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[2229] Tensile Capacity of Main Bar Splice at a Reduced Precast Shear Wall Thickness

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1. INTRODUCTION

This study focuses on the development of an earthquake resistant connection for precast concrete shear wall using an innovative, simple and economical technique of connecting vertical bars. As shown in Fig. 1, the design of this connection incorporates the idea of confining the four lap splices around the tubular steel sheath by a spiral steel. The ends of two main bars are inserted inside the steel sheath and grouted with high strength mortar. The proposed location of the joint is at mid-height of the precast shear wall which is less stressed during earthquake excitations [1].

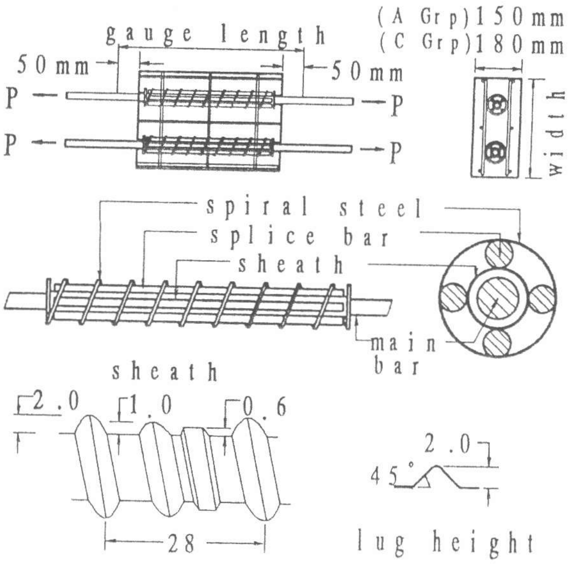

Figure 1. Design of Connection

This is the second in the series of experimental tests. The pioneering test was done in 1992 [1] where its tensile capacity was investigated at 200-mm wall thickness. The proposed connection was concluded to be structurally adequate as it provided tensile resistance more than main bar yield strength when the wall thickness was 200mm and the splice length was 20 times the lapped bar diameter. This conclusion led to the idea of further testing the tensile capacity of the connection at a reduced wall thickness of 150mm and 180mm. The main bar was changed from D25(SD390) to D22(SD490) and the steel sheath diameter from 42 mm to 38 mm. The variation of strength was done because in the first series, all connection failures happened after yielding of main bar. In this series, collapse before main bar yielding was expected.

2. SPECIMENS

A total of 24 wall specimens were tested for 180-mm thickness and another 24 specimens for 150 mm. For each thickness, there were eight variations of parameters at three specimens per variation.

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Each variation was treated as a group. Group A1 to A8 (A Group) were the designations used for 150-mm thick wall specimens, while Group C1 to C8 (C Group) were assigned for those with 180-mm thickness. The design of A Group is identical to that of C Group except the thickness. These group designations are used in the succeeding discussion. It should be noted that in the first series of experiments, D25(SD390) main bar was used, but in this series, D22(SD490) was employed. Screw-type reinforcing bars were used for convenience in testing. To ensure failure on main bar or sheath, high strength bars 4-D13(SD780) were used as lapped splices.

The constant factors are compressive strengths of concrete and grout, steel yield strength, spacing and height of the ribs of main bars and lapped bars. Table 1 summarizes the variations of parameters. The reference of all variations of parameters was the design of specimens in group A1 and C1. These specimens are herein called "reference specimens". They have a length of 600 mm, width of 400 mm, main bar spacing of 200 mm, splice length of 20d (20 times the lapped bar diameter), steel sheath diameter of 38 mm, lug heights of 2.0 mm and 1.5 mm in one turn(28-mm pitch), spiral steel(6-mmϕ) pitch of 60 mm with 75-mm inner diameter, and 4-D13(SD780) lapped bars.

