コンクリート工学年次論文報告集，Vol.14，No.2，1992

論 文

[2177] Mechanical Properties of Steel-Concrete Composite Member

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1. INTRODUCTION

Steel-concrete composite structures have been used in buildings, bridges and, recently, in port and harbor facilities. The steel-concrete-steel sandwich element is one of various types of the composite structures. It is expected to have a potential in the submerged tube tunnel constructions since it seems to have the following advantages: "excellent" mechanical behavior, watertightness, light weight, rapid construction and reasonable cost [1]. The structural mechanism and characteristics of sandwich structures need to be more investigated and a suitable design method to be established. Several researches have shown that the shear resisting mechanism after diagonal cracking is a tied-arch mechanism [2,3]. In this study, eight steel-concrete sandwich beams were tested to investigate the flexure and shear resisting mechanism.

2. OUTLINE OF EXPERIMENTS

Eight test specimens, as shown in *Fig.*1 and *Table* 1, are used. They are 300 mm in height and 250 mm in width. Among them, six are Sandwich Beams without steel web ($SB-1 \rightarrow SB-6$) and the others are with fullweb ($SBw-1$ and $SBw-2$). Parameters of the experimental programme are (1) reinforcing method against shear, (2) shear span-depth ratio and (3) thickness of steel flanges. Steel plates of 4.4 mm in thickness and 50mm in width are used as shear reinforcement in specimens $SB-1$ and $SB-2$. Specimens $SB-3$ through $SB-6$ do not have any shear reinforcement and then, to transmit shear stresses between steel and concrete and to prevent the compression steel plate from buckling , shear connectors are used. These are angles with equal legs of $50 * 50 * 6$ mm and linked to the steel plates in the transverse direction of the beam by the fillet welding. To avoid buckling the spacing of shear connectors is 200 mm. This value is determined based on references such as Ozawa et al. [3], Yokota and Kiyomiya [2], and Wright and Oduyemi's condition; that is the ratio of the stud spacing to steel

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plate thickness should not exceed 33 [4]. From $SB-1$ through $SB-5$ a steel plate of 8.7 mm in thickness was installed as a lower and upper plate. In other specimens a 13.6 mm thick plate was used. Concrete was purchased from a local batch plant. The maximum size of aggregate was 20 mm. In the specimens $SB-1$ through $SB-6$, concrete was placed at one time. In $SBw-1$ and $SBw-2$, either one side was concreted, and, the following day, concrete was placed in the other side of the specimens. Slump tests for both concretes gave 16 and 20 cm, respectively. Mechanical properties of materials are shown in Table 1 and Table 2. Age of concrete during experiments was from 35 to 71 days. The specimens were simply supported and tested by a symmetric two point-load by means of a 1000 KN oil jack. The loading process contained several steps of loading, unloading and reloading which were done during "elastic behavior", after flexural cracking, after diagonal cracking or after the yielding of a steel element. Deflection of specimens and strains of steel plates, shear connectors and concrete were measured at every load stage. And crack propagation was observed in detail.

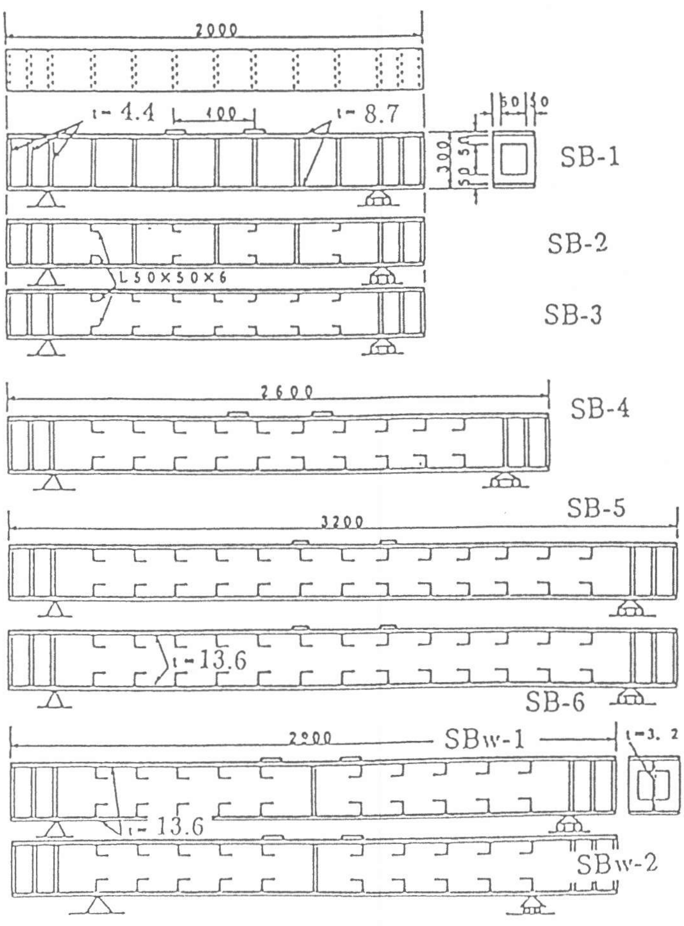

Fig.1 Test Specimens (mm)

