	Dummy	2.02	0.95
14	NR4-6M-2	200	297.0	2	Rib	2.5	4	400	0.0250	6	Non	2.02	0.95
15	SR8-6M-2	200	297.0	2	Rib	2.5	8	400	0.0500	6	$\phi16\times80$	2.02	0.95
16	DR8-6M-2	200	297.0	2	Rib	2.5	8	400	0.0500	6	Dummy	2.02	0.95
17	NR8-6M-2	200	297.0	2	Rib	2.5	8	400	0.0500	6	Non	2.02	0.95
18	SC-6M-2	200	297.0	2	Check.	2.5	-	400	0.0696	6	$\phi16\times80$	2.02	0.95
19	DC-6M-2	200	297.0	2	Check.	2.5	-	400	0.0696	6	Dummy	2.02	0.95
20	NC-6M-2	200	297.0	2	Check.	2.5	-	400	0.0696	6	Non	2.02	0.95
21	NR4-9M-2	200	295.5	2	Rib	2.5	4	400	0.0250	9	Non	3.05	0.96
22	NR4-12M-2	200	294.0	2	Rib	2.5	4	400	0.0250	12	Non	4.08	0.96
23	NR8-9M-2	200	295.5	2	Rib	2.5	8	400	0.0500	9	Non	3.05	0.96
24	NR8-12M-2	200	294.0	2	Rib	2.5	8	400	0.0500	12	Non	4.08	0.96
25	NR2-6S-2	200	297.0	2	Rib	2.5	2	200	0.0250	6	Non	2.02	0.95
26	NR4-6S-2	200	297.0	2	Rib	2.5	4	200	0.0500	6	Non	2.02	0.95
27	NR6-6L-2	200	297.0	2	Rib	2.5	6	600	0.0250	6	Non	2.02	0.95
28	NR10-6L-2	200	297.0	2	Rib	2.5	10	600	0.0417	6	Non	2.02	0.95
29	NC-6S-2	200	297.0	2	Check.	2.5	-	200	0.0696	6	Non	2.02	0.95
30	NC-6L-2	200	297.0	2	Check.	2.5	-	600	0.0696	6	Non	2.02	0.95

Note: $m=n_e h_e/L_b$ for ribbed plate(see Eq(3) for checkered plate)

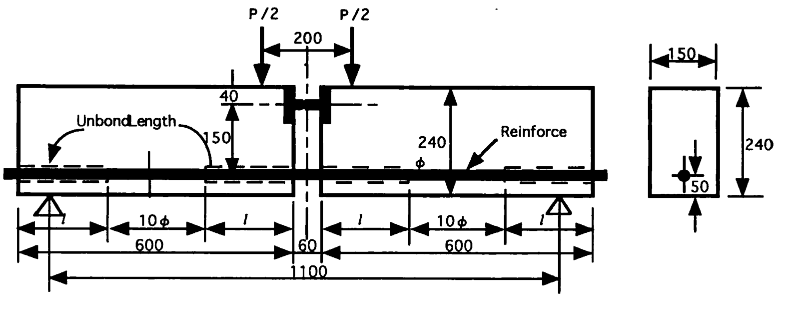

Fig. 1 Beam Test(RILEM/CEB/FIP)

