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STRENGTH AND CRACK BEHAVIORS OF STEEL-CONCRETE COMPOSITE BEAMS

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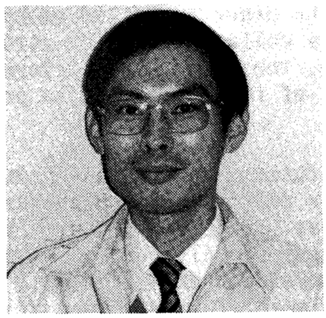

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SYNOPSIS

Composite structures with steel plate and concrete have considerable potential for use in offshore facilities. In those structures, steel plates are provided either on one surface only or both surfaces of structural members. For composite structures, adequate shear reinforcement is required to obtain appropriate mechanical properties. Flexural and shear resisting mechanisms have to be confirmed. This paper presents the results of flexural and shear loading tests on composite beams with newly proposed shear reinforcement. Furthermore, load carrying mechanisms against shear and flexural forces are investigated and their design methods are established on the basis of the test results. The flexural resisting mechanism of the composite beam was considered to be almost the same as that of an ordinary reinforced beam. Shear resisting mechanism was the tied-arch action, and the shear strength of the beam could be predicted by the crushing of strut concrete and yield of shear reinforcement.

Keywords: steel-concrete composite beam, loading tests, flexure, shear, crack

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1. INTRODUCTION

Composite structures with steel and concrete are often used in bridge decks and in buildings. In facilities under marine environments, structures made by combining steel plates and reinforced concrete have considerable potential for use, and have recently been introduced to breakwater caissons, seawalls, pontoons, and tunnel elements. A structure with steel on one side and concrete surface on the other is called an open-sandwich, while that with steel plates on both sides is called a sandwich structure. By replacing reinforcing bars with steel plates, the structural performance of the member could be almost unchanged and the cost of form work and the period of construction could be considerably saved.

Marine structures made of open-sandwich or sandwich members are expected to have such superior advantages as high strength and toughness, lightweightness, and watertightness. These merits will bring safer and more economical construction in offshore areas. Although a few guidelines regarding the design on composite structures have been published [1], the structural mechanisms have not been fully made clear. For this type of composite member, in particular, adequate shear reinforcement is required because the shear connectors will inevitably initiate cracks [2]. Shear resisting mechanism of the member have to be investigated and new methods of shear reinforcement have to be developed.

This paper presents the results of flexural loading and shear loading tests on composite beams with shear reinforcements. Under the stress flow along the cracks occurring from shear connectors, load-bearing mechanisms will be investigated and calculation methods on load-carrying capacity of a composite beam will be established on the basis of test results. Furthermore, calculation on buckling strength of the steel plate and crack widths will also be discussed.

2. STEEL-CONCRETE COMPOSITE STRUCTURES

The typical composite structures are depicted in Figure 1, which are an open-sandwich structure and a sandwich structure. Both types of structures are expected to have the following excellent characteristics in marine areas, while their detailed structural mechanisms and properties will be made clear in this paper:

(1) They have high strength and rigidity. Thus, thin section may achieve their required behaviors. This makes it possible to construct a complicated facility and to have many varieties of its size and shape. Furthermore, lighter weight is advantageous for structures to be constructed on soft sea mud.

(2) The outer steel plate prevents sea water from seeping in when cracks form. This greatly contributes to a floating structure and an undersea tunnel.

(3) Bar installment, form work, and false work could be very much reduced. Skilled workers on such work are not essential to fabricate a structure.

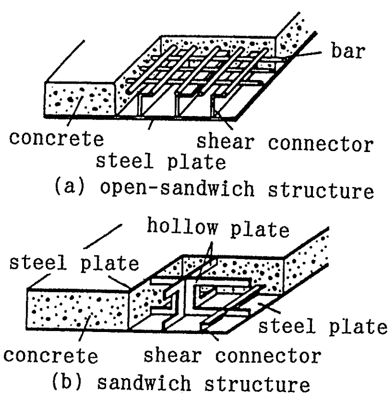


Figure 1 Typical composite sections

Furthermore, each structural element can be fabricated in a factory as a prefabricated structure. This can cut down the construction period on the site, often on the sea.

