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論 文

[2216] Steel-Concrete Sandwich Members without Shear Reinforcement

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1. INTRODUCTION

A steel-concrete sandwich structure is a composite structure in which double steel skins and concrete core behave monolithically. The steel plates have two roles of permanent formwork and reinforcement. This system has been applied to port and harbor facilities since the early 1980s, and has potential applications in submerged tunnel construction and also in nuclear containment and blast resistance structures [1]. To study further the basic structural behavior of steel-concrete sandwich structures, a series of tests has been undertaken. Parts of the tests have been already reported [2,3]. Here, two specimens are added and an attempt to describe the beam behavior is presented. This study shows also some analytical results concerning essentially the specimens without shear reinforcement and the influence of shear connectors.

2. OUTLINE OF TESTS

Ten simple beam specimens (such as SB-4-1 and SB-4-2 shown in Fig. 1) have been tested. The major parameters of the experiment programme were (1) reinforcing method against shear, (2) shear span to depth ratio, (3) thickness of steel skin plates and (4) size of shear connectors.

Steel plates used as shear reinforcement are linked to steel skin plates by fillet welding at both upper and lower sides. Diaphragms, which are shear reinforcing plates placed in planes normal to the member axis, are composed of double plates permitting the concrete core to have some continuity along the specimen axis. Three specimens SB-1 to SB-3 were designed to investigate the influence of shear reinforcing by diaphragms. Beams SB-3, SB-4 and SB-5 were used to study the effect of shear span to depth ratio. Then SB-5 and SB-6 were tested to evaluate the influence of the steel skin plates' thickness. Beams SBw-1 and SBw-2 were designed to investigate the effect of full steel web which

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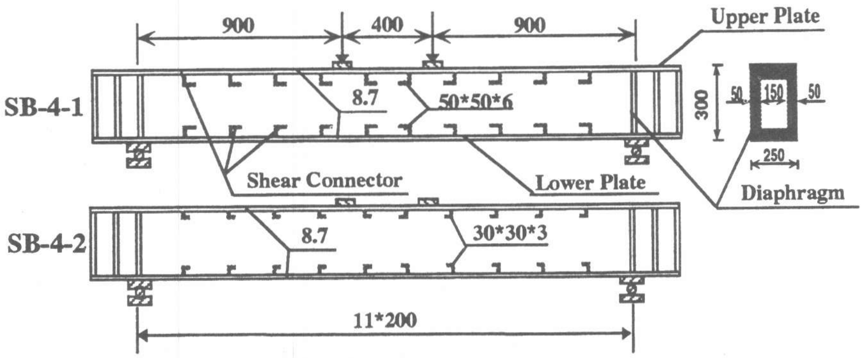


Fig. 1 Specimens SB-4-1 and SB-4-2

Table 1 : Specimens

Speci-men	a cm	a/d	t_f mm	t_d mm	t_w mm	S_d cm	ρ_s %	f_c' MPa	C.T.
SB-1	60	2	8.7	4.4	–	20	2.9	34	–
SB-2	60	2	8.7	4.4	–	40	2.9	34	1
SB-3	60	2	8.7	–	–	–	2.9	34	1
SB-4	90	3	8.7	–	–	–	2.9	34	1
SB-4-1	90	3	8.7	–	–	–	2.9	34	1
SB-4-2	90	3	8.7	–	–	–	2.9	34	2
SB-5	120	4	8.7	–	–	–	2.9	30	1
SB-6	120	4	13.6	–	–	–	4.53	30	1
SBw-1	105	3.5	13.6	–	3.2	–	4.53	32	1
SBw-2	85	2.83	13.6	–	3.2	–	4.53	32	1

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
C.T : L-shape connector type : $1 = 50 * 50 * 6$ mm and $2 = 30 * 30 * 3$ mm.

is a shear reinforcing plate placed in a plane parallel to member axis. And recently the size of shear connectors was tested by using two specimens SB-4-1 and SB-4-2. All specimens were simply supported and tested by a symmetric two-point loading system as shown in fig. 1. Table 1 indicates the experimental variables.