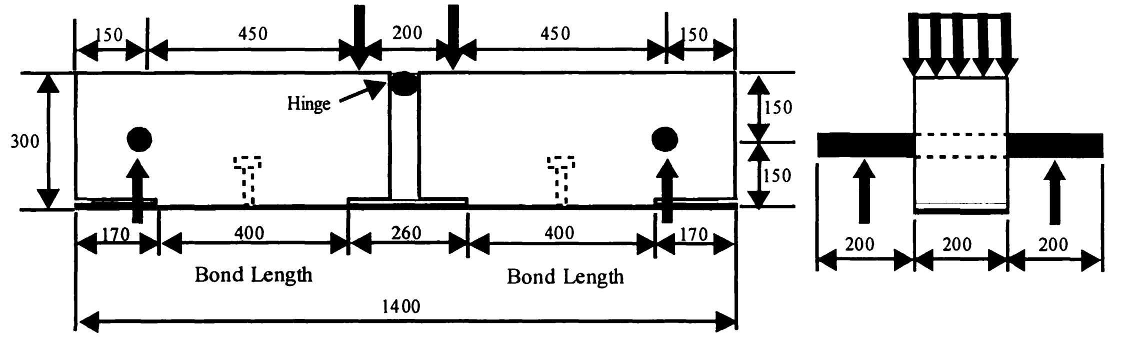

Fig. 2 Test Specimen(Type 1)

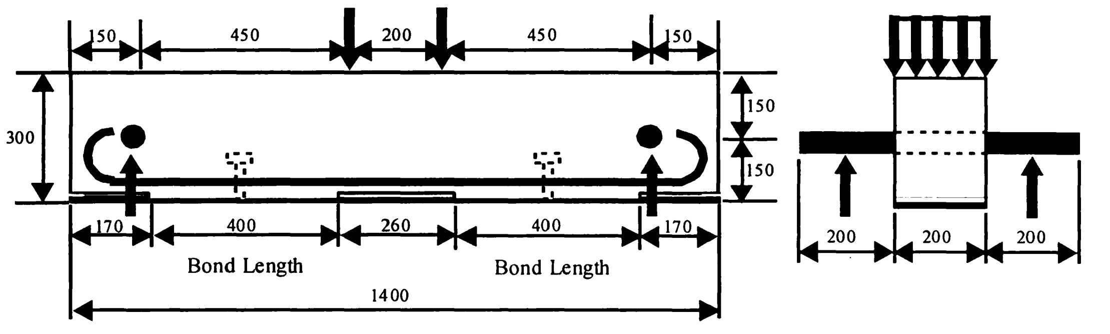

Fig. 3 Test Specimen(Type 2)

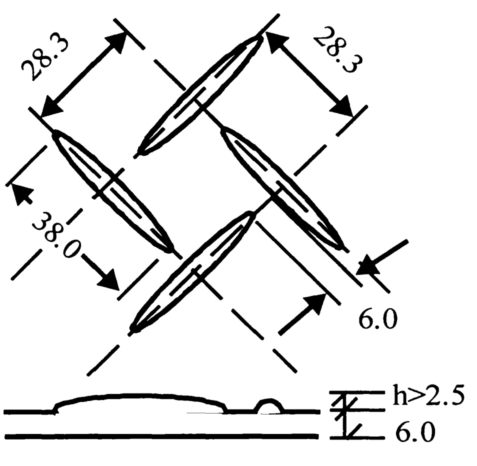

Fig. 4 Checkered Plate

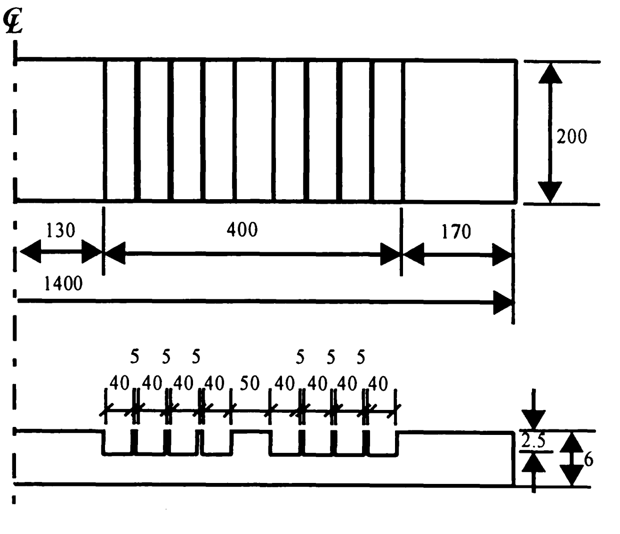

Fig. 5 Ribbed Plate

area of each plate. Some of the specimens also had a dummy stud, which was just used to prevent only the separation between the steel plates and the concrete block without shear interaction. The concrete block in the specimens had enough web reinforcements to prevent shear failure of concrete.