(4) Although material cost is not always low because of high price of steel work, total cost could be reduced due to rapid construction, small labor work on fabricating, and so on.

The above superior characteristics will be obtained only when proper combination of steel plates and concrete is made. The authors have been using structural steel such as angles for shear connectors instead of normal headed studs. The structural steel can make steel plates so stiff that extra formwork and falsework could be reduced. The structural properties of the steel for use as shear connectors are made clear by the authors [3].

The subjects which should be examined at the basic design stage on composite marine constructions are as follows:

(1) shear transfer of shear connectors,
(2) ultimate flexural strength and ultimate shear strength,
(3) fatigue endurance due to wave forces [4],
(4) structural details at connections [5,6], and so on.

This paper refers to the ultimate strength and crack properties of composite structures.

For the open-sandwich structure, concrete is directly exposed to sea water and the steel plate is not. That is, durability of the structure may not be serious when concrete cover thickness for embedded steel bars and the maximum crack width are considered in design as in an ordinary reinforced concrete. On the other hand, for the sandwich structure, the steel plate may corrode. In particular, special attention has to be paid for application in severe circumstances such as in the splash zone. Heavy protection such as cathodic protection is essential. In the circumstances when the structure is always submerged in sea water, little oxygen is supplied and there will be less possibility of corrosion [7].

3. OUTLINE OF TESTS

3.1 Test Beams

Structural details of the test beams are shown in Figure 2. All beams have the same geometric proportions (27 cm in width, 25 cm in depth, and 350 cm in length). They are designated HS-1 through HS-7 in accordance with the method of the shear reinforcement as shown in Figure 2. HS-1 through HS-5 are open-sandwich beams in which the bottom surface of the specimen is a steel plate of 6 mm in thickness. Furthermore, two reinforcing bars of 19 mm in diameter are arranged at 5 cm below the top surface. HS-6 and HS-7 are sandwich beams in which two steel plates are arranged at the top and the bottom surfaces. Angles with equal legs of 50 by 50 by 6 mm are used as the shear connectors. They are attached to the steel plate by the fillet welding.

Reinforcing bars or steel plates are used as the shear reinforcements. Beams HS-1 through HS-7 have different methods of shear reinforcement. In HS-1, stirrups of reinforcing bars of 16 mm in diameter are arranged, which are welded to the tensile steel plate at their toes of 50 mm in length. HS-2 has almost similar reinforcement as HS-1, but stirrups are welded to the flange of angles.

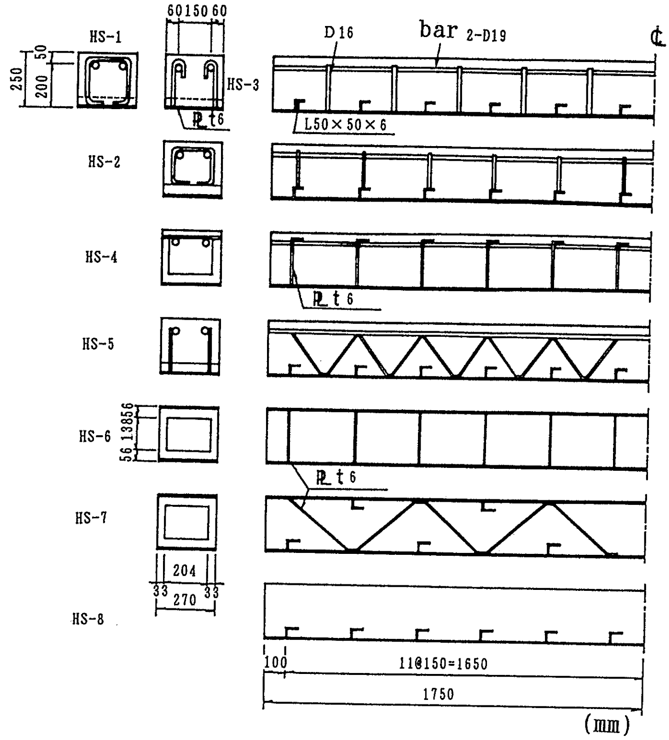


Figure 2 Test specimens

HS-3 uses J-shaped long bars of 16 mm in diameter. In HS-4, HS-6, and HS-7, steel plates of 6 mm in thickness are arranged as shear reinforcement. The middle of the steel plate is cut out so as to make the same effective area of shear reinforcement as that of the reinforcing bar. The intervals of shear reinforcement are basically 300 mm.

