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STUDY ON THE JOINT AND BOND STRENGTH OF SPLICED BARS USING SPIRAL SHEATHS FOR PRECAST CONCRETE STRUCTURES

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ABSTRACT

A new precast concrete system was developed in Japan, and its more remarkable points are that the main bars are jointed at the central portion of the members where the seismic forces are supposed to be small. The precast members are simple elements such as beams and columns, and are jointed at the member ends, that is at the beam-column joints. The proposed system is that when the precast members are cast, instead of main bars, metallic corrugated sheath and lapping bars are embedded at the position of main bars with lateral reinforcements. Once at the construction site, the main bars are inserted and high strength mortar is grouted into the sheaths to fix the bars in precast members.

The experimental studies were focused on connections of main bars using lap splices for a precast concrete frame type structures. These studies include pull out tests of lapping joints and bond strength of sheaths. The test results showed that the system main bar-mortar-sheath behaves as a unit. This system with lapping bars is good enough to transfer the stresses to the surrounding concrete without failure. Also the bond behavior of sheaths is almost similar to that of deformed bars with the same diameter of sheath.

KEYWORDS
precast concrete, main bars, lapping bars, spiral sheath, high strength mortar, lapping joint strength, bond strength,

INTRODUCTION

Numerous researches on how to connect precast concrete members have been conducted in Japan. Most of them have the reinforcements embedded in the precast members, therefore the concrete and main bar joints are located at the beam column joints. The bars are jointed using sleeves joints filled with mortar for columns and the beam bars are anchored at the beam column joints. Even though the precast members of these systems are of easy production and handling, problems like the cost of sleeve joints for columns which are expensive and the complicated reinforcement details at the beam-column joints make those systems a somewhat disadvantageous (Imai, 1993).

A new precast concrete system was developed in Japan (Imai et al., 1991), which tries to solve most of the problems presented by other precast concrete systems. This system provides an economical and practical way of connecting precast concrete elements ensuring adequate stiffness, strength and ductility. One of its outstanding points is that the main bars are jointed at the central portion of the members where the seismic forces are supposed to be small as shown Fig. 1. The precast members are simple elements such as beams and columns, and are jointed at the beam-column joints with concrete cast in site. The columns are full precast concrete and the beams are solid half precast concrete.

The proposed system is that when the precast members are cast, instead of main bars, metallic corrugated sheaths and lapping bars are embedded at the position of main bars with the lateral reinforcements. Each sheath is lapped with one or two bars, depending on whether the element is a column (two lapping bars) or a beam (one lapping bar). This is because generally the columns are subjected to bi-directional stresses, while the beams to unidirectional ones. Also, the length of beams is usually long enough to allow longer splicing of bars than those in columns without reaching the plastic hinge region during strong earthquakes. In the case of two lapping bars, for columns, the total sectional area of them is larger than that of the main bar. On the other hand, in case of one lapping bar for beams, a bar with the same size as the main bar is used. For the beam bottom bars, long lapping bars are used and considering the stress due to vertical loads, the lapped parts are shifted from the center of beam, as shown in Fig. 1.

At the construction site, once the precast columns and beams are positioned, the lower bars of the beams are inserted into the sheaths so that they pass straightly through the joints. At this stage the shear reinforcements of the beam-column joints are placed, after which the column and the beam upper reinforcements are placed. High strength mortar of 600 kgf/cm^2 (58.8 MPa) is grouted in to the sheath of the columns. To realize this, the mortar is pumped in at one inlet port (metallic tube) and there is a common space that links all the sheaths at the bottom part of the column as shown in Fig 2. Therefore the mortar flows up from the bottom almost at the same time, displacing the air progressively between the sheaths and main bars. The process is stopped when the mortar comes out from the top of the column. Next step is to cast concrete in the upper part of the beam, slab and beam-column joint as well. Then the beam lower main bars are also grouted from the inlet port located at the upper part of the beams near the beam ends. The grout crosses from one sheath to the other through connectors located at the beam ends. To confirm a good circulation of the grout mortar inside the sheaths, in the opposite side, an outlet port is provided as shown in Fig. 3 . Once the injection was completed the both ports were plugged. The transfer mechanism of this lapping joint method is that the stress at one end of main bar is transferred progressively from the mortar to the lapping bars by bond stresses on the sheath and vice versa.