2.2 Experiment Parameter

Experiment parameters were 4 as followed: (1) with or without stud: S; with stud, D; with dummy stud and N; without stud, (2)embossments: P; Plain, R; ribbed plate and C; checkered plate, (3)thickness of steel plates: 6mm, 9mm and 12mm,(4)bond length; 200mm, 400mm and 600mm. Details of all specimens are listed in **Table 1**. Specimens No. 1-10 in the table arranged a hinge at top and center of them which were imitated the beam test proposed by RILEM/CEB/FIP [5]. On the contrary, they were changed to No. 11-No. 30 which had bending reinforcement and no arranged the hinge at center and top of the specimens. The reason is written in *3.2 Failure Mode* clearly.

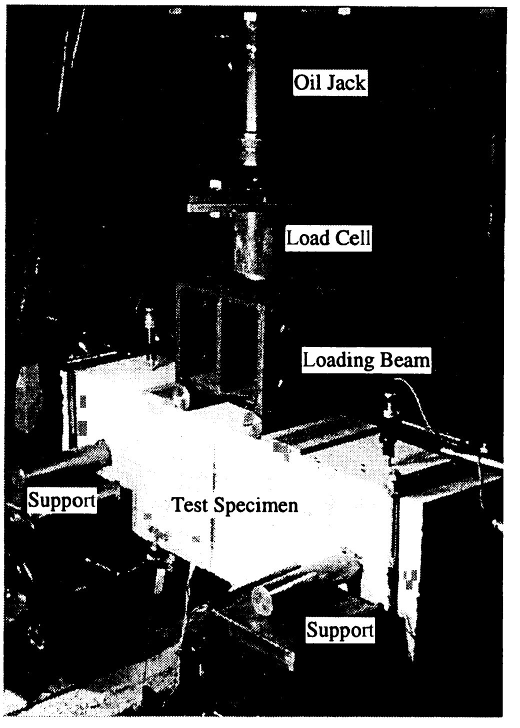

Photo 1 Test Apparatus

2.3 Proposed equations

The direct shear tests[2-4] that we already carried out have proposed the following equations. First, the shear bond strength of the checkered plate as shown in **Fig. 4** can be estimated as,

$$\frac{\tau_b}{F_c} = m\left(0.623 + 24.7\frac{\sigma_0}{F_c}\right) + 0.6\frac{\sigma_0}{F_c} \quad (1)$$

where τ_b: shear bond strength; F_c: cylinder strength of concrete; σ_0:confinement stress; m: bearing area ratio. In case where, confinement stress was free(σ_0=0), then,

$$\frac{\tau_b}{F_c} = 0.623m \quad (2)$$

The bearing area ratio of the checkered plate is expressed as follows;

$$m = \frac{A_c}{\sqrt{2}s_c^2} = 0.0696 \quad (3)$$

where A_c: area of embossment(78.5mm^2) ; s_c: intervals of embossment(28.3mm).

Second, the shear bond strength of the ribbed plate can be estimated as,

$$\frac{\tau_b}{F_c} = m\left(0.892 + 16.3\frac{\sigma_0}{F_c}\right) + 0.6\frac{\sigma_0}{F_c} \quad (4)$$

As same as the equation for the checkered plate, σ_0= 0, then,

$$\frac{\tau_b}{F_c} = 0.892m \quad (5)$$

where $m=n_e h_e/L_b$. n_e: number of embossments ; h_e: height of embossments ; L_b: bond length.. Last, the ultimate shear strength of a stud(Q_u) can be written as follows[6],

$$Q_u = 0.5A_s\sqrt{E_c F_c} \quad (6)$$

where A_s: cross section area of stud; E_c: Young's modulus of concrete; F_c: concrete strength.