HS-8 is an open-sandwich and HS-9 is a sandwich beam. Both beams have no shear reinforcement and were made for the purpose of discussing the effect of shear reinforcement. The breadth of both the beams is 400 mm. Furthermore, crack formation will be examined with HS-8. For this purpose, T shapes, channels, and headed studs as well as angles are used as shear connectors. In beams with angles as shear connectors, the intervals of shear connectors are 200 mm, 300 mm, and 500 mm.

3.2 Materials

The mechanical properties of steel are presented in Table 1. The concrete mix was designed for the maximum aggregate size of 10 mm, slump of 8±2.5 cm, and air content of 5±1 %. The compressive and tensile strengths of concrete are 267 kgf/cm^2 and 43.3 kgf/cm^2 for the test beams of a/d=4.8 (described thereafter), and 287 kgf/cm^2 and 47.6 kgf/cm^2 for the beams of a/d=1.8 respectively. They were obtained by the strength test using ϕ10×20 cm cylinders and 10×10×40 cm prisms.

Concrete was placed from the side surface of the beam to avoid as much as possible degradation of concrete at the shear connectors due to bleeding.

Table 1 Mechanical properties of steel

type	yield stress (kgf/mm^2)	tensile strength (kgf/mm^2)	elongation (%)
angle	32	46	27
plate	39	48	27
bar (D19)	39	57	21
bar (D16)	38	55	22

Table 2 Summary of test results

No.	a/d	P_{max} (tf)	δ_u (mm)	P_{cr} (tf)	P_{bu} (tf)	mode of failure
HS-1	4.8	20.5	31.6	-	-	flexure
HS-2	4.8	18.0	21.5	-	-	flexure
HS-3	4.8	20.8	31.6	-	-	flexure
HS-4	4.8	21.0	23.4	-	-	flexure
HS-5	4.8	20.9	26.8	-	-	flexure
HS-6	4.8	21.7	19.1	-	21.7	flexure
HS-7	4.8	20.9	21.5	-	13.0	flexure
HS-8	4.8	13.7	-	-	-	shear
HS-1	1.8	54.1	10.7	47.0	-	shear
HS-2	1.8	52.0	10.5	52.0	-	shear
HS-3	1.8	59.8	15.6	52.0	-	shear
HS-4	1.8	56.2	11.0	56.0	-	shear
HS-5	1.8	54.9	8.8	43.3	-	shear
HS-6	1.8	62.5	12.6	-	61.9	flexure
HS-7	1.8	57.0	11.0	48.0	19.0	flexure, shear

P_{max}: maximum load δ_u:midspan deflection at P_{max}
P_{cr} : load at first diagonal cracking
P_{bu} : load at buckling of the steel plate

3.3 Loading Procedure and Instrumentation

The shear span / depth ratio (a/d) are 4.8 and 1.8 in this study. The beams were all tested on a two-point loading system with monotonically increasing loads.

Deflection of the specimen, strains of the steel plate and the reinforcing bar were measured at every load stage throughout the loading. The deflection was measured by displacement transducers in 50 mm capacity. The strains were measured at several points along the span using electric resistance strain gauges of 30 mm long for concrete and of 6 mm long for steel. The strain gauges for the steel plate were attached on its outer surface at the center of the beam width. The crack widths were measured by displacement transducers, which were installed on the side surface of the beam at 5 cm high from the bottom. The crack propagation was observed and sketched by visual inspection.

4. TEST RESULTS AND CONSIDERATION

The test results are summarized in Table 2. The results of HS-8 and HS-9 are also presented in this table, which was obtained by the authors' previous tests on sandwich beams without shear reinforcement [2]. Moreover, to compare with the present results, the ultimate force is modified in accordance with the present width of the specimens.