Table 1 Specimens

Specimen	a cm	a/d	t_f mm	t_d mm	t_w mm	S_d cm	ρ_s %	f'_c MPa
SB-1	60	2	8.7	4.4	–	20	2.9	34
SB-2	60	2	8.7	4.4	–	40	2.9	34
SB-3	60	2	8.7	–	–	–	2.9	34
SB-4	90	3	8.7	–	–	–	2.9	34
SB-5	120	4	8.7	–	–	–	2.9	30
SB-6	120	4	13.6	–	–	–	4.53	30
SBw-1	105	3.5	13.6	–	3.2	–	4.53	32
SBw-2	85	2.83	13.6	–	3.2	–	4.53	32

t_f : thickness of steel flange, t_w : thickness of web
t_d : thickness of diaphragm, , a : shear span,
S_d : spacing between diaphragms, $\rho_s = \frac{t_f}{d}$,
f'_c : compressive strength of concrete
b : width = 250 mm, d : height = 300 mm,

Table 2 Steel Properties

Element	t	f_y	E	f_t
Plate 1	8.7	337	193	504
Plate 2	13.6	342	197	522
Diaph.	4.4	290	193	403
Web	3.2	390	193	489

t : thickness of the element, mm ;
f_y : yield strength, MPa ;
f_t : tensile strength, MPa ;
E : Young's modulus, GPa

3. TEST RESULTS AND CONSIDERATION

Initial flexural cracking loads, diagonal cracking loads, steel yielding loads and ultimate loads are given in *Table* 3. And load-midspan deflection curves are shown in *Fig*.2. Crack patterns are given in *Fig*.3. All specimens have shown several shrinkage cracks. Thus, it was difficult to detect initial flexural cracks since they initiated along the diaphragms or the shear connectors. In specimen $SB-1$ all flexural cracks followed the diaphragms. The first diagonal crack appeared at 215 KN. It started on the diaphragm 2 (near the loading point) at nearly 60 mm from the bottom of the beam. The second diagonal crack (333 KN) started on the diaphragm 3. Both of them propagated toward the loading point. Then, concrete started crushing at about 650 KN until the failure of the diaphragm 2. In the other specimens, flexural cracks appeared and propagated very rapidly. Comparing to a reinforced concrete beam, with the same steel ratio, the