3. RESULT

3.1 Deflection

Relations between applied load and central deflection of Type 1 and Type 2 are shown in **Fig. 6** and **Fig. 7** respectively, which have almost the same experiment parameters. In these figures, we plotted the analytical results based on elementary beam theory and FEM[7] in which we supposed two conditions of the interface between steel plate and concrete; in FEM1, perfect bond without relative slip was assumed, while in FEM2, contact condition with some slip and separation was considered.

In type 1 which was more similar to that by RILEM/CEB/FIP, large deflection due to local

Table.2 Test Results

No.	Tag.	Parameters m	L_b (mm)	t (mm)	Material Properties F_c (MPa)	E_c (GPa)	f_{sy} (MPa)	Shear bond strength (a) Exp. τ_b (MPa)	(b) Emboss. τ_{bcal}(e) (MPa)	(c) Stud τ_{bcal}(s) (MPa)	(d) (b)+(c) E+S τ_{bcal} (MPa)	(e) (a)/(d) τ_b/τ_{bcal} (%)
1	SP-6M-1	0.0000	400	6	24.7	23.1	316	0.85	0.00	0.94	0.94	90
2	SR4-6M-1	0.0250	400	6	24.7	23.1	316	0.99	0.55	0.94	1.49	66
3	DR4-6M-1	0.0250	400	6	24.7	23.1	316	0.47	0.55	0.00	0.55	84
4	NR4-6M-1	0.0250	400	6	24.7	23.1	316	0.21	0.55	0.00	0.55	38
5	SR8-6M-1	0.0500	400	6	24.7	23.1	316	0.98	1.10	0.94	2.05	48
6	DR8-6M-1	0.0500	400	6	24.7	23.1	316	0.72	1.10	0.00	1.10	66
7	NR8-6M-1	0.0500	400	6	24.7	23.1	316	0.52	1.10	0.00	1.10	47
8	SC-6M-1	0.0696	400	6	24.7	23.1	316	0.93	1.07	0.94	2.01	46
9	DC-6M-1	0.0696	400	6	24.7	23.1	316	0.93	1.07	0.00	1.07	87
10	NC-6M-1	0.0696	400	6	24.7	23.1	316	1.08	1.07	0.00	1.07	101
11	SP-6M-2	0.0000	400	6	31.2	25.8	296	0.74	0.00	1.13	1.13	65
12	SR4-6M-2	0.0250	400	6	31.2	25.8	296	1.03	0.70	1.13	1.83	56
13	DR4-6M-2	0.0250	400	6	31.2	25.8	296	0.77	0.70	0.00	0.70	111
14	NR4-6M-2	0.0250	400	6	31.2	25.8	296	0.80	0.70	0.00	0.70	114
15	SR8-6M-2	0.0500	400	6	31.2	25.8	296	1.24	1.38	1.13	2.51	49
16	DR8-6M-2	0.0500	400	6	31.2	25.8	296	1.30	1.38	0.00	1.38	94
17	NR8-6M-2	0.0500	400	6	20.3	22.1	304	1.04	0.91	0.00	0.91	115
18	SC-6M-2	0.0696	400	6	31.2	25.8	296	1.49	1.35	1.13	2.47	60
19	DC-6M-2	0.0696	400	6	31.2	25.8	296	1.34	1.35	0.00	1.35	99
20	NC-6M-2	0.0696	400	6	31.2	25.8	296	1.40	1.35	0.00	1.35	104
21	NR4-9M-2	0.0250	400	9	20.3	22.1	274	0.70	0.45	0.00	0.45	154
22	NR4-12M-2	0.0250	400	12	20.3	22.1	302	0.60	0.45	0.00	0.45	132
23	NR8-9M-2	0.0500	400	9	20.3	22.1	274	0.97	0.91	0.00	0.91	107
24	NR8-12M-2	0.0500	400	12	20.3	22.1	302	1.27	0.91	0.00	0.91	141
25	NR2-6S-2	0.0250	200	6	20.3	22.1	304	0.69	0.45	0.00	0.45	151
26	NR4-6S-2	0.0500	200	6	20.3	22.1	304	1.32	0.91	0.00	0.91	146
27	NR6-6L-2	0.0250	600	6	20.3	22.1	304	0.69	0.45	0.00	0.45	151
28	NR10-6L-2	0.0417	600	6	20.3	22.1	304	0.90	0.75	0.00	0.75	120
29	NC-6S-2	0.0696	200	6	20.3	22.1	312	1.05	0.88	0.00	0.88	119
30	NC-6L-2	0.0696	600	6	20.3	22.1	312	0.69	0.88	0.00	0.88	78