Table 1. Variation of Parameters

Varied Factors	A Group t=150mm	C Group t=180mm	W x L (mm x mm)	Main Bar	Main Bar Spacing	Lapped Length	Sheath Diameter	Spiral Pitch
Reference Factors	A1	C1	400 x 600	2-D22	200mm	20d	38mm	@60mm
Main Bar (2-D25)	A2	C2		2-D25			42mm	
Spacing (300 mm)	A3	C3	600 x 600	2-D22	300mm		38mm	
Length (15d) (25d)	A4 A5	C4 C5	400 x 470 400 x 730		200mm	15d 25d		
Diameter (42mm)	A6	C6	400 x 600			20d	42mm	
Pitch (@30mm) (@90mm)	A7 A8	C7 C8					38mm	@30mm @90mm

d: lapped bar diameter

D22 main bar was changed to D25 in A2 and C2 to compare the tensile capacity of the connection at a reduced wall thickness to that of the previous experiment [1]. Main bar spacing was changed from 200 mm in the reference specimens to 300mm in group A3 and C3. Lengths of lapped bars were varied from 20d to 15d in groups A4 and C4 and to 25d in groups A5 and C5. The 38-mm sheath diameter was varied to 42 mm (confined to spiral steel with inner diameter of 80 mm) in groups A6 and C6. For the pitch of spiral steel which is 60 mm in the reference specimens, 30 mm in groups A7 and C7 and 90 mm in groups A8 and C8 were the alterations made.

All tubular steel sheaths were set in a vertical position. At both ends of each sheath, two main bars were inserted until they met at mid-height. The lower end of the sheath was sealed with a rubber cap to prevent mortar leakage. The high strength mortar was poured from the top end filling the space between the sheath and main bar. After one week of curing, four lapped splices were positioned around a joined sheath and main bar which were then confined to a spiral steel. The assembly can be seen in Fig. 1. These assemblies, together with mesh reinforcements 6-D10(SD295A) for both longitudinal and lateral directions, are placed in the formworks of the specimens lying in a horizontal position.

3. MATERIAL PROPERTIES

Table 2 summarizes the specified and the actual material properties. Each value was the average of three test pieces.

Concrete. Ordinary type of concrete with a specified compressive strength of 300 kgf/cm^2 was cast. The actual compressive strengths at the beginning and at the end (after 9 days) of testing were 410 kgf/cm^2 and 420 kgf/cm^2, respectively.

Grout. The specified compressive strength of mortar was 600 kgf/cm^2 but the actual compressive strength was 690 kgf/cm^2.

Table 2. Material Properties

(a) Concrete and Grout unit: kgf/cm^2

	Specified Strength	Actual Compressive Strength	Actual Splitting Strength
Concrete	300	410, 420	31, 34
Grout	600	690	-

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

Size	Grade	Specified Yield Strength	Actual Yield Strength	Actual Tensile Strength	Remarks
D25	SD490	5.0	4.97	5.19	main bar
D22	SD490	5.0	5.18	6.45	main bar
D13	SD780	8.0	8.76	10.60	splice
D10	SD295A	3.0	3.83	4.39	mesh
ϕ6	---	---	4.50	7.08	spiral

Steel. All steel bars yielded at strengths higher than the specified except D25(SD490) which had an actual yield strength slightly lower than the specified. D22(38mmϕ sheath) and D25(42mmϕ sheath) main bars were confined to 6.0–mm spiral steel with inner spiral diameter of 75 mm and 80 mm, respectively. This spiral steel yielded at 4.50 $tonf/cm^2$.

Tubular Steel Sheath. Lugs with heights of 2.0 mm and 1.0 mm consist of the corrugation in one turn. Spiral sheath overlap with a height of 0.6 mm may also be considered another lug. The pitch of each lug was 28 mm. The thickness is 0.25 mm.

4. TEST SETUP AND TESTING PROCEDURE

With the specimen positioned horizontally, equal tensile forces were applied simultaneously by oil jacks at each end of main bars. The elongation between the ends of each rebar was recorded using cantilever type displacement transducers. As shown in Fig. 1, the deformation along the main bar axis was obtained by extrapolating the actual displacements of gauges. In each of the two specimens in groups A6, C6, A7, C7, A8, and C8, two strain gauges were placed on the quarter points of the total length of spiral steel and the other two at the center and quarter point of a lapped bar.

5. TEST RESULTS

Test results of maximum tensile loads (per main bar) are plotted in Fig. 2. It also shows the average bond stresses on the surface of main bar and steel sheath. The values were calculated using the following formula:

$$\tau = \frac{P_{max}}{(l_s \times \phi)} \qquad (1)$$

where,

P_{max} = maximum tensile load of one main bar

l_s = lap splice length

ϕ = perimeter of main bar, sheath, or four lapped bars.