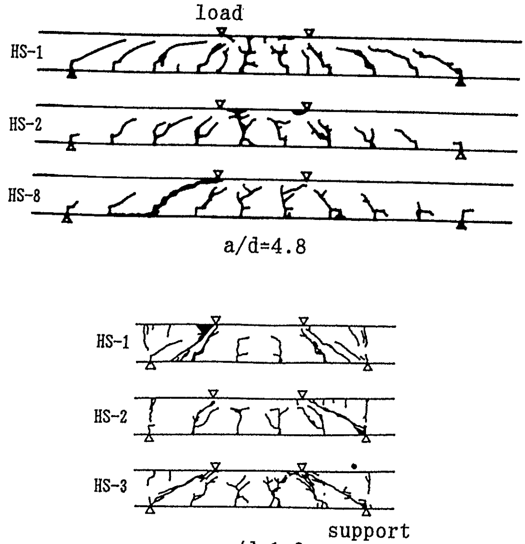


Figure 3 Crack formation

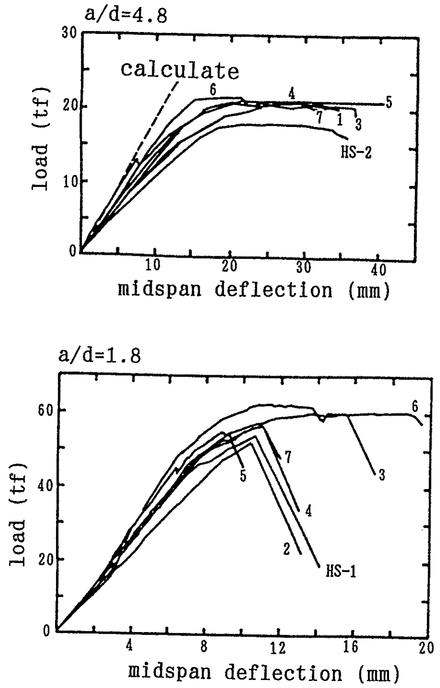


Figure 4 Load-deflection curves

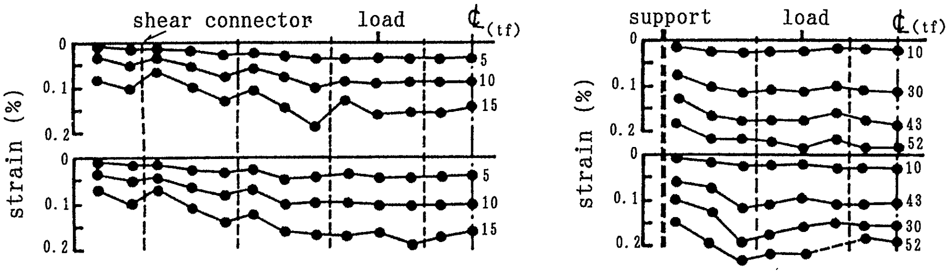
4.1 Cracking and Failure Mode

The crack formation of the specimen at ultimate is shown in Figure 3. All the flexural cracks were initiated at the shear connectors. Furthermore, additional failure cracks were formed between the two shear connectors after the tensile steel plate yielded.

In the case of a/d=4.8, diagonal tension cracks occurred in the shear span. Diagonal cracks did not pass through any sections of HS-1 through HS-7 with shear reinforcement, and the crushing of concrete occurred. In HS-8, however, these cracks reached the top surface (compression fiber of concrete) and the specimen suddenly failed. The composite beam specimens had cracks from the end of the shear connector at the tensile steel plate to the loading point and/or to the shear connector at the compressive steel plate.

In the case of a/d=1.8, almost all the specimens had diagonal tension cracks which passed through some sections. Moreover, after the crack occurred, typical flexural shear failure occurred. In both cases of a/d, HS-6 and HS-7 had the buckling of compressive steel plate near the ultimate stage.