This system has several advantages like easy production and handling of precast members, main bar joints located at less stressed positions, vary simple reinforcing details at beam-column joints. Also was proved that the seismic behavior of frame structures using this system does not differ from those structures made of conventional reinforced concrete. Therefore no special considerations are necessary for the structural calculations.

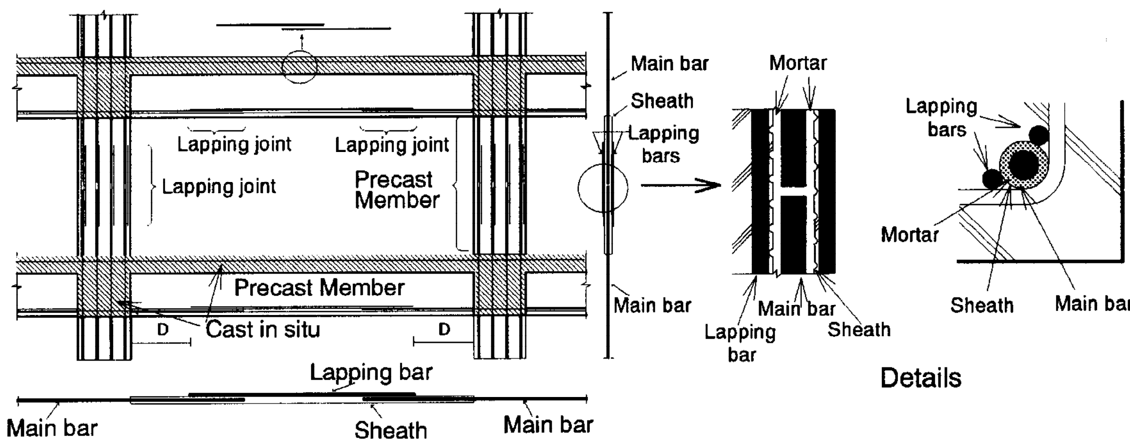


Fig. 1 Outline of the main bar post insertion system

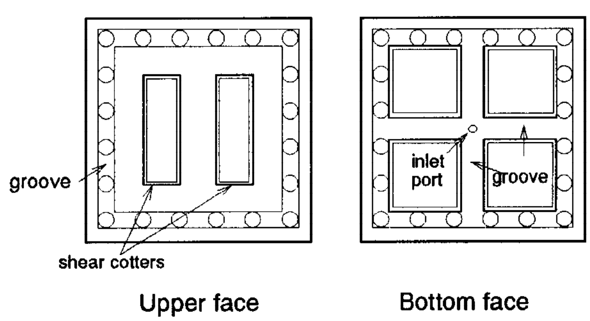


Fig. 2 Details of the column ends

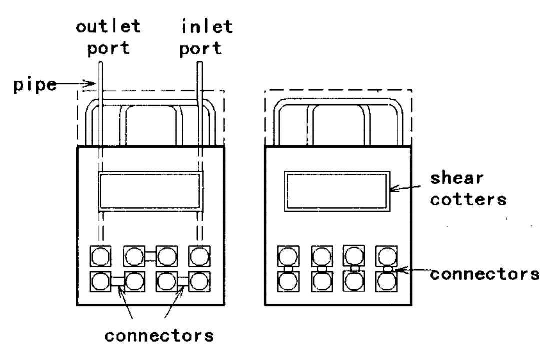


Fig. 3 Details of the beam ends

EXPERIMENTAL PROGRAM

One of the most important aspects in the design of precast concrete structures is the connections, either the concrete and the bar connections. Since the bars are straightly located at the beam-column joints they showed a very ductile behavior (Castro J. J. et al., 1993). However the bar connections make this system very particular. Therefore they must be studied carefully, specially those parameters that influence the strength of the bar joints (Yanez et al., 1991).
The experimental program was divided in two parts. The first part was carried out to study the joint strength of lapping splices, specially the influence of the lapping splice type, position of main bar inside the sheath, as well as the length of

the lapping splices. In the second part the main objectives were to study the bond strength of sheath and compare its behavior with those of deformed bars which have similar diameters to the sheath diameter. Also the influences of the lug of sheath, sheath diameter and lateral reinforcement were studied. These two parts will be discussed separately.

JOINT STRENGTH OF LAPPING SPLICES

Specimens.