Since the tensile capacity of the three specimens in a group are of slight differences, only the results per group are discussed. Typical load–displacement diagrams are shown in Fig. 3. Using the test results graphed in Fig. 2, the effects of the following variables on the tensile capacity of the connection are discussed.

Effect of wall thickness. Fig. 2(a) shows that the average ultimate loads of wall specimens with 150–mm and 180–mm thickness in groups A2 and C2 are almost equal at 20 tonf. These loads are almost equal to 0.8Fy (Fy = 25.2 tonf (SD25 SD390), actual yield strength of main bar multiplied by its cross sectional area). Compared to group B1 (200 mm thickness), these loads are lower although the strengths of main bar and concrete in A2 and C2 are higher than those in B1. The reduction in thickness caused a decrease in tensile strength.

Effect of main bar spacing. As shown in Fig. 2(b) and 2(c), the tensile capacity of the connection is almost constant when the spacing is changed from the reference of 200 mm in group A1 and C1 to 300 mm in groups A3 and C3, respectively. Where Fy = 20 tonf (D22 SD490), groups A1 and C1 attained an average capacity of 21 tonf/cm^2 (1.05Fy) and 22.3 tonf/cm^2 (1.11Fy), respectively. Groups A3 and C3 reached 1.13Fy and 1.16Fy, respectively. As can be noticed in Fig. 3, the yield point of the connection in group A1 is at P = 17.0 tonf and δ = 1.5 mm, but in group C2, it was at P = 20.0 tonf and the same displacement of 1.5 mm. Both groups A3 and C3 have connection yield points at around 18–tonf load and 1.2–mm displacement. The bond stress on the sheath is at around 65 to 70 kgf/cm^2 from 200 mm to 300 mm spacing of main bars. At this range of bond stress, all connections in these two groups failed on the sheath.