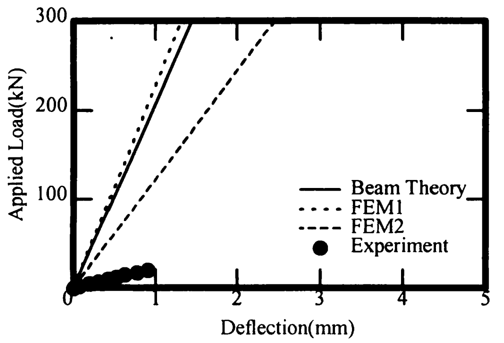

Fig. 6 Deflection(NR4-6M-1)

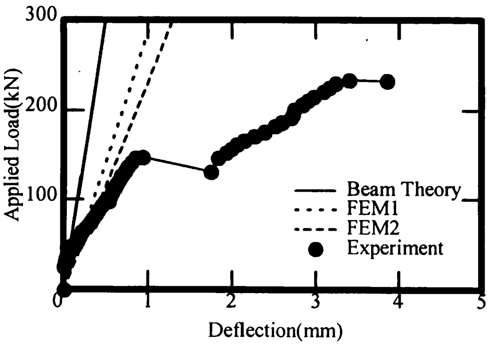

Fig. 7 Deflection(NR4-6M-2)

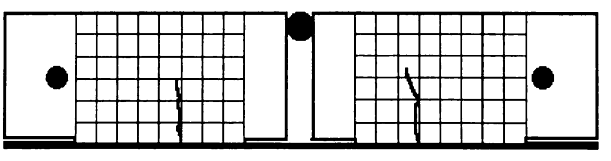

Fig. 8 Failure Mode(Type 1)

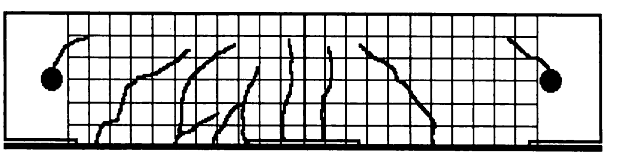

Fig. 9 Failure Mode(Type 2)

bending of steel plate at the central portion without concrete was observed.

On the contrary, in Type 1 an initial bending stiffness coincides with analytical values, and afterward, the stiffness was gradually decreased as cracks were developed.

3.2 Failure Mode

Most specimens of Type 1 were failed with separation of steel plate from concrete without cracking of concrete. In a few of them with stud connector, single crack was observed as shown in **Fig. 8**. The interface between steel plate and concrete, consequently, could not damage in all of Type 1. These failures could suggest that the most of their bending deformation concentrated on the steel plate around central portion with a lack of bending stiffness as described in previous section *3.1*.

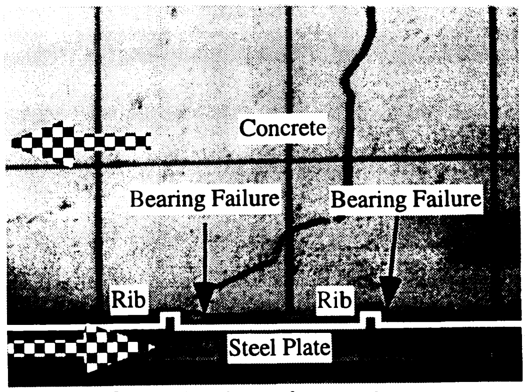

Photo 2 Bearing Failure

On the other hand, in Type 2 where the lack of the stiffness was improved, A good flexural cracking pattern was observed as shown in **Fig. 9**. Furthermore, as shown in **Photo 2**, a bearing failure mode on the interface between steel and concrete also occurred which was the same failure mode appeared in the previous direct shear tests[2]-[4].