Two series of tests were carried out. In the first series, a pull out test was carried out to study the influence of the lapping splice type, position of bar inside the sheath and the number of lapping bars on the joint strength. In this Series the length of the lapping bars was 20 times the diameter of the lapping bar (20 d). The section of the specimens is shown in Fig. 4 (a) where the distance between the bar axes is 100 mm and the differences among the splices are shown in Fig. 4 (b). Splice types A and B correspond to those used for precast columns, while C and D for precast beams.

Main bars of D25 (diameter of 25 mm), with specified yield strength of 4000 kgf/cm^2 (392 MPa, SD390) for main bars, and two bars of D19 (SD390) or one bar D25 (SD390) for lapping bars were chosen for this investigation. The specified concrete strength for the precast concrete specimens was 300 kgf/cm^2 (29.4 MPa) and the specified compressive strength of the grout mortar was 600 kgf/cm^2 (58.8 MPa).

Table 1. Specimens of Series 1

Spec.	Spec. Length (mm)	Section (mm)	Main bar	Lapping bar	Lapping length (mm)	Lateral reinf.
AA-1						4-D10@100
BB-2						
EA-3	1000			2-D19	760	4-D10@150
EB-4						
EA-5		400 x	4-D25			4-D10@200
CC-6		700				4-D10@100
FC-7				1-D25	1000	4-D10@150
FD-8						
FC-9						4-D10@200
EC-10	1200			2-D19	760	2-D10@150
				1-D25	1000	
DG-11		350 x	3-D25			
FC-12		700		1-D25	1000	3-D10@150
GD-13						

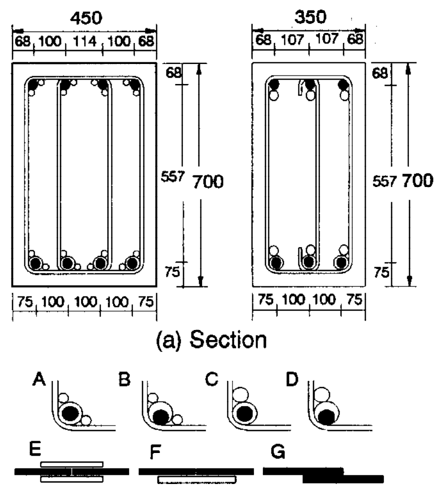


Fig. 4 Specimen Series 1

In the second series the length of the lapping bars was varied from 10d to 35d, to determine the suitable length over which the failure is not due to lost of between sheath and concrete. The specimens have a section of 450x600 mm with a separation between main bars of 100 mm as is shown in Fig. 5. Only the bottom bars were tested. Main bars were also D25, with specified yield strength of 5000 kgf/cm^2 (492 MPa, SD490) with a lapping bar of the same characteristics. The specified strengths for concrete and grout mortar were same as those used in Series 1

Table 2. Specimens of Series 2

Spec.	Section (mm)	Spec. length (mm)	Lapping length n x d	Lateral reinf.
S10D		700	500	
S15D		950	750	
S20D	400 x	1200	1000	With
S25D	600	1450	1250	Sheath
S30D		1700	1500	
S35D		1950	1750	
N10D		700	500	
N15D		950	750	
N20D	400 x	1200	1000	Without
N25D	600	1450	1250	Sheath
N30D		1700	1500	
N35D		1950	1750	