3. EXPERIMENTAL RESULTS

Some experimental and analytical results are given in Table 2. SB-1, SB-2 and SB-3, which had a small shear span to depth ratio a/d=2, had a tied-arch behavior at their final stage. SB-1 and SB-2 showed firstly large diagonal cracks, secondly their diaphragms yielded, and then as soon as this beginning of shear failure, a flexural failure started due to the yielding of the lower steel skin plate. Then the deflection increased rapidly until the cut of a diaphragm in the shear span near the loading point. All specimens without shear reinforcement failed in shear; SB-3 in a tied-arch mode and

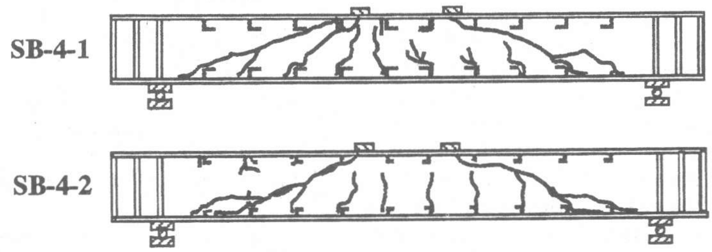


Fig. 2 Crack Pattern

Table 2 : Experimental and Calculation Results

Specimen	Experimental Results					Calculation Results					
	P_{dcr}	P_{wy}	P_{sy}	P_{max}	Failure Mode	Flexural Capacity			Shear Capacity		
						A	B	C	D	E	F
SB-1	215	441	588	700	S,F	609	712	712	614	892	636
SB-2	127	294	441	550	S,F	609	712	712	450	728	472
SB-3	98	–	–	280	S	609	712	712	286	564	308
SB-4	78	–	190	230	S	406	474	475	240	392	//
SB-4-1	90	–	–	165	S						
SB-4-2	110	–	–	158	S						
SB-5	78	–	–	115	S	296	356	356	208	299	//
SB-6	78	–	–	132	S	405	555	555	250	474	//
SBw-1	118	579	353	805*	F,S	475	634	634	660	720	//
SBw-2	98	549	540	862*	F,S	587	784	784	681	792	//

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 without compressive plate, kN ;
B : flexural capacity neglecting compressive concrete, kN ;
C : flexural capacity computed by discrete 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;
Failure Mode: S=shear failure, F=flexural failure.

the others (a/d=3 and 4) in a diagonal tension failure mode. Only specimens SBw-1 and SBw-2, with full web, showed a clear flexural failure mode. All cracks started at the shear connectors. From the initial loading state, the response of specimens to loads was not linear. Firstly, the concrete surrounding the shear connectors in the tension zone cracked causing a progressive loss of stiffness of concrete core. Secondly, since the longitudinal connection between the plates and the concrete was not rigid, the slipping of the connection caused a further reduction of stiffness. Steel plates and shear connectors of the specimens without shear reinforcement (SB-3 through SB-6) remained elastic until the loss of shear capacity of concrete.

4. EFFECT OF SHEAR CONNECTORS

Fig.3 shows the experimental and analytical results of load-deflection curve of SB-3. Fig. 4 shows the experimental and analytical load-deflection curves of three specimens SB-4, SB-4-1 and SB-4-2. Wright and Oduyemi [1] have used stud connectors in their research on steel-concrete sandwich structures. These were considered to transmit shear stress between steel and concrete, to prevent diagonal tension cracks in the same way as stirrups in reinforced concrete members and to prevent buckling of compression steel plate. In Japan, where a steel structure of a sandwich member is supposed to sustain dead loads during construction work [4], many researchers have been using L-shape shear connectors. These are considered not only to fulfill the role of shear transfer between concrete and steel but also to strengthen and prevent the deformation of steel skin plates during placing of concrete. Unfortunately the shear connectors also have a detrimental effect as they act as crack inducers and weaken the shear resistance of concrete. In all specimens of the actual programme, except SB-1, L-shape shear connectors (angles) had been used. Specimens SB-4 and SB-4-1 had the same steel structure with angles of 50*50*6 mm. Their concretes had the same compressive strength of 34 MPa but different slumps of 18±2 and 15±2 cm respectively. The concrete in SB-4-1 had also better placing and curing conditions than the one of SB-4. Thus the concrete contouring the shear connectors was little weaker in SB-4 than in SB-4-1 and then had less load transfer capability and caused larger slip between concrete core and steel in SB-4 as shown in Fig.4.