In the sandwich beams (HS-6 and HS-7), the compressive steel plate was buckled. The buckling loads were 21.7 tf for HS-6 and 13.0 tf for HS-7 in case of a/d=4.8. In case of a/d=1.8, those were 61.9 tf for HS-6 and 19.0 tf for HS-7. When buckling occurred, HS-6 reached the ultimate stage but HS-7 had strength enough to carry further loads.



a/d=4.8 (above: HS-1, bottom: HS-2)

a/d=1.8 (above: HS-2, bottom: HS-3)

-------- shear connector

Figure 5 Strain of tensile steel plates

4.2 Deflection

Load-midspan deflection curves are shown in Figure 4. In the case of a/d=4.8, midspan deflection still increased after the maximum load applied was confirmed, and conspicuous peak of the curve was not observed. The ultimate load of each specimen was almost the same except for those of HS-2 and HS-8. That of HS-8 was considerably smaller than those of the others because shear failure occurred. Though buckling of the compressive steel plate occurred in HS-6 and HS-7, the load-carrying capacity was not degraded after the buckling.

In the case of a/d=1.8, the rapid decreasing in load applied after ultimate was observed. The maximum load applied in HS-6 was larger than those of the others. The calculated deflection of the specimen is also shown in Figure 4, and it was smaller than those of the test results.

4.3 Strain of Steel

Figure 5 shows the distribution of measured strains of the tensile steel plates along the span at a few load stages. The strain at the failure of the beam was larger than the yield value (0.18 %); that is, the tensile steel plate was confirmed to become yield in all the tests.

In the case of a/d=4.8, almost constant strains were obtained within the constant moment span. In the shear span, strains varied almost linearly, but gave discontinuity at the shear connectors. Therefore, flexural resisting mechanism of the beam may be considered to be almost the same as that of a conventional reinforced concrete beam. Moreover, transmission of forces between concrete and steel plate was conducted not by bond but mainly by the shear connectors.

In the case of a/d=1.8, after the diagonal tension crack occurred, almost constant strains were measured except for at the supports. Moreover, stress concentration occurred as the shear connector in some of the beams. Therefore, the shear resisting mechanism may be based on the tied-arch action. Almost constant stress was induced in the tensile steel plate which balanced the compressive stress along the diagonal crack.

The distribution of strains along the midspan section is shown in Figure 6. Through the loading stages, the strain measured at 20 cm below the top surface was nearly equal to be zero. Moreover, the distribution of the other three