Main bars 4-D25 , Lapping bars 1-D25 (SD390)
Lateral reinf. 4-D10(SD295A)@150

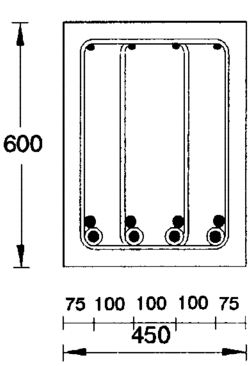


Fig. 5 Specimens series 2

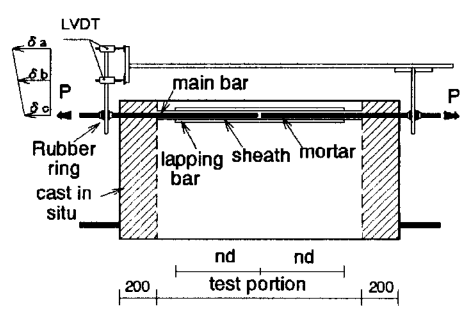
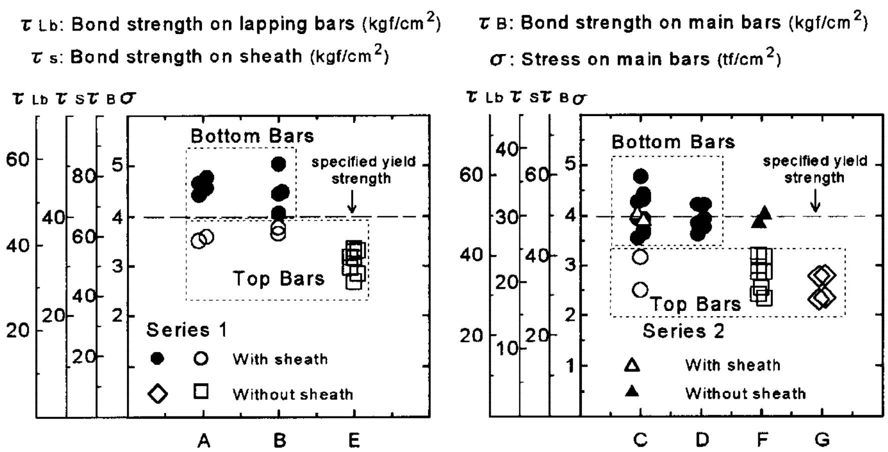


Fig. 6 Loading system

The loading system was identical for both series as is shown in Fig. 6. Tension force P was applied horizontally at both ends of each main bar by oil jacks. To obtain the maximum load, tension force was applied monotonously with an increment of 1 tonf until failure. Displacements between the ends of each main bar were measured by mean of two sets of devices placed at the corner and side bars. Each set consists of four linear voltage displacement transducers (LVDT). The displacement of each bar was obtained by extrapolating the measurements of the transducers.

Test results

Influence of the number of spliced bars (Series 1). Figures 7 (a) and (b) show the tensile joint strength of main bars for the different types of joint splices . In this figures, the average bond stresses on the main bar, on the sheath and on the lapping bar at maximum loads are also indicated. Even though the amount of lateral reinforcement (p_w) is slightly different, no large difference is observed between splice types A and C, also between splice types B and D. Therefore, regardless of the number of lapping bars no remarkable influence on the bar joint strength is recognized.



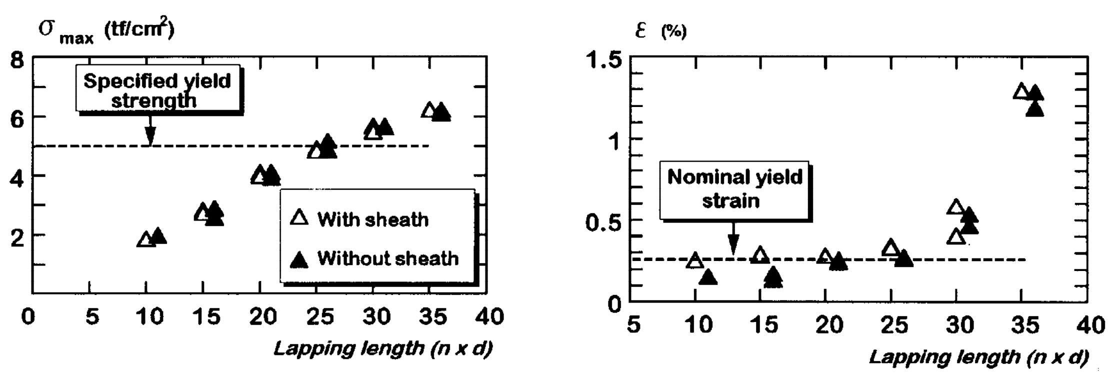
(a) Splices with two lapping bars *(b) Splices with one lapping bars*

Fig. 7 Influence of the position of main bar inside of the sheath

Position of the bar inside the sheath. From Figs. 7 (a) and (b), no remarkable difference can be recognized between splice types A and B, also between splice types C and D. This means that the eccentricity of the bar from the sheath axis has no influence on the joint strength.

Influence of the sheath. Some slightly higher values of the strength of the top bar joints with sheaths (splice types A, B, and C) than those without sheaths (splice types E, F and G) are found in Figs. 7 (a) and (b).

Influence of the lapping length (Series 2). The maximum stress and relative slip deformations for different lapping length are shown in Fig. 8 (a) and (b). The relative slip deformation was obtained dividing the measured slip by the length of specimens. For the specimens with lapping length less than 25 d, the main bars were directly pulled out, with a little deformation of main bars. This is because of the insufficient anchorage length which made the concrete key between the lugs of sheath fail in shear. Here, no case of bond failure was observed between the main bars and the surrounding high strength mortar.