3.3 Applied Load and Pull-Out Force

Relations between applied load and pull-out force in steel plate of Type 1 and Type 2 are shown in **Fig. 10** and **Fig. 11**, respectively. The pull-out force was the membrane force in steel plate measured by the several strain gauges at the center of the steel plates. In the specimen of Type 1, the neutral axis did not change during loading, thus, the linear relation was obtained.

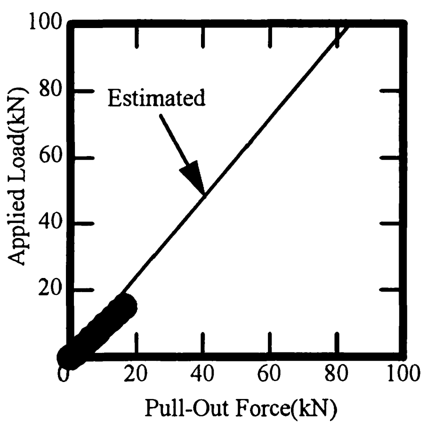

Fig. 10 Applied Load-Pull-Out Force (NR4-6M-1)

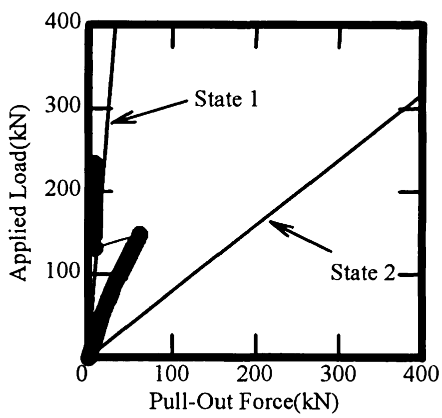

Fig. 11 Applied Load-Pull-Out Force (NR4-6M-2)

On the other hand, in Type 2, where the neutral axis could rise as bending cracks developed, the section stiffness shifted from State1 of elastic state to State 2 of ultimate limit state.

From the difference of deformation and failure mode between Type 1 and 2 described above, Type 2 is more feasible to the bond test specimen for practical composite beam member than Type 1.

3.4 Bond Strength

The obtained bond strength herein indicates the maximum average shear stress which was derived from dividing the maximum pull-out force of steel plate by a bonded area. In Table 2, the strength as

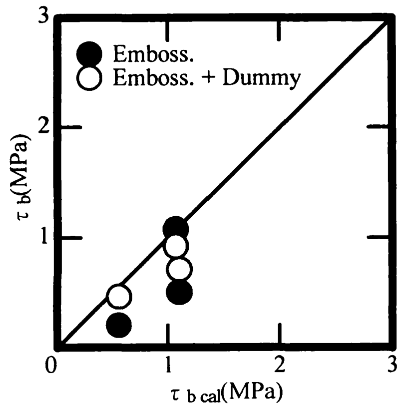

Fig. 12 Shear Bond Strength (without stud, Type 1)

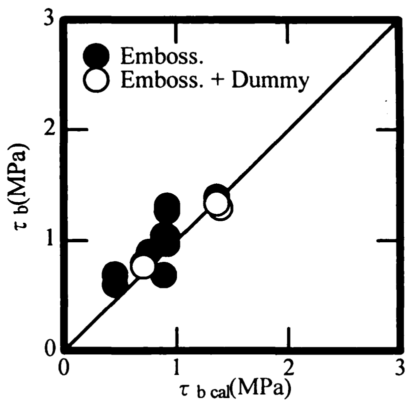

Fig. 13 Shear Bond Strength (without stud Type 2)