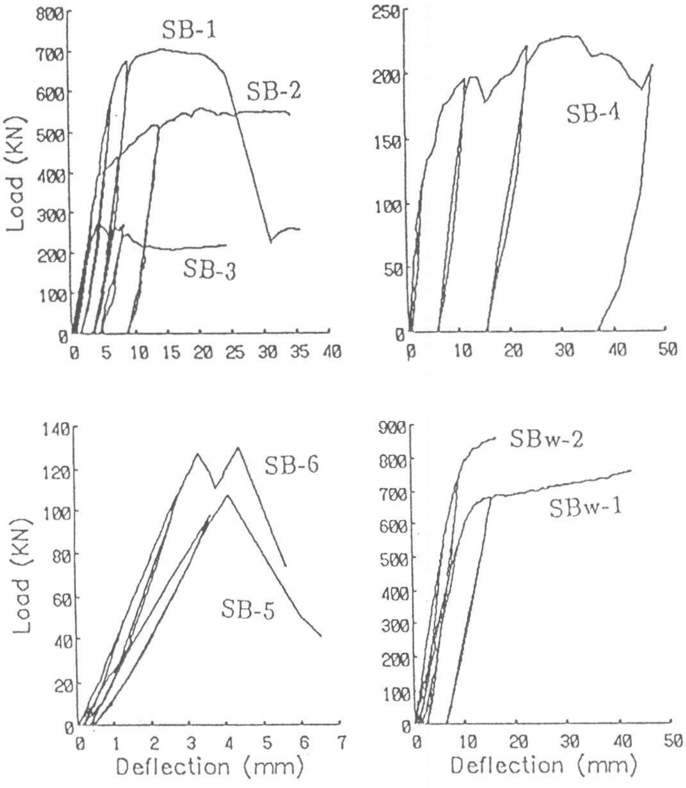

Fig.2 Load-Deflection Curves

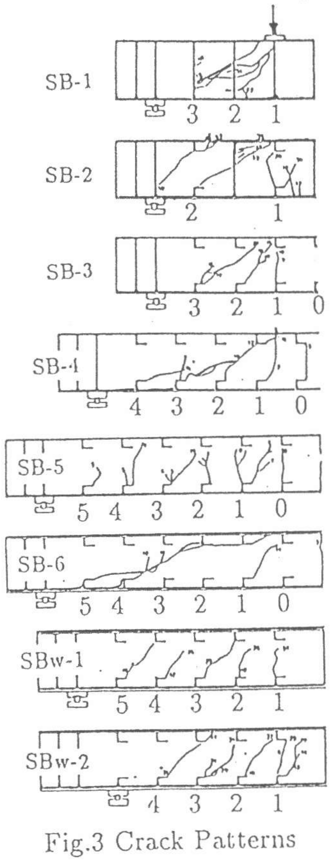

Fig.3 Crack Patterns

flexural cracking of sandwich specimens occurred very early as shown in *Table*3. In all specimens, the diagonal tension cracking started at the shear connectors and followed almost the same sequence. First, it started at the first shear connector near the loading point. Then another crack appeared at the second shear connector. In the case of the beams with large shear span-depth ratio ($\frac{a}{d} = 4$; $SB-5$, $SB-6$) and the beams with fullweb, the diagonal cracking was propagated from the shear connector at the bottom toward the shear connector at the upper plate making an angle of 45 - 53°. Then, an almost horizontal crack appeared at the support, followed shear connectors, joined the diagonal crack of the shear span center, followed the top shear connectors and reached finally the loading point. This caused the shear failure. In case of $\frac{a}{d} = 3$ $(SB-4)$, the first diagonal crack had an angle of 45°. The second and the third cracks joined the first one. The three specimens with $\frac{a}{d} = 2$ showed a tied arch behavior at their final stage before collapse. Here, it would be interesting to note that the second diagonal crack of specimen $SB-2$, unusually, moved from the top to the bottom.

4. DISCUSSION ON CALCULATION METHODS

Table 3 shows the calculation results done by several methods. Flexural capacity is computed by (A) using the equation of a reinforced concrete beam with compression plate neglected, (B) neglecting the compressive concrete of a doubled reinforced beam and (C) the Discrete Element Technique. (B) and (C) methods gave the same results

Table 3 : Experimental and Calculation Results

Specimen	Experimental Results						Calculation Results						
	P_{fcr}	P_{dcr}	P_{wy}	P_{sy}	P_{max}	Failure Mode	P_{fcr}	Flexural Capacity			Shear Capacity		
								A	B	C	D	E	F
SB-1	–	215	441	588	700	S,F	117	609	712	712	614	892	636
SB-2	117	127	294	441	550	S,F	117	609	712	712	450	728	472
SB-3	88	98	–	–	280	S	117	609	712	712	286	564	308
SB-4	19	78	–	190	230	S	78	406	474	475	240	392	//
SB-5	49	78	–	–	115	S	58	296	356	356	208	299	//
SB-6	15	78	–	–	132	S	83	405	555	555	250	474	//
SBw-1	59	118	579	353	805*	F,S	88	475	634	634	660	720	//
SBw-2	98	98	549	540	862*	F,S	110	587	784	784	681	792	//

P_{fcr} : load at first flexural crack, KN ; P_{dcr} : load at first diagonal crack, KN ;
P_{sw} : load at yielding of lower plate, KN ; P_{wy} : load at yielding of diaphragm or web, KN ;
P_{max} : ultimate load, KN ; * : ultimate load not reached.
A : load corresponding to flexural capacity using the assumption of an RC beam with compressive plate neglected, KN ; B : flexural capacity, compressive concrete neglected, KN
C : flexural capacity computed by descrite element technique, KN ; D : shear strength by Okamura's equation, KN ; E : shear strength by the equation proposed by Yokota and Kiyomiya ;
F : shear strength of a deep beam (JSCE), KN.