(a) Main bar stress (b) Main bar slip deformation

Fig. 8 Influence of the lapping length

For a length larger than 25 d the failure was due to yielding of main bars in which the bond transfer has adequately developed the tensile capacity. Also test results showed a more flatter slope in their correlation that the observed when the length is shorter than 25 d. Once the yield capacity of the main bars is reached, within the yielded length the reinforcement stress is approximately constant thus no force is being transferred in this region. Therefore, the consequence of yielding in the anchored zone is a reduction in the length effective in the force transfer mechanism which leads to failure.

In the actual calculation of a frame type structures, 40 d is used as a lapping length, which ensure an enough length to allow the tensile force to be transferred from one bar to the other, through the lapping bars by bond stress.

Additionally from this figures no remarkable differences can be observed between the bars with or without sheaths. Therefore it is possible to affirm that the steel bars behave monolithically with the sheath and mortar.

BOND SPLITTING STRENGTH

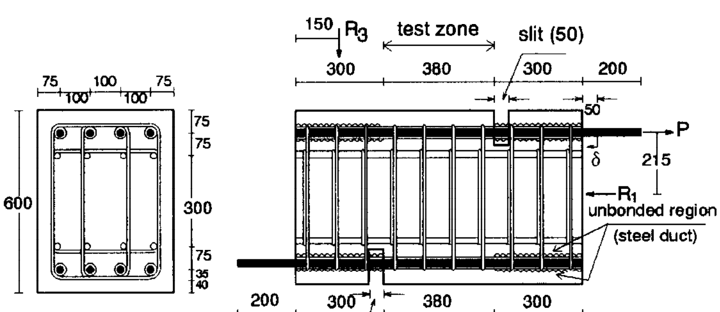
In the system conformed by main bar-mortar-sheath, the high strength mortar confined by the spiral sheath is assumed to be so strong that bond failure will not occur between the lugs of main bars and the interior lugs of the sheath. But, as the proposed system uses steel sheaths with diameters more than 40 mm, the surface of concrete to resist the splitting forces become smaller, therefore it may be possible that the bond strength per unit of surface area decrease. Consequently it is necessary to study the bond characteristics of the spiral sheath.

Specimens

The specimens were designed to represent a confined section of precast concrete columns or beams. Table 3 shows the differences between the specimens . Figure 9 shows the section of the typical specimens which had a section of 450 mm x 600 mm with a distance between bar axes of 100 mm. The bonded part (test zone), was positioned in the middle of the specimen, with a length of 380 mm which is 20 times the diameter of the bars D19. The bond free length at either side was 300 mm. A slit of 50 mm was provided at the end of the bonded region to prevent the concrete of the unbonded region from restraining the concrete at the test zone. The bond free end was obtained by placing a steel ducts around the bars and sealing the ends with plastic tape to prevent concrete from flowing in.

Table 3 Characteristic of specimens

No	Param.	Lug height (mm)	Sheath diam. (mm)	Lateral reinf. Ratio (p_w %) SD295A	Defor. Bars SD490
1	Lug	1.5			
2	height	2.0	42		
3	(mm)	3.0		4-D10@100 (0.63)	
4	Sheath		38		
2	diam.		42		
5	(mm)		46		
6				2-D10 @100 (0.32)	D25
7	Lateral			2-D13 @100 (0.56)	
2	reinf.		42	4-D10 @100 (0.63)	
8	Ratio			4-D13 @150 (0.75)	
9				4-D13 @100 (1.13)	
19	Defor.				D25
20	Bars	without sheath			D29
21	SD490			4-D10 @100 (0.63)	D35
22					D41



Section

Detail of reinforcement

Fig. 9 Specimen

The concrete of the specimens was cast horizontally. The position of bars was the basis of naming the upper cast bars as "top bars" and the lower cast ones as "bottom bars". The specified concrete strength for the precast concrete specimens was Fc=300 kgf/cm^2 , and for the grout mortar it was 600 kgf/cm^2. The main bars were D25 threaded type bars, with specified yield strength of 5000 kgf/cm^2 (492 MPa, SD490), which were used to avoid the yielding of bars during the experiment.