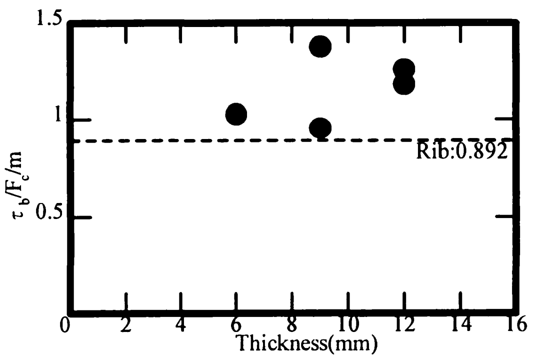

Fig. 14 Shear Bond Strength (various thickness)

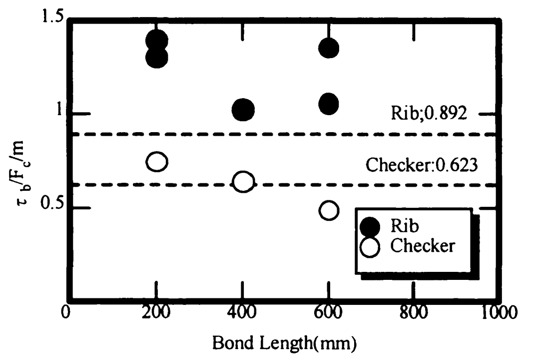

Fig. 15 Shear Bond Strength (various bond length)

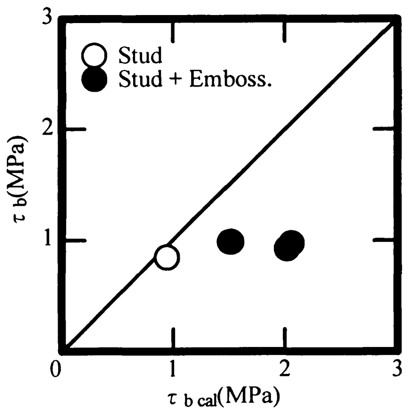

Fig. 16 Shear Bond Strength (with stud, Type 1)

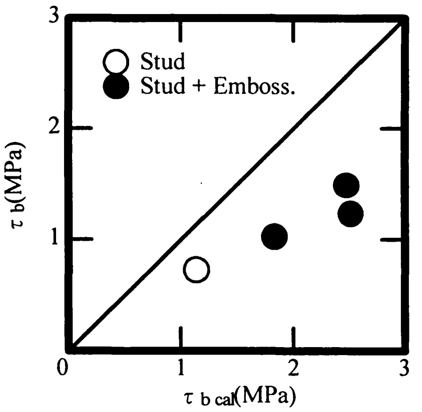

Fig. 17 Shear Bond Strength (with stud, Type 2)

(a) and its reference values as (b)-(d) are listed; (b) is estimated bond strength of embossments by Eq. (2) or Eq. (5), (c) is shear strength of stud divided Q_u of Eq. (6) by the bonded area, and (d) is the sum of (b) and (c) to compare with (a), which is also called accumulative strength.

(1) Specimens without stud

The obtained strengths of Type 1 were below the estimated values as shown in **Fig. 12** owing to the unfeasible failure mode, while the strengths of Type 2 were almost beyond the estimate value as shown in **Fig. 13**. Furthermore, the tendency of Type 2 was independent on the parameters such as the thickness of plate and the bond length as shown in **Fig. 14** and **Fig. 15**. It could be found, consequently, that the proposed equation to estimate the bond strength of embossed steel plate based on the direct shear tests had a validity to apply to the composite beam member associated with the bearing failure on the interface between steel plate and concrete.

(2) Specimens with stud

As shown in **Fig. 16** and **Fig. 17**, the obtained strengths of Type 1 and Type 2 were below the accumulative strength. The reason why the strength could not attain the estimated value will be described the next section *3.5*

3.5 Bond Stress and Slip

Figure 18 shows an example of the relation bond stress and slip in Type 2 without stud, in which bond stress arose up to the bond strength with no slip and afterward drastically decreased as slip occurred. This brittle tendency was also observed the previous direct shear test.