which are larger than (A). Since there is no clear flexural failure, the calculation results of the flexural strength are compared with the experimental results of the load P_{sy} which caused the lower plate to yield before the development of large strains in the shear reinforcement. Table 3 shows that the calculation results overestimate the flexural strength of the sandwich specimens. Thus the basic concept of reinforced concrete theory does not approximate the flexural strength of sandwich beams. In the calculation of the shear capacity, three methods are used. Comparing to the experimental ultimate loads it seems that Yokota and Kiyomiya's formula [2] (column E) overestimates the shear capacity of the specimens without fullweb. The comparison of the calculation by this equation with the experimental results of the specimens with fullweb shows that although the shear failure was preceded by the flexural failure, the loading force exceeded very much the calculated values without causing the appearance of the tied arch action. Thus it seems that the formula of Yokota and Kiyomiya and the one of Ozawa et al., which are almost similar and are assuming a tied-arch action, can not predict the ultimate loads of steel concrete sandwich beams. It seems also that these equations of the tied-arch action take only few parameters into account. For instance, both of them do not consider the effect of the compressive strength of concrete f_c' since the depth of the neutral axis, x, is inversely proportional to f_c'. The experimental ultimate load of specimen $SB-3$ which has a small shear span-depth ratio is a little smaller than the values calculated by the JSCE equation of the deep beam capacity (column F). Although Okamura and Higai's formula (column D) provides a relatively good estimation of the shear strength , it seems that, to fit better the experimental results, the contributions of some factors, such as the dowel action of the steel plates, may be reduced. To study this possibility, the factor β_p representing essentially the dowel action is multiplied by a factor α. Thus the equation becomes:

$$V_c = 200.\sqrt[3]{f_c'}(0.75 + \frac{1.4}{a/d})(1 + \alpha\beta_p + \beta_d)bd \quad (1)$$

and, using the experimental results, α is written :

$$\alpha = \frac{1}{\beta_p}\left[\frac{V_{exp} - V_s}{200.\sqrt[3]{f_c'}(0.75 + \frac{1.4}{a/d})bd} - 1 - \beta_d\right] \quad (2)$$

where α is the reduction factor of the dowel action, V_{exp} stands for the experimental shear capacity of one span, KN; V_s represents the contribution of the shear reinforcement, KN; $\beta_p = \sqrt{100\frac{A_s}{bd}} - 1$; $\beta_d = \sqrt[4]{\frac{1}{d}} - 1$; f_c' is the compressive strength of concrete, MPa; A_s is the tension steel plate area, m^2; and a, b and d stand for the shear span, the beam width and the depth respectively, m. As shown in *Table* 4, the coefficient α varies considerably from negative to positive values depending on many factors such as the shear reinforcement and the shear span-depth ratio. Thus such modification of the shear strength equation can not be done in this way. It needs the consideration of other factors such as the shear span, the depth, the spacing between the shear connectors, and the compressive strength of concrete since the cracks are gathered at the shear connectors and then the interlocking action of aggregates may be reduced. Moreover, it seems necessary to study more deeply the structural mechanism of sandwich members and establish a suitable design method.

Table 4. Reduction Coefficient of Dowel Action

Specimen	P_{exp}	P_s	α
SB-1	441	328	-0.75
SB-2	294	164	-0.57
SB-3	280	0	+0.94
SB-5	115	0	-0.30
SB-6	132	0	-0.04
SBw-2	549	393	+0.08

P_{exp} : shear capacity of the spacimen, KN
P_s : shear capacity of shear reinforcement,
α : reduction coefficient of dowel action

5. LOAD CARRYING MECHANISM

The calculation results of the neutral axis using the strains of both steel plates during the first loading steps (elastic behavior) do not correspond to the center of flexure of the elastic beam theory. The strains at transverse sections show that Bernoulli's principal is not applicable to the sandwich member. In fact, longitudinal strains in concrete and the steel plates are not proportional to the distance from the neutral axis. The calculation of the longitudinal forces of steel plates and concrete shows also that the transverse section of the sandwich beam is not in equilibrium. Furthermore, it was noted that after the appearance of the diagonal cracks, the strains of the upper steel plate showed some gradual change and became tensile strains. These experimental facts indicate that the steel-concrete sandwich beams do not have the same load carrying mechanism as the ordinary reinforced concrete beams. According to these facts, the sandwich composite beams have a truss-like mechanism. When the beam is loaded, diagonal compression struts are created between the upper and the bottom sides of the beam depending on several factors such as the compressive strength of concrete, the steel plate thickness, the existence of the fullweb, the diaphragms and the tie plates, the depth of the beam and the spacing of the shear connectors.

6. CONCLUSION

The experimental results of eight steel concrete sandwich specimens were described. It was concluded that the steel concrete sandwich members do not have the same behavior as the ordinary reinforced concrete beams but they may have a truss-like mechanism.

ACKNOWLEDGEMENT

The authors would like to express their thanks to Mr. J. Morikawa, undergraduate student, and Messrs. K. Satoh and S. Satoh, technicians, Civil Engineering Department, Saitama University, for their assistance during the course of the experiment.

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