Tension force P was applied horizontally at the end of each main bar by four oil jacks controlled by a load cell. The displacement of the corner and side bars from the end face of the concrete at the loading side were measured using four linear voltage displacement transducers (LVDT, with an accuracy of 1/1000 mm), attached to a rigid steel frame at the mid-height of the specimens.

Test results

Comparison with the existing equations. To compare the test results of bond strength with those calculated using the existing equations, the applied maximum forces were converted into average bond stresses (τ_{exp}) using Eq. (1). Also, bond splitting strengths calculated by Fujii-Morita Eq. (2) (Fujii et al., 1983), τ_{cm}, Kaku-Zhang-Iizuka-Yamada Eqs. (4) and (5) (Kaku et al. 1992), τ_{ck}, for continuous bars, Orangun-Jirsa-Breen Eq. (3) (Orangun et al., 1977), τ_{coj} for bars with lap splices, are plotted for every equation.

$$\tau_{exp} = \frac{P_{max}}{l_s \phi} \qquad \text{(1)}$$

$$\tau_{cm} = (0.307 b_i + 0.427 + 24.9 \frac{k A_{st}}{s N_t d_b}) \sqrt{\sigma_B} \quad \text{(multiplied by 1.22 for bottom bars)} \qquad \text{(2)}$$

$$\tau_{coj} = (1.2 + 3 \frac{d_b}{l_s} + \frac{A_w \, {}_w\sigma_y}{35.2 N_t s d_b}) 0.265 \sqrt{\sigma_B} \quad \text{(divided by 1.3 for top bars)} \qquad \text{(3)}$$

$$\tau_{ck} = [0.08 + 0.12 b_i + k_n (\frac{2.5 + 875 p_w}{l_b / d_b + 7000 p_w} + 18.0 \frac{p_w b}{N d_b})] \sigma_B^{0.6} \qquad \text{(4)}$$

if $\sigma_{wy} \le 115 \sigma_B^{0.6}$

$$\tau_{ck} = [0.08 + 0.12 b_i + k_n (\frac{2.5 + 875 p_w}{l_b / d_b + 7000 p_w} + 18.0 \frac{p_w b}{N d_b})] \frac{\sigma_{wy}}{115} \qquad \text{(5)}$$

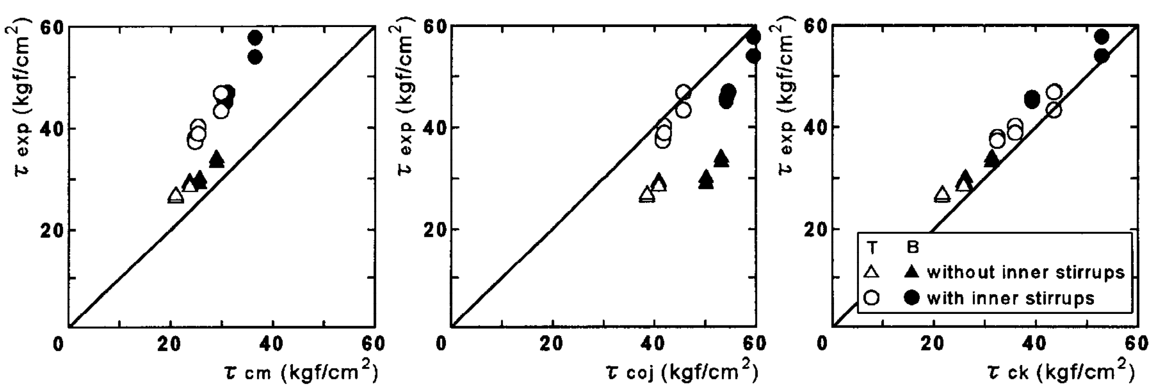
$$k_n = 1 + 0.85 \frac{(n-2)}{N}$$

Where:

P_{max}: maximum force in the main bar (kgf);	ϕ: perimeter of main bar or sheath (cm);
$l_b = l_s$: bonded length or lapped length (cm);	b_i: parameter for clear spacing between main bars;
k: parameter of the bond failure mode;	$A_{st}=A_w$: area of lateral reinforcement (cm2);
N_t: total number of main bars;	n: number of tie legs;
d_b: diameter of main bar or sheath (cm);	${}_w\sigma_y$: yielding strength of lateral reinforcement (kgf/cm^2);
c: half clear spacing between bars or half available concrete width per bar or splice resisting splitting in the failure plane (cm);	s: spacing of the lateral reinforcing bars (cm);
b: width of beam (cm);	σ_B: concrete cylinder strength (kgf/cm^2);
	$p_w = \rho_v$: lateral reinforcement ratio.