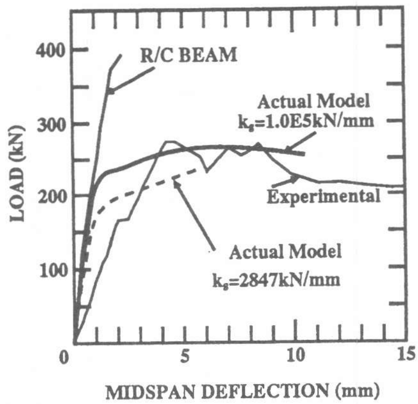


Fig.3 LOAD–DEFLECTION, SB–3

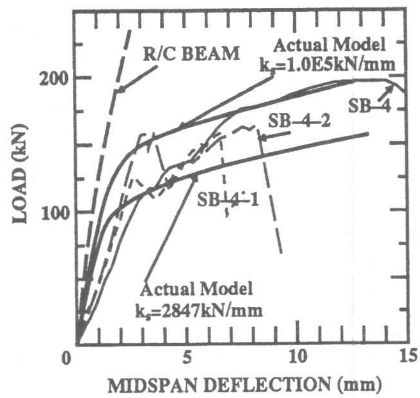


Fig.4 LOAD–DEFLECTION, SB–4

The only structural difference between SB-4-1 and SB-4-2 was the connector size with 30*30*3 mm in the case of the later. In SB-4-2 the first large diagonal crack with a large loss of stiffness appeared at a total load of 157 kN while SB-4-1 had a sudden large loss of stiffness at 125 kN only. This may be attributed to the fact that small shear connectors have less detrimental effect on concrete than large ones.

5. FINITE ELEMENT ANALYSIS

A nonlinear FEM program has been developed. It is composed of the reinforced concrete plate model used in WCOMR (which is a program for nonlinear analysis of reinforced concrete walls [5]), a bilinear beam element for steel plates, and a linkage element model. The computation technique is based on "Frontal Skyline" method. This uses simultaneously the skyline storage, which is an efficient technique to reduce both the computation time and the storage requirements, and the frontal procedure which permits to reduce the half band-width of the global stiffness matrix [6].
Firstly the specimens were analyzed as doubly reinforced concrete beams with full contact between concrete core and steel plates. Secondly they were assumed to have contact

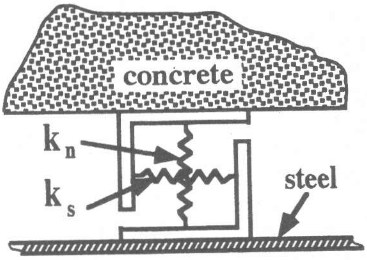


Fig. 5 Link Model

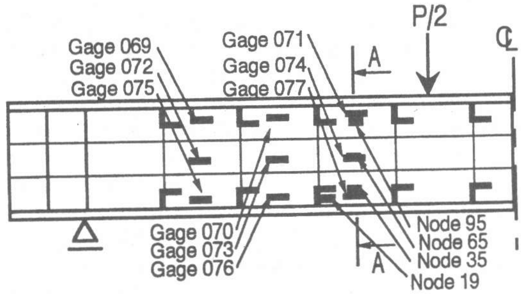
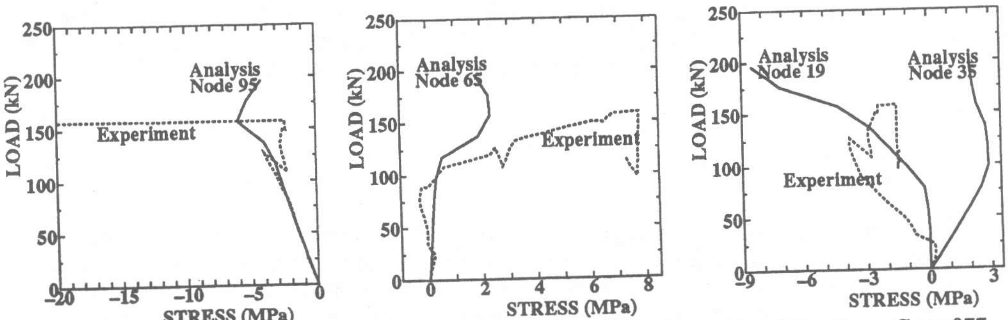