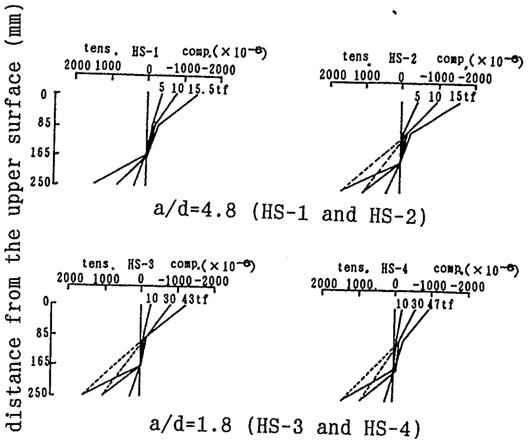


Figure 6 Distribution of strain in the midspan section

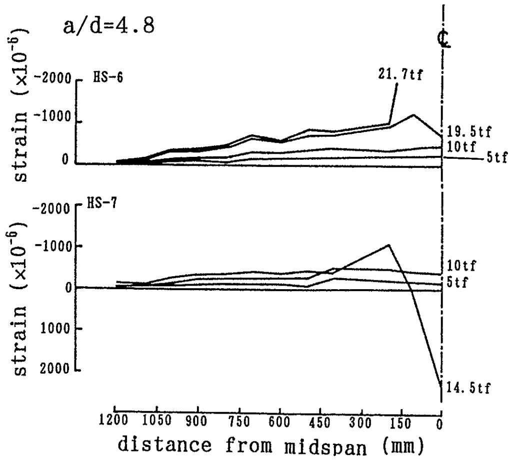


Figure 7 Distribution of strain of the compressive steel plate

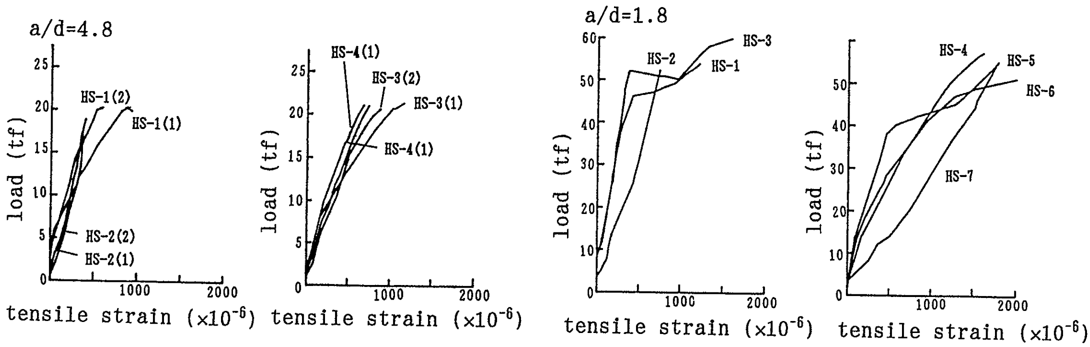


Figure 8 Strain of the shear reinforcement

measured strains was almost linear. Therefore, ordinary equation for obtaining the flexural strength of a reinforced concrete beam may be applied to the beams.

The distributions of strain of the compressive steel plate of HS-6 and HS-7 along the span are shown in Figure 7. At the load of 10 tf applied, the upper surface of the plate had maximum strains of 500×10^{-6} in the case of a/d=4.8 and 200×10^{-6} in the case of a/d=1.8. The maximum strain was observed near the shear connector not at the midspan of the beam. In HS-7, after buckling began, the strain became tensile; that is, the compressive plate did not contribute much to the overall load carrying capacity of the beam.

Measured strains of the shear reinforcement are shown in Figure 8. Those values in HS-3 through HS-7 reached the yield values in the case of a/d=1.8. After diagonal tension cracks occurred, and then compressive stress of diagonal strut along the crack reached a certain value (probably compressive strength), the stress of the shear reinforcement reached yield value and the specimen failed. The measured strain of HS-7 was larger than those of the others because of the inclined arrangement of the shear reinforcement for HS-7. On the other hand, in the case of a/d=4.8, their ultimate strains were about 0.1% and smaller than the yield values.

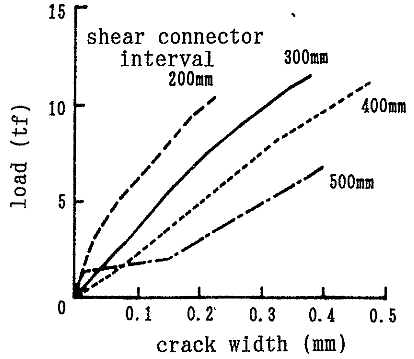


Figure 9 Effect of shear connector intervals on crack width

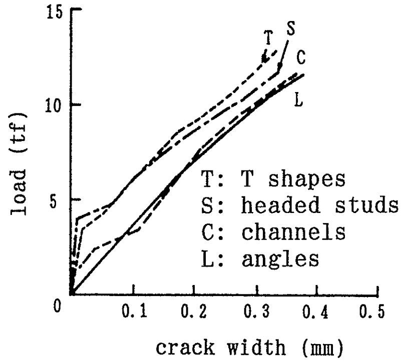


Figure 10 Effect of shear connector shapes on crack width

Table 3 Flexural resisting load (a/d=4.8)

No.	test (tf)	calculated (tf)				test/cal.
		A	B	C	D	
HS-1	20.5	20.3				1.01
HS-2	18.0	20.3				0.87
HS-3	20.8	20.3				1.02
HS-4	21.0	20.3				1.03
HS-5	20.9	20.3				1.03
HS-6	21.7		21.0	25.7	24.0	1.03*
HS-7	20.9		21.0	25.7	24.0	1.00*

*) B as the calculated result

4.4 Crack Width

Relationship between applied load and crack width is shown in Figure 9. The shear connector intervals are varied in this figure. Crack widths were smaller as the intervals decreased, and had linear relationship.