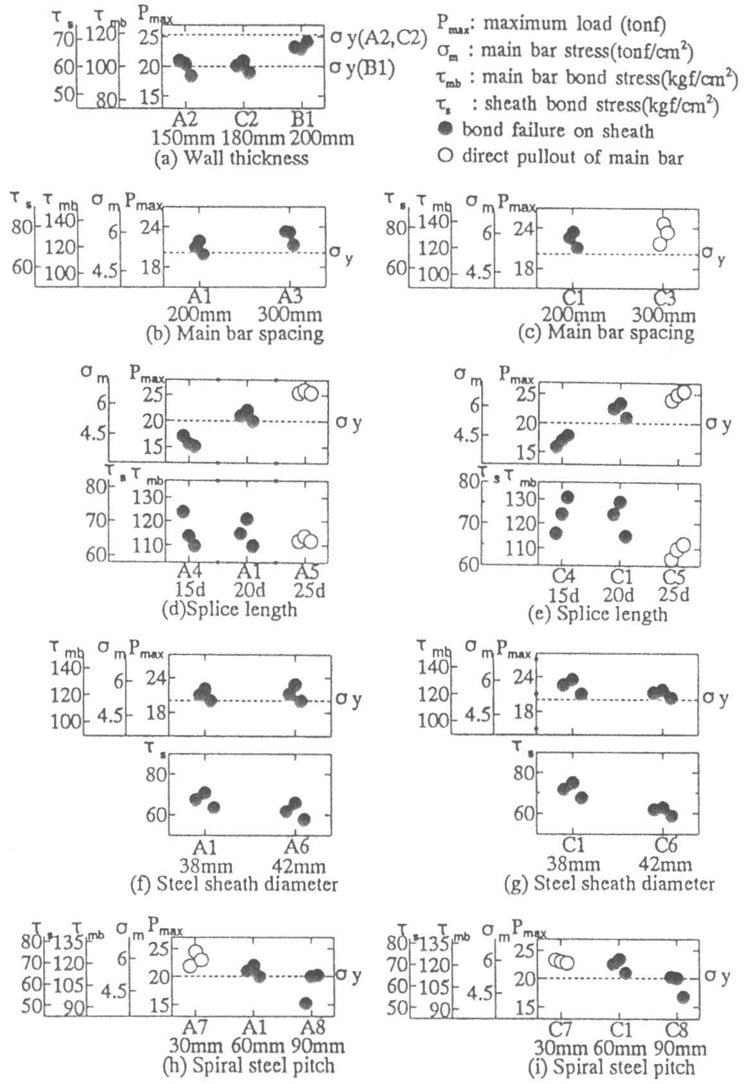

Figure 2. Maximum Loads

Effect of splice length. Fig. 2(d) and 2(e) indicate that the ultimate tensile load increases as the splice length increases from 15d to 20d and 25d. Groups A4 and C4 reached a maximum load of 15.8 tonf (0.79Fy) and 16.9 tonf (0.85Fy). No yield point was noticed on the diagrams which means that it was not able to accomodate large deformations. The connections collapsed at δ = 1.5 mm. The failure was bond failure on the sheath in these two groups. On the other hand, the maximum tensile capacity of groups A5 and C5 were 25.7 tonf (1.3Fy) and 24.9 tonf (1.2Fy), respectively. The connections of the specimens yielded at around 18–tonf load and 1.5–mm displacement but were able to sustain additional strengths almost equal to 1.3Fy and displacements of 10 mm. The failure was direct pullout of main bar at bond stress of approximately 110 kgf/cm^2.

Effect of steel sheath diameter. It can be observed in Fig. 2(f) and 2(g) that the diameter of the steel sheath has a slight influence on the tensile strength of the joint. In 150–mm thick specimens, the maximum loads for 38–mm and 42–mm sheath are almost constant, while in 180–mm thickness, the load at 42–mm is a slightly lower. Both Groups A6 and C6 reached 1.06Fy which is almost equal to that of groups A1 and C1. The final displacement of the connection when the sheath diameter is 42 mm is smaller at around 5.0 mm compared to almost 8.0 mm when the diameter is 38 mm. The yield points of all the connections in the four groups mentioned were at around 18 tonf–load and 1.5 mm–displacement. The specimens failed by bond on the sheath.

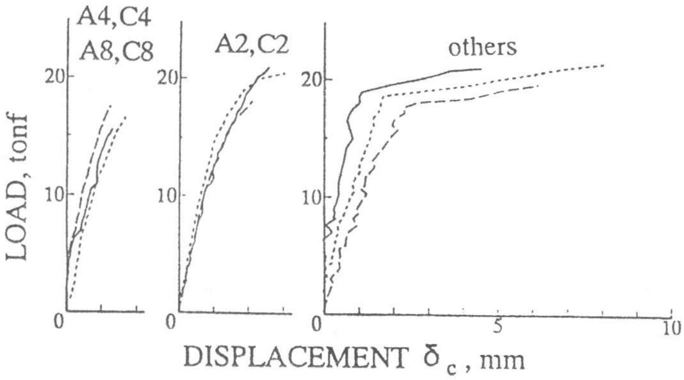

Figure 3. Typical Load – Displacement diagrams

Effect of spiral steel pitch. In Fig. 2(h) and 2(i), it can be noticed that the tensile capacity of the connection decreases as the pitch of spiral increases. Groups A8 and C8 with a spiral steel pitch of 90 mm failed at a capacity of 18.3 tonf (0.91Fy) and 19 tonf (0.95Fy), respectively. The maximum displacement was similar for the groups at around 1.5 mm to 2.0 mm. After reaching the yield point, the connections failed. They were not able to sustain large deformations. The failure was by bond on the sheath. On the other hand, both groups A7 and C7 with a spiral pitch of 30 mm reached 1.15 Fy although they yielded at a load of 18 tonf. Their displacements at yield were at around 1.5 to 2.0 mm and the maximum displacements were almost 10 mm. Direct pullout of main bar was the failure mode. At 30–mm pitch the strain on spiral at quarter points was around 400μ while at 60 and 90–mm pitch, strain increased to approximately 600μ and 1000μ, respectively. These values are below yield strain (2100μ). The lapped bar confined by 30–mm pitch spiral was stressed most having a strain of 1700μ at the center and 900μ at quarter point. These values prove that lapped bars carried the entire load after the concrete cracked especially at high load levels.

The typical crack patterns are shown in Fig. 4. The failure pattern in all the specimens started by cracking at the center perpendicular to main bars. All other cracks occured from the ends of the specimen and progressed along the location of main bars. These cracks developed little by little to the center as the tensile load increases. This implies that the bearing stress on the sheath increases gradually throughout the length. This bearing stress caused components perpendicular (radial stresses splitting the concrete) and parallel to the main bar. In longer specimens, intermediate cracks perpendicular to the main bars occured because the parallel components in that region is bigger than perpendicular ones. The main cause of connection failure was the longitudinal cracking of concrete.

