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論 文

[2212] An Experimental Study on the Tensile Capacity of Vertical Bar Joints in a Precast Shearwall

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1. INTRODUCTION

Connections and detailings are the usual problems in precast shearwalls. In the conventional method, the joints are usually located at wall ends. This implies that both concrete and reinforcing bars are jointed at critical locations where, during the action of lateral loads, large stresses occur. This research aims to establish a new, simple and economical technique of connecting vertical bars in precast concrete shearwalls at locations where seismic forces are less. In this technique, the precast concrete panels and the vertical reinforcements, which are separated in the fabrication plant, are connected at the site. Concrete joints are at member ends while vertical bar joints are at mid-height of the wall. As a pioneering research, the tensile capacity of the newly developed method of connection is investigated.

2. OUTLINE OF EXPERIMENT

2.1 DESIGN OF SPECIMENS

A typical middle portion of a precast shearwall shown in Fig. 1(a) is the basis of design for test specimens. Forty five specimens were tested. These were divided into 15 groups at three specimens per group. The specimens are varied according to the following: size and length of lapped bars, spacing of vertical (main) bars, lug height of steel sheath, pitch of spiral steel, and the applied cyclic load. For all the specimens, the wall thickness was maintained at 200mm, the tubular steel sheath diameter at 42mm, and the distance of the side edges of wall from the last main bar at 1/2 the main bar spacing.

Table 1 shows the description of all the groups. The first three specimens, designated as group B1, were set as the standard specimens with 4–D13mm splice bars, 200mm spacing of main bars, lapping length of splice bars at 20 times the lapped bar diameter (20d), lug

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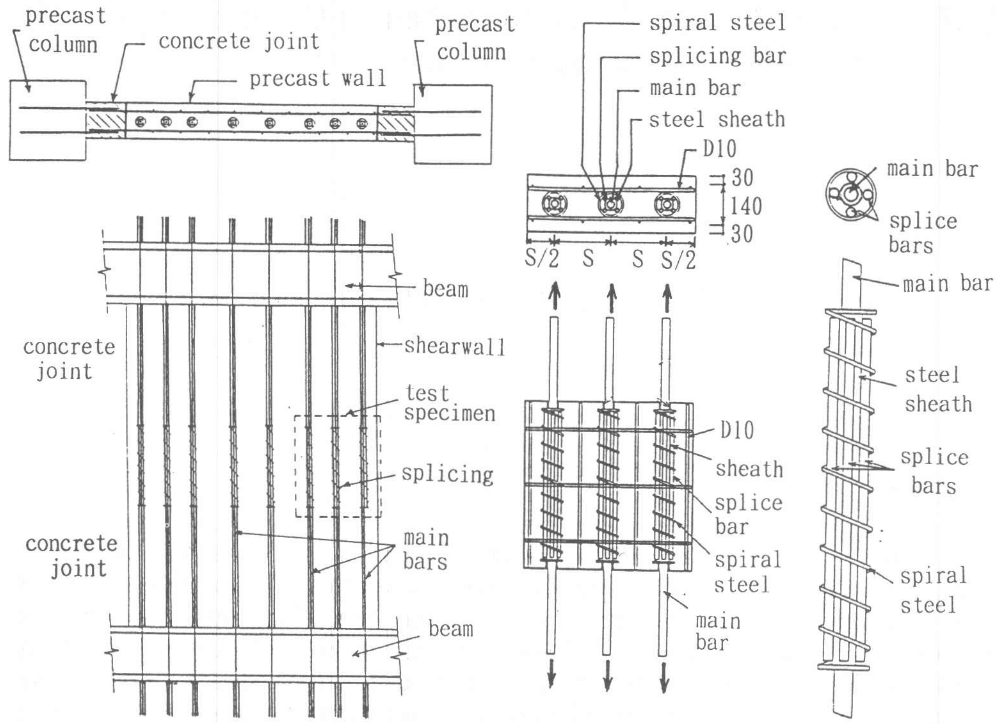


(a) Typical Shearwall (b) Specimen (c) Bar Joint

Figure 1. Details of Specimen.

Table 1. Properties of Test Specimens.