On the other hand, **Figure 19** shows an example of the relation bond stress and slip in Type 2 with stud, in which we plotted the reference values as (a)-(c); (a) is the relation of Type 2 without stud as same as **Fig. 18**, (b) is the relation of a stud connector by the following equation:

$$\frac{Q}{Q_u} = \frac{3.15\delta}{1+3.15\delta} \tag{7}$$

when Q: Shear force; Q_u: Ultimate shear strength of studs by Eq.(6); δ: Slip between steel and concrete(mm)

and (c) is the sum of (a) and (b). The experimental curve had a similar tendency of the curve (c), that is, bond stress rose up to the bond strength of (a), dropped once, and rose again along the curve (b). This tendency, which was the failure of the embossed plate was so brittle that the bond stress could not hold until slip was considerably large enough for the stud to give full play to its strength, enabled the bond strength of the embossed steel plate with the stud not to attain the accumulative strength.

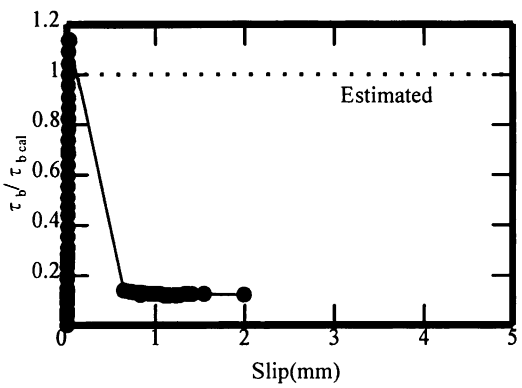

Fig. 18 Slip Behavior(NR4-6M-2)

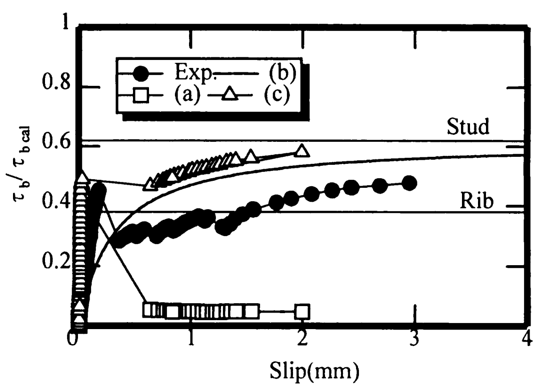

Fig. 19 Slip Behavior(SR4-6M-2)

4. CONCLUSION

As the results of shear bond tests of embossed steel plate with 30 open sandwich beam type specimens, the following conclusions are obtained.

(a) Type 1 specimens, those were more similar to the existing bond test for deformed steel bars in R/C beam, failed with separation of steel plate from concrete and without the bearing failure on the interface between the steel plate and concrete owing to a lack of the bending stiffness. Thus their bond strengths were below the estimated values from the direct shear tests.

(b) Type 2 specimens, where the bending stiffness was improved, fairly failed with flexural crack and the bearing failure mode.

(c) The bond strengths of Type 2 without stud could be evaluated by the proposed equation from the direct shear test method of full sandwich composite elements that we previously carried out.
(d) The bond strengths of Type 2 with stud could not attain the estimated accumulative strengths, because the brittle behavior of the embossment could not accord with the ductile one of the stud connector.

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NOTATIONS

A_c: area of embossments(mm^2)
B: width of beam(mm)
d: effective depth of beam(mm)
E_c: Young's modulus of concrete(GPa)
F_c: concrete strength(MPa)
f_{sy}: steel yield point(MPa)
h_e: height of embossments(mm)
L: length of bonded area(mm)
m: bearing area ratio
n_e: number of embossments
P: applied load intensity(kN)
p: reinforcement ratio of rebars(%)
p': reinforcement ratio of steel plate(%)
s_c: intervals of embossments(mm)
δ: slip(mm)
σ_0: confinement stress(MPa)
τ_b: bearing strength(MPa)
$\tau_{b\ cal}$: estimated bond strength by the equation(MPa)

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