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Resistance Mechanism of Low Strength Concrete Columns with Wing Walls

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ABSTRACT: Seismic evaluations of existing reinforced concrete buildings have been extensive in Japan since the 1995 Kobe earthquake. According to various reports, e.g. Hirosawa & Sakamaki (2001) or Takano et al. (2002), seismic evaluations found many RC buildings to have very low concrete strength, *i.e.* less than half of the design concrete strength. Although those buildings that have serious problems are intended for continuous use, we still do not have any fundamental knowledge of low strength concrete. In this paper, seismic tests of RC columns with wing walls were conducted to investigate the resistance mechanism of those columns. Test columns were manufactured with low strength concrete of around 10MPa and plain round bars as the longitudinal reinforcement of the column. In the test results, splitting failures at the boundary between the column and wing walls were observed in all test columns. It is confirmed that the calculated shear strength (by cumulating the strength of the column in consideration of bond deterioration of the main bars and the strength of the wall) agreed with the test results.

1 GENERAL INSTRUCTIONS

1.1 Objects

Previous studies on low strength concrete were performed focusing on shear strength because the shear member was a significant factor in the evaluation of the seismic performance of existing buildings. However, from the experimental results it was found that the flexural strength with plain round bars was much lower than the values calculated by the present equation. The resistance mechanism of RC members with both low strength concrete and plain round bars requires further investigations because the seismic performance of existing buildings depends mainly on the flexural members. The objective of this study is to clarify the seismic behaviour of those columns with wing wall. Based on the test results a design procedure to obtain the maximum strength with consideration of bond slip characteristics was developed.

1.2 Low strength concrete

It was reported that in many samples of low strength concrete, less than 10MPa was found in concrete cylinders obtained from existing buildings constructed during the 1960s and 1970s, although according to previous studies, 180kgf/cm^2 (18MPa) was typically used in Japan before the building standards revision in 1970. 6.1% of the total inspected concrete cylinders of the existing buildings were found to have a strength of less than 13.5MPa (13.5MPa is the recommended lower limit of concrete strength in the Standard for Seismic Evaluation of

Existing Reinforced Concrete Building (2001). In this study, low strength concrete of around 10MPa was manufactured in the laboratory for the test columns.

2 OUTLINE OF EXPERIMENTS

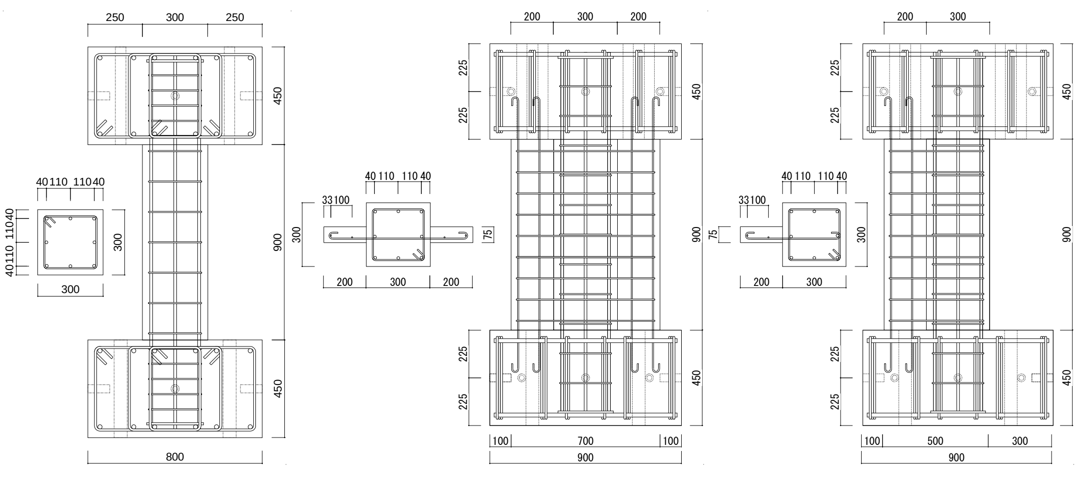
2.1 *Test columns*

Test column without wing wall LS30-15 was designed as the shear failure type according to the Standard for Structural Calculation of Reinforced Concrete Structures of the Architectural Institute of Japan (1991). Columns with wing walls were also designed as the shear failure type. The column sections were 300mmx300mm. The shear span ratios of the column part were 1.5. The sections of the wing wall were 75mmx200mm. The specified low strength concrete strengths were 18MPa and 9MPa. Normal strength concrete of 18MPa was used in NS15-15W2. Details of the columns are shown in Figure1. Plain round bars (13ϕ) were used as the main bar in all of the columns. The main bars were anchored with steel plates in the stub. The hoop reinforcement ratio p_w varied from 0.15 to 0.3% in the column part. Deformed round bars (D6) were used in the hoops and wall reinforcements. The wall reinforcement ratio p_s was 0.43%. The wall reinforcements were anchored in the column section and in the stubs. A list of the columns is shown in Table 1.