Figure 10 (a) shows the comparison between the experimental and the calculated values using Fujii's Eq. Specimens without inner stirrups have a good agreement, while the experimental values for specimens with inner stirrups are much bigger than the calculated.

In Figure 10 (b), the calculated values using Orangun's Eq. showed greater values than the experimental for the specimens both with and without inner stirrups. Some of the specimens with inner stirrups showed good correspondence.



(a) Fujii's Eq. (b) Orangun's Eq. (c) Kaku's Eq.

Fig. 10 Experimental bond strengths and calculated values

Figure 10 (c) shows the comparison between the experimental values and the calculated using Kaku's Eq. The effect of the inner stirrup is considered as well as the correlation among the top and bottom bars related to the strength of concrete. The test results from specimens both with and without inner stirrups present a good correlation with the calculated values.

Lug of the sheath. The relationship between the lug height of the sheath and the bond strength is shown in Fig. 11. When the lug height is varied from 1.5 mm to 2.0 mm the bond strength increases. But when it is varied from 2.0 mm to 3.0 mm a slight increase on the bond strength is observed. None of the calculated values using Fujii's Eq., Orangun's Eq. and Kaku's Eq. fits with the tests results.

Sheath diameter. The effect of the sheath diameter on the bond strength is shown in Fig. 12. As the diameter increases the bond strength decreases for either the top or bottom bars. It seems that the confinement effect due to the concrete between sheaths decreases when the sheath diameter becomes larger. The effect of the bar diameter is considered in each of the existing bond strength equations. Calculated values are obtained by using the sheath diameter instead of the bar diameter. Calculated values using Kaku's Eq. and Orangun's Eq. fit well with some of the test results. The calculated values using Fujii's Eq. give lower values than the experimental results.

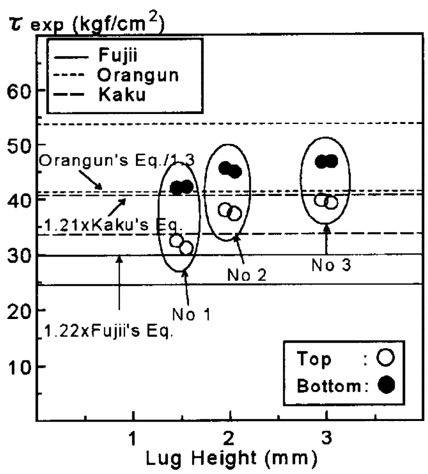


Fig. 11 Effect of the lug height of sheath

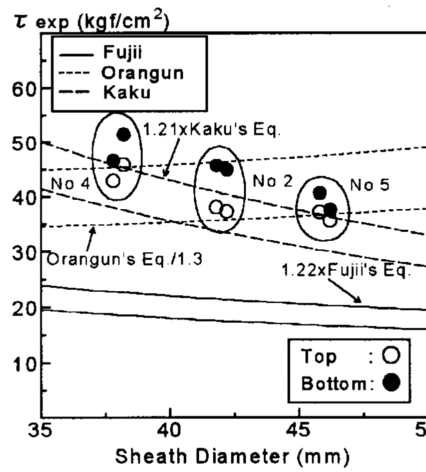


Fig. 12 Effect of the sheath diameter

Lateral reinforcement ratio. The effect of the variation of lateral reinforcement ratio is shown in Fig. 13. As the lateral reinforcement ratio increases the bond strength also increases. For specimens with inner stirrups (4-D10 or 4-D13), the increase of the strength became more evident than those for the specimens without inner stirrups (2-D10 or 2-D13), in which the bond strength increases slightly. It is found here that the amount of lateral reinforcement has a great influence on the bond strength.

The values obtained using Kaku's Eq. shows a quite good agreement with the test results for either the top or bottom bars. Orangun's Eq. gives a good approximation to the test results for the top bars, while for the bottom bars this equation gives higher values. Fujii's Eq. gives lower calculated values than the experimental.

Comparison of the bond strength of sheath with those of large size deformed bars. The comparison of the bond strengths of sheaths with those of large size deformed bars is shown in Figs. 14(a) and (b). The bar diameters are 25, 29, 35 and 41 mm, and for sheaths are 38, 42, 46 mm. Sheaths were selected considering the geometrical properties of a bar with diameter of 41 mm, lug height: 2 mm, distance between lugs: 28 mm.