Figure 10 shows the crack width under different shapes of shear connectors. Applied loads to initiate cracks in beams with T-shapes and headed studs were slightly smaller than those of the other beams. Crack widths, however, showed almost the same increment regardless of the kinds of shear connectors. As described before, cracks were concentratedly initiated at the tip of shear connectors, but those cracks did not cause harm in the beams.

5. ULTIMATE STRENGTH

On the basis of these presented test results, flexural strength, shear strength, buckling strength, and crack widths will be discussed.

5.1 Flexural Strength

Composite beams in this study follow the established behavior of equivalent standard reinforced concrete beams. Flexural strength may be predicted by the same equation as used in a conventional reinforced concrete beam. Table 3 presents the tested and calculated strengths in the case of a/d=4.8. In the calculation, the conventional limit state design method was applied.

To the beams of HS-1 through HS-5 which have only one plate, the calculated and the experimental values showed good agreement (A in Table 3). To HS-6 and HS-7 beams, since concrete cover to the plate was zero and the areas of the two plates were equal, normal equation cannot be applied directly. In this study, the following three assumptions were considered:

(B) compressive plate was neglected,
(C) iteration was conducted by lifting the assumed neutral axis, and
(D) compressive stress of concrete was neglected.

During the above calculation, bi-linear stress-strain relationship was assumed for calculating stress of steel. As presented in Táble 3, the calculated results with (C) and (D) are about 20 % larger than the experimental results. By assumption (B), the calculated and the test values showed good agreement. This is because the compressive steel plate lost strength due to buckling, which has been confirmed the results shown in Figure 7.

5.2 Buckling Strength

Buckling strength of the beam which has the compressive plate will be considered. In the test, the mode of buckling of the compressive plate occurred at the fixed end of the shear connectors. Then Euler's equation for the column with fixed ends may be applied. The buckling stresses of HS-6 and HS-7 were 2763 and 307 kgf/cm^2, respectively. Bending moment which induced those stresses was calculated. The calculated loads inducing those stresses are 21.0 tf for HS-6 and 3.7 tf for HS-7. The calculated and the experimental values (21.7 tf) of HS-6 showed good agreement. In HS-7, however, the experimental result (13.0 tf) was considerably larger than the calculated one. Full investigation regarding the buckling is further required.

5.3 Shear Strength

Shear resisting mechanism is principally tied-arch action. Ultimate shear strengths of the specimens in the case of a/d=1.8 are presented in Table 4, which were obtained from the calculation and the experiment.

The shear force (V_s) contributed by shear reinforcement was obtained by the following equation [10] for traditional truss mechanism.

$$V_s = A_w \cdot f_{wy} (\sin\alpha_s + \cos\alpha_s)/s_s \cdot z \qquad \text{.....(1)}$$

where, A_w : area of shear reinforcement
f_{wy} : yield stress of shear reinforcement
α_s : angle of shear reinforcement to member axis
s_s : intervals of shear reinforcement
z : distance between resulting compressive force and tensile plate

In the present calculation, the following three past equations were used to calculate the shear forces (V_c) contributed by factors other than shear reinforcement:

(A) presented in the JSCE Standard for Concrete Structures in which all safety factors are set equal to 1.0 [10],
(B) the empirical equation obtained by Ozawa et al. [11],
(C) the empirical equation to deep beams obtained by Niwa [12], and
(D) the proposed equation by the authors.

Thus, the two values were added and then the shear strength of the specimen were obtained.

Table 4 Shear resisting load (a/d=1.8)

No.	test (tf)	calculated (tf)				test / cal.*
		A	B	C	D	
HS-1	54.1	40.4	57.1	51.5	53.9	1.00
HS-2	52.0	40.4	57.1	51.5	53.9	0.96
HS-3	59.8	40.4	57.1	51.5	53.9	1.11
HS-4	56.2	40.4	57.1	51.5	53.9	1.04
HS-5	54.9	49.1	59.7	60.2	53.9	1.02
HS-6	62.5	39.0	68.2	51.5	61.7	1.01
HS-7	57.0	47.7	76.9	60.2	61.7	0.92