Fig.6 Gages and Nodal Points Locations



a) Position : Gage 071 b) Position : Gage 074 c) Position : Gage 077

Fig. 7 HORIZONTAL STRESS IN CONCRETE, SB–4–1

only at locations of connectors and shear reinforcement. The contact is modeled by a linkage element consisting of two springs among which one represents the normal action and another the shear force between concrete and steel as shown in Fig.5. The stiffness coefficient relative to normal action (denoted as k_n) was taken very large to avoid analytically "punch" of upper skin plate into concrete. The coefficient representing the shear rigidity (denoted as k_s) seemed to have an important influence on the beam behavior. Then, to test this effect, several values were attributed to k_s for linkage elements at the lower side of specimens (tension zone). A large coefficient $k_s = 1.0E7$ kN/mm was used for the compressive zone which was considered to have high shear rigidity due to the compression of concrete. And, due to the low strength of concrete near the connectors, concrete compressive and tensile strengths were reduced by 50% in the tension zone and 20% in the compression area but not reduced at central layers.

6. DISCUSSION ON ANALYTICAL RESULTS

Some analytical results are shown in Figs 3, 4 and 7. With the experimental load-deflection curves three numerical results are drawn. These are (1) result of the specimen assumed as a reinforced concrete beam showing an almost linear response to loads with a sudden failure, (2) result of a member using the actual model with a shear rigidity $k_s = 2847$ kN/mm (which is considered to be large since it is equal to the maximum of the values obtained by Yamada and Kiyomiya for an L-shape mixed with a stud connector [7]), and (3) result of the actual model using a large $k_s = 1.0E5$ kN/mm. Fig. 7 shows the horizontal stresses in concrete measured by mold gages and analytical results of the nodes located almost at the same positions as the corresponding gages as shown in Fig. 6. Referring to experimental stress values, it is seen that point at upper side is in compression, the second which is in the center is near the neutral axis and that lower side is in compression, too. Then the transverse section A-A does not verify Bernoulli's principle. This is attributed to the fact that shear forces are transferred by shear connectors only. Fig. 7 also shows an acceptable agreement between experimental and analytical results in case of the upper (compression) and the central zones of the member. But the analytical results do not agree with the stresses measured in the tension zone. Figs. 3 and 4 show that shear rigidity k_s of linkage elements has a large influence on the member behavior and it can be seen that the lower values of k_s cause larger deflections at the beginning of loading and the higher values of k_s give rise to higher ultimate load. It is seen also that experimental deflections are clearly larger than analytical ones at low level of loads. This may be attributed to the fact that concrete contouring shear connectors may have some defects due to concreting difficulties such as voids and shrinkage cracks and then has a low resistance capability at the beginning. And then, as the load is increasing the compression zone behind the connector increases also and gets stronger resistance. Thus it would be important to modify the linkage element permitting the coefficient k_s to follow the variation of concrete rigidity.

7. CONCLUSION

Some experimental and analytical results of steel-concrete sandwich beams have been described. Beams without shear reinforcement had a shear failure in a tied-arch mode when shear span to depth ratio a/d was 2 and in a diagonal tension failure mode when a/d=3 to 4. Concrete may have some defects and cause a small shear rigidity of connectors in tension zone at the beginning of loading. Further work on linkage element model is currently under way.

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REFERENCES

1. Wright, H. and Oduyemi, T., "Full Scale Tests on Double Skin Composite Elements," Third International Conference on Steel-Concrete Composite Structures, Fukuoka, Japan, Sep. 26-29, 1991.
2. Malek, N. et al., "Mechanical Properties of Steel-Concrete Composite Member," Transactions of JCI, Vol. 14, 1992, pp.469-474.
3. Machida, A. et al., "Experimental Study on Mechanical Properties of Steel-Concrete Sandwich Beam," Transactions of JCI, Vol. 14, 1992, pp.475-482.
4. JSCE Research Subcommittee on Steel-Concrete Sandwich Structures, "Design Code for Steel-Concrete Sandwich Structures -draft-," JSCE, Concrete Library International, no. 20, Dec. 1992.
5. Okamura, H. and Maekawa, K., "Nonlinear Analysis and Constitutive Models of Reinforced Concrete," Gihodo, 1990.
6. Thompson, E. and Shimazaki, Y., "A Frontal Procedure Using Skyline Storage," Int. J. Numerical Methods in Eng., Vol. 15, 1980, pp 889-910.
7. Yamada, M. and Kiyomiya, O., "Experimental Study on the Loading Capacity of L-Shape and Headed Stud Shear Connectors for Composite Structures," Transactions of JCI, Vol. 14, 1992, pp.461-468.