As the diameter increases, the bond strength per unit surface area decreases, as shown in Fig. 14(a). In case of the deformed bars placed at the bottom of the specimens, the strength decays considerably, while for the top bars the strength decays slightly. In case of sheaths the strength decreases slightly for both the top and bottom bars. In Fig. 14(b), the bond force for the bonded length is shown. For the experimental values, Pmax is the bond force. As much as the bar diameter increases the bond force also increases. The bond force of sheaths remains constant. Also, the bond forces of the top bars with diameters of 29, 35, and 41 mm are almost similar to that of sheaths with diameters of 38, 42, and 46 mm. Therefore, it is possible to estimate roughly that the bond behavior of sheath is similar to that of the deformed bars with the same diameters to the sheath.

The calculated values using Kaku's Eq. fit with the experimental values of the deformed bars and sheaths placed at the bottom. Orangun's Eq. gives a good agreement with the test results of sheaths with diameters of 42 and 46 mm placed at the top.

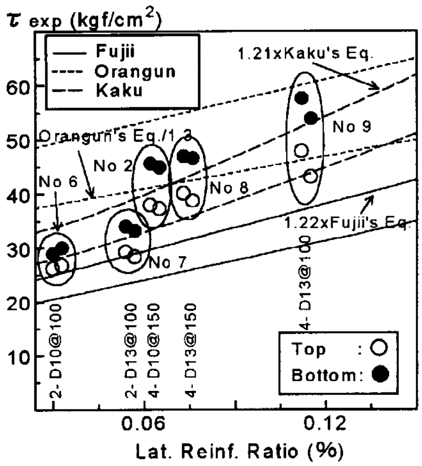
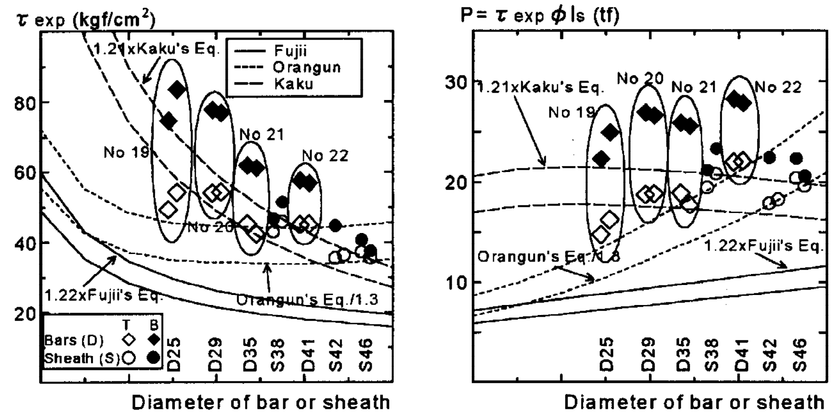


Fig. 13 Effect of the lateral reinforcement



(a) Bond strength

(b) Bond force

Fig. 14 comparison between bond strength of sheath and those of large size deformed bars

CONCLUSIONS

The test results of an investigation carried out on the bar connections for a precast concrete system has been reported, focusing on the joint and bond strength of spliced bars using spiral sheath. The test results are summarized as follows:

1) The system composed by main bar-mortar-sheath behave as a unit, and proves to be adequate in transferring the tensile stress of each main bar through the surrounding concrete.
2) At least 25 d (diameter of lapping bar) is necessary to ensure that the yield stress will be transferred from the main bars to the lapping bar. In the real design of a frame structure, a length of 40 d is used for this type of splices which are also located outside of the region where flexural yielding is anticipated. This makes the system even more safety under seismic actions.
3) The eccentricity of the main bar from sheath axis has no effect on the joint strength of spliced bars.
4) Since 2 mm of lug sheath seems to be the turning point after which the bond strength does not increase, a lug height of 2 mm is recommended for spiral sheath.
5) The test results show the importance of the amount and distribution of the lateral reinforcement on the bond strength, specially when inner hoops are used.
6) The bond behavior of sheaths is almost similar to that of deformed bars with the same diameter to the sheath. Therefore it is possible to make a rough estimation of the bond strength using diameters of bars similar to the diameter of sheaths.
7) Kaku s equations fits well to the test results, therefore it can be used for the estimation of bond strength of spiral sheaths

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