		Group	Splice Bar No & Size	Main Bar Spacing	Lapped Length	Lug Height	Spiral Pitch
Standard		B1	4-D13	200mm	20d	2.0mm	60mm
Number & Size	2-D19 3-D16	B2 B3	2-D19 3-D16	200mm	20d	2.0mm	60mm
Spacing	300mm 400mm	B4 B5	4-D13	300mm 400mm			
Lapped Length	25d 30d	B6 B7		200mm	25d 30d		
Lug Height	1.5mm 3.0mm	B8 B9			20d	1.5mm 3.0mm	
Spiral Pitch	30mm 90mm 120mm	B10 B11 B12				2.0mm	30mm 90mm 120mm
Cyclic Load	1.3Fy*2/3 1.5Fy*2/3 1.6Fy*2/3	B13 B14 B15	10 repetitions of cyclic loading (standard specification)				

d = diameter of splice bar,
Fy = σ_{Yo}*A (σ_{Yo} = steel specified yield stress, A = area)

height of corrugated tubular sheath at 2mm, and pitch of spiral steel at 60mm. Groups B2 and B3 differ from the standard by the size and number of splice bars. 2–D19mm and 3–D16mm were used, respectively. With the increase in splice bar sizes, the lapped lengths also increased making the specimens longer than the standard. In groups B4 and B5, the alteration made from B1 was the spacing of main bars at 300mm and 400mm, respectively. The lapping length was changed from 20d to 25d and 30d in groups B6 and B7, respectively. The 2.0–mm lug height of sheath became 1.5mm and 3.0mm in groups B8 and B9, respectively. The 60–mm pitch of confining spiral steel in the standard specimen was changed to 30mm in group B10, 90mm in group B11, and 120mm in group B12.

Groups B13, B14, and B15 were identical to the standard but these were subjected to cyclic loading up to a maximum load of (2/3) x 1.3 Fy (Fy = specified yield stress, σ_{YO}, multiplied by cross sectional area of main bar), (2/3) x 1.5Fy, and (2/3) x 1.6Fy, respectively, at 10 repetitions each. The splice length used in the specimens was 20d but the AIJ requires 30d. The factor (2/3) was obtained from 20d/30d.

2.2 MATERIALS

The material properties can be seen in Table 2. Ordinary type of concrete with compressive strength of 300 kg/cm^2 and SD390 main bars were used. The steel utilized for lap splice bars were SD345 for D16mm and D19mm and SD785 for D13mm. SD295A double cross D10mm was the lateral mesh reinforcement. The tubular sheath has a thickness of 0.25mm and an inner diameter of 42mm. The rib height was varied at 1.5mm, 2.0mm, and 3.0mm. A high strength mortar with specified compressive strength of 600 kg/cm^2 was grouted. Plain type spiral steel with 6–mm diameter was used. The inner spiral diameter was 80mm for 3–D13mm lapped bars and 100mm for other splice bars.

Table 2. Material Properties

(a) Concrete unit: kgf/cm^2

Specimen	B1	B2	B3	B4	B5	B6	B7	B8	B9
Compression	294	286	286	286	286	315	315	326	291
Splitting	27	30	30	30	30	28	28	38	27

Specimen	B10	B11	B12	B13	B14	B15
Compression	306	314	308	272	295	284
Splitting	33	29	33	29	27	29

Sp. Comp. Strength:
Concrete:300kgf/cm^2
Grout :600kgf/cm^2

(b) Grout unit: kgf/cm^2

	3 days	28days	56days	71days	90days
ave. compressive strength	411	649	675	688	692

(c) Steel unit: $tonf/cm^2$

Size	Grade	Spec. Yield Strength	Actual Yield Strength	Actual Tensile Stress	Remarks
D25	SD390	4.0	4.33	6.04	main bar
D19	SD345	3.5	3.70	---	splice
D16	SD345	3.5	3.52	---	splice
D13	SD785	8.0	8.66	---	splice
ϕ6	---	---	4.50	7.08	spiral
D10	SD295A	3.0	3.65	3.65	mesh

2.3 PROCESS OF ASSEMBLY

For each sheath positioned vertically, two main bars were inserted at both ends of the sheath letting the bar ends meet at its mid-height. With the help of rubber cap sealing the lower end of the sheath, the high strength mortar was poured from the top end, thereby, filling the space between the sheath and the main bar. Lapping bars were placed around the sheath which were then confined by small diameter spiral steel as can be seen in Fig. 1(c). With the main bars and the assembly positioned horizontally, the concrete was poured.

3. LOADING SYSTEM AND APPARATUS

In Fig. 1(b), the way of loading is shown. At each end of main bars, equal tensile forces were applied by oil jacks at the same time. Cantilever type displacement transducers were attached at both ends of each rebar. The displacement between the ends of each bar was measured on two of the main bars. As illustrated in Fig. 2, the actual deformation δc along the axis of the main bar was obtained by extrapolation. Groups B13, B14, and B15 were subjected to 10 cycles of loading from zero to specified maximum load. After 10 cycles, these were loaded until the maximum tensile capacity.