*) D as the calculated result

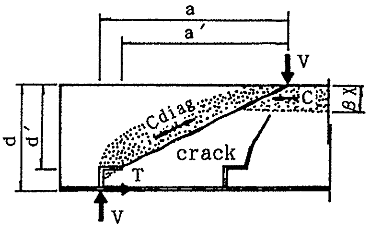


Figure 11 Shear resisting mechanism

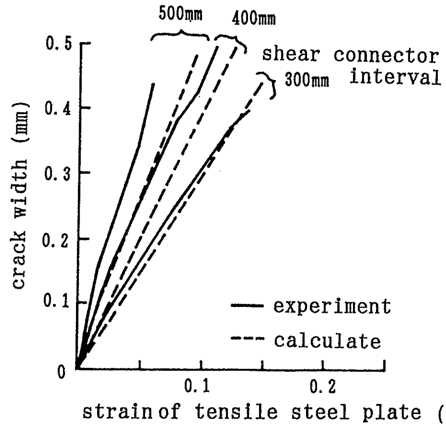


Figure 12 Relationship between crack width and strain of the steel plate

By using (A), the calculated values were rather smaller than the experimental ones, but by using (B) and (C), the calculated values were almost the same as the experimental ones.

Considering the existence of the shear connectors as shown in Figure 11, the shear span (a) and the effective depth (d) were modified as a' and d', respectively. When compressive stress along the diagonal tension crack reaches the ultimate value, shear failure of the beam occurs.

$$V_c = b \cdot k_1 \cdot f'_c \cdot \beta x \cdot \sin\alpha \qquad .....(2)$$

where, b : width of a beam
k_1 : coefficient for reduction in strength of concrete (= 0.85)
β : coefficient expressing compressive zone of concrete (= 0.8)
f'_c : compressive strength of concrete
x : neutral axis depth at ultimate
α : angle ($\tan\alpha$ = d'/a')

The calculated results under assumption (D) are presented in Table 4. The validity of the test results to those by the equation was confirmed.

5.4 Crack Width

An equation for calculating crack widths in composite beams has not been established so far. In the present paper, the following assumption was made to calculate it.

(1) Crack intervals are equal to the interval of shear connectors because cracks will be initiated at the tip of the shear connector.
(2) Strain in the concrete is equal to that of steel plate between the two shear connectors because the concrete and the plate will be perfectly bonded when applied loads are small.

The crack width is the product of strain of the plate and the interval of the shear connectors. The relationship between the calculated crack width and average strain of the plate is shown in Figure 12. In this figure, the strain of the plate was measured at the center between the two shear connectors.

When the intervals of the shear connectors are 300 mm and 400 mm, the calculated and the experimental values were almost the same. In case of 500 mm, however, the calculated crack width was slightly smaller than the experimental value. The above procedure to calculate crack width was confirmed to be applied to normal intervals of shear connectors.

6. CONCLUSIONS

On the basis of the results presented herein, the following conclusions may be drawn regarding the composite beams:

(1) In the case of a/d=4.8, flexural failure clearly occurred; that is, yield of the steel plate and crushing of concrete occurred. No failure occurred at the area of shear connectors. All the beams had almost the same ultimate loads regardless of the method of the shear reinforcement proposed herein.

(2) The beams without shear reinforcement collapsed due to shear forces at the small stages of applied loads. In composite beams, adequate shear reinforcement should be necessary to obtain excellent mechanical characteristics. The procedure of the shear reinforcements presented in this paper was effective for preventing the beam from shear failures.

(3) The ultimate load of the beam with adequate shear reinforcement was predicted by the method of a conventional reinforced concrete beam. To calculate the ultimate flexural load on the sandwich beam, it was made clear that the compressive steel plate can be neglected.

(4) In the case of a/d=1.8, shear failure occurred after the initiation of diagonal concrete cracks at the shear connector on the support, and the shear reinforcement yielded.

(5) The shear strength of the beam which was adequately reinforced against shear forces may be calculated by tied-arch mechanism considering compressive strength of concrete and the existence of shear connectors.

(6) Flexural cracks were initiated at the tip of shear connectors. Cracks were concentrated there and their widths became large. Crack width was predicted accurately by multiplying the tensile strain of the steel plate and the shear connector intervals.

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