7. DISCUSSION OF TEST OBSERVATIONS

It can be noticed in Fig. 3 that the load displacement diagrams resemble like the stress–strain curve of the main bar although the former has a lower yield strength. The sharp curves on the diagrams can be referred to as connection yield point. This point occurs at a load of 16 to 19 tonf and a displacement of 1.5 to 2.0 mm. Should there be a continuous bar and assuming 2100 tonf/cm^2 for

Young's modulus, the yield displacements are 1.4 mm (A4,C4), 1.66 mm(A5,C5) and 2.0 mm (others), respectively. From the actual yield displacements, it can be stated that actual main bar displacement during failure is only a part of the total displacement along the gauge length. The other part is accounted for by concrete displacements. The concrete splits longitudinally and laterally due to circumferential stress exerted by sheath. The yield of the connection happens when the sheath has displaced 1.5 to 2.0 mm (equivalent to half of lug width, approximately equal to lug heights). At this displacement, the concrete has been pushed and the bearing area is reduced or lost. Some specimens reached higher loads after yielding because of the force needed to push the other blocking concrete.

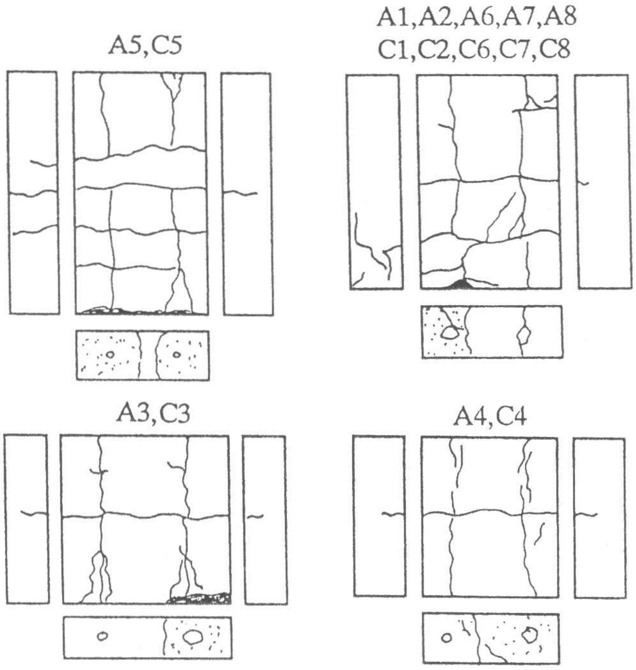

Figure 4. Typical Crack Patterns

8. CONCLUSION

The conclusions and recommendations drawn are the following:

1. The tensile capacity of main bar splice is reduced when the thickness of the wall is reduced from 200 mm to 180-mm and 150-mm.
2. Within the elastic stage of main bar, the failure of the connection is by bond on the sheath.
3. A main bar spacing of 300 mm in 180 mm-thick wall, pitch of spiral of 30 mm and splice length of 25d will provide direct pullout of main bar.
4. At 150 mm and 180 mm-thickness, the yield of the connection is at approximately 18.0-tonf load and a displacement of 1.5 mm to 2.0mm which is almost equal to half of sheath lug width.
5. From 15d to 25d splice length, the maximum load increases as the splice length increases. The maximum bond stress on the sheath is approximately 70 kgf/cm^2.
6. The diameter of steel sheath has a negligible effect when the wall thickness is 150 mm. At 180-mm thickness, 42-mm sheath gives slightly lower resistance compared to 38 mm.
7. Spiral steel with greater pitch is more stressed compared to that with smaller pitch although smaller pitch has greater confinement capacity.
8. Increasing the spacing of main bar from 200 mm to 300 mm will provide slight increase on the capacity.

It is recommended to conduct additional tests in order to obtain quantitative information on the failure mechanics of the connection.

REFERENCE

1. Adajar, J. C., Yamaguchi T. and Imai H., "An Experimental Study on the Tensile Capacity of Vertical Bar Joints in a Precast Shearwall", Transactions of the JCI, Vol. 15, Dec.,1993, pp.564-567.