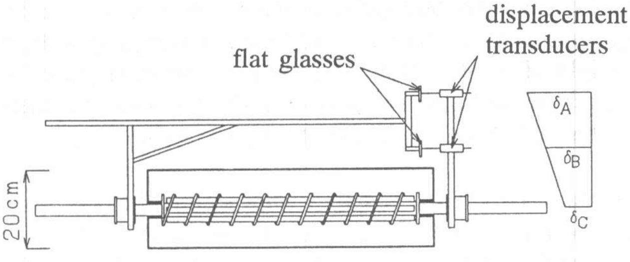
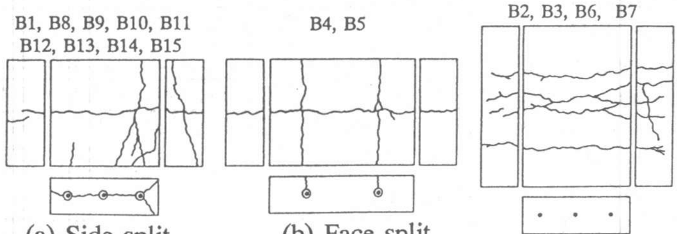


Figure 2. Set-up of Displacement Transducers

4. TEST RESULTS

Each group had three specimens which incurred similar failure modes. Only the results per group will be discussed. Typical crack patterns of specimens are shown in Fig. 3. Two types of failure occured: bond failure around the sheath and direct pull-out of main bar.



(a) Side split (b) Face split
bond failure around the sheath (c) Direct pull-out
Figure 3. Typical Crack Patterns

The standard specimens under B1 failed in bond failure around the sheath at an average tensile capacity of 23.5 tonf, approximately 1.2 Fy. When the number of lapped bars was changed from 4-D13mm to 3-D16mm in B2 and 2-D19mm in B3, the failure was a direct pull-out of main bar and the tensile force of the joint reached nearly 1.5Fy which is equivalent to the tensile strength. This can be seen in Fig. 4(a). In B4 and B5 shown in Fig. 4(c), varying 200-mm spacing of main bars to 300mm and 400mm increased the average tensile strength to approximately 1.4Fy. In these specimens, face cracks occured and the failure was on bond around the sheath. When the lapped length 20d became 25d or 30d in B6 and B7, the specimens failed by direct pull-out of main bar at a load of approximately 1.5Fy as plotted in Fig. 4(b).

Compared to 2.0-mm sheath lug height in B1, 1.5-mm lug height in B8 had a lower capacity at only 20 tonf but 3.0-mm lug height in B9 had a resistance almost equal to that of 2.0mm as shown in Fig. 4(d). B8 and B9 showed the same failure mode which is bond failure around the sheath. In B10, B11 and B12 as can be seen in Fig. 4(e), the bar joint capacity decreases as spiral pitch increases. All specimens in these groups failed by slippage on the sheath. The results were: 28.4 tonf at 30-mm spiral pitch, 22.8 tonf when the pitch was 90mm and 21.1 tonf when it was 120mm. The standard specimen with 60-mm spiral pitch provided a tensile strength of 23.5 tonf. The specimens in B13, B14, and B15, which were subjected to 10 repetitions of cyclic loading, still reached a maximum connection capacity almost equal to that attained by the monotonically loaded standard specimens. This is presented in Fig. 4(f). A typical load-displacement diagram is shown in Fig. 5. The envelope curve of repeated loading coincides with that of monotonic loading.

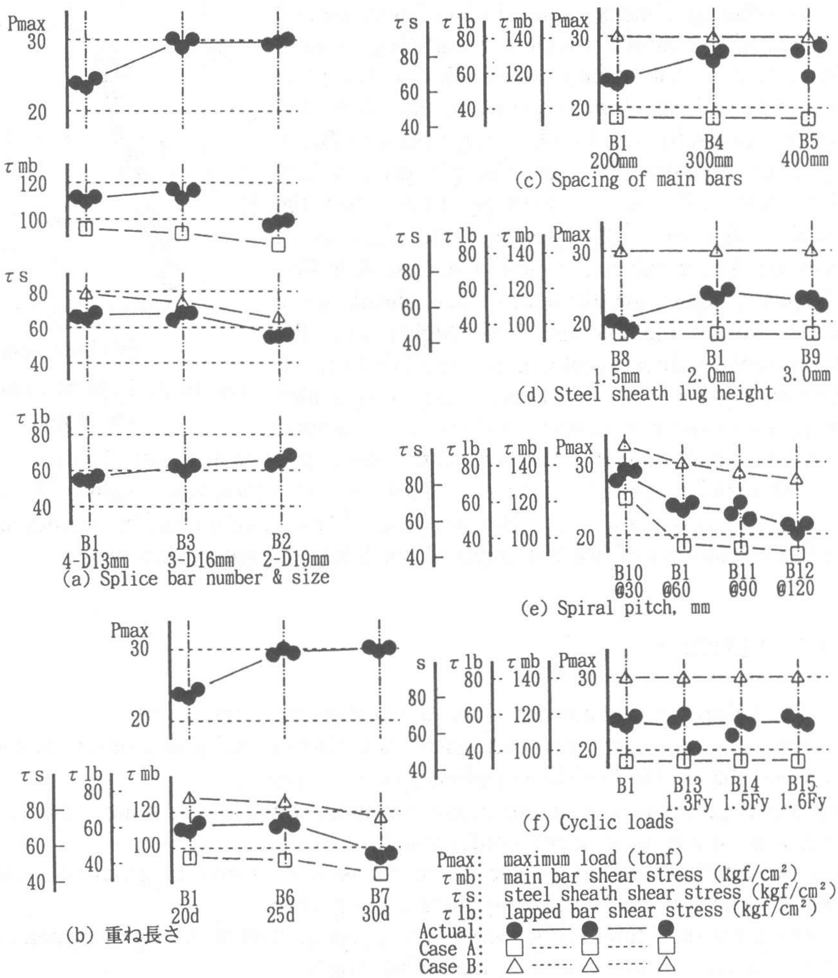


Figure 4. Failure Modes of Specimens

5. ANALYSIS

The groups are classified according to the two failure modes mentioned above. In groups B2, B3, B6, and B7, which have greater lapping lengths, the mode of failure was a direct pullout of main bar inside the sheath without splitting the cover concrete. This is accounted for by an adequate gripping pressure exerted by the confining spiral steel, concrete, and lapped bars against the tubular steel sheath. This gripping force exceeded the bond capacity of grout against main bar inside the sheath.

All other specimens incurred bond failure around the sheath. This implies that lesser confining pressure was acting on the sheath which allowed the main bar, grout and sheath act together as one. A related analysis has been done by Orangun, Jirsa, and Breen [1]. Their equation for bond strength gives values which are 16%, on the average, lower than the actual results when the d_b used is the diameter of main bar. These values are plotted as Case A in Fig. 4. When d_b equals the diameter of steel sheath, these values are on the average 19% higher than the experimental values as shown in Case B of Fig. 4. Morita's equation [2] for bond strength was also compared to the experimental values. The diameter of bar, d_b, used in the calculation was the diameter of steel sheath. The confining spiral steel were assumed to be the transverse reinforcements. Much lower values by as much as 50% were obtained when this equation was used. This equation can be considered inappropriate for the calculation of bond strength of bar joints in precast shearwall.

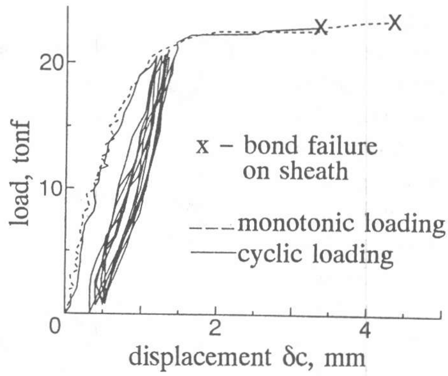


Figure 5. Typical Load–Displacement Diagrams

6. CONCLUSION

The following conclusions were drawn from the experiment:

1) In vertical bar joints with corrugated steel sheath, the failure modes are: bond failure around the sheath and direct pull–out of main bar.
2) A direct pull–out of main bar occurs when the splice length is more than 25d, that is when there is an adequate lateral confinement.
3) A bond failure around the sheath can be assumed when the grout resisitance inside the sheath exceeds the lateral confinement of splicing.
4) With a constant total cross sectional area, splice bars with larger diameter provide greater strength due to an increase in the splice length.
5) At a splice length more than 25d, the bar joint capacity remains almost equal to the tensile strength of main bar.
6) A change from 200–mm spacing of main bars to 300 mm gives large increase in the joint capacity but a change from 300 mm to 400 mm provides slight increase in the capacity.
7) A lug height of more 2.0 mm has a strength almost equal to that of 2.0 mm.
8. A decrease in spiral pitch improves the structural performance of the bar connection.
9) 10 repetitions of cyclic loads have no effect on the structural performance of the joint.
10) Orangun, Jirsa, and Breen's equation can be used to roughly estimate the bond strengths of main bar and steel sheath.

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