Table 1. List of test columns

Test Column	Axial Force η	Concrete Strength [MPa]	Longitudinal Reinforcement SR235	Hoop Reinforcement SD295A	Wall Reinforcement SD295A	Wing Wall
LS30-15	0.15	9	8-13φ	2-D6@140	-	No
NS15-15W2	0.15	18			D6@100	Both-side
LS30-15W2	0.30	9				
LS15-15W2	0.15					
LS30-30W2	0.30			2-D6@70		
LS30-15W1				2-D6@140		One-side

η=N/BD N:Constant axial force level



Column without wall Columnwith wing wall on both-side Column with wing wall on one-side

Figure 1. Details of the test columns and location of strain gages.

Table 2. Mix properties of concrete

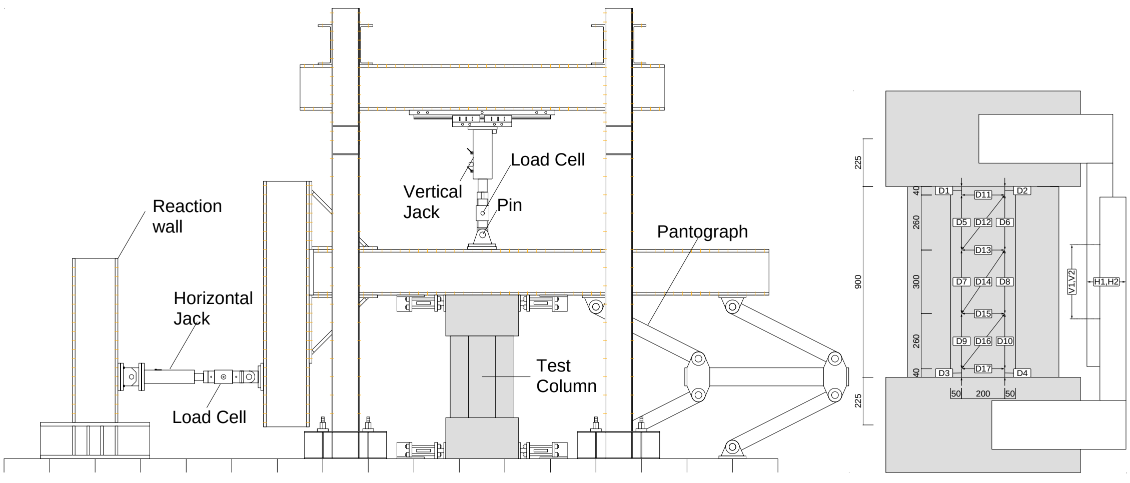
Specific Strength	Unit (kg/m^3)				
	Cement	Water	Sand	Coarse Aggregate	Additive
Fc18	294	191	870	916	1.61
Fc9	195	215	923	864	1.95

2.2 *Material*

The mix properties of low strength concrete of 9MPa shown in Table 2 were defined by the preliminary mixing tests. The only considered parameter in the mixing tests was the water/cement ratio, which was the predominant factor in concrete strength. The water/cement ratio of 9MPa concrete in this paper was 110% using a high-range water-reducing additive to avoid segregation in flesh concrete. A compressive test was performed 28 days after concrete casting. The strengths of concrete Fc9 and Fc18 were 12.2MPa and 18.4MPa respectively. The yield strengths of the 13ϕ plain round bars and the shear reinforcement D6 were 345MPa and 361MPa, respectively as derived from tensile tests.

2.3 *Test setup and instruments*

The test setup was designed to subject the column to lateral load reversals, while the axial load remained constant. The top stub was fixed to the L shaped steel beam and the bottom stub was fixed to the reaction floor with high-tension bolts. To ensure that the top and bottom stubs remained parallel during reversal loadings a pantograph system was used. The test setup is shown in Figure 2. Displacement transducers used to measure flexural and shear displacements were mounted on the columns and the wing walls. The lateral and vertical loads were measured by load cells instrumented to the jacks with a pin joint. Lateral load was applied through the L shaped beam under displacement control. Considered axial loads were constant at 0.15 and 0.30 (η=N/BD) during lateral loading. The end of the vertical jack was supported with a high performance slider. The lateral loadings were carried out under displacement control, attempting two cycles for each of the peak displacement levels of drift angle R=1/400rad,1/200rad, 1/100rad, 1/66rad, 1/50rad, 1/33rad and 1/20.



Loading apparatus

Measurement system

Figure 2.Test setup and displacement transducers on the test column.

LS30-15 NS15-15W2 LS30-15W2 LS15-15W2 LS30-30W2 LS30-15W1

Figure 3.Crack patterns at drift angle 1/20rad.

3 TEST RESULTS

3.1 Crack patterns

Crack patterns at drift angle 1/20rad. are shown in Figure 3. Flexural cracks occurred at the boundary between the stub and the column by drift angle 1/400rad. in all test columns. In the test column without wing wall LS30-15, shear cracks did not occur throughout lateral loadings. The spalling and crushing of the concrete cover progressed in the hinge region at the top and bottom ends of the column when the drift angle increased. Bond slip failure of plain round bars was observed. In the column with wing wall on both-side of normal strength concrete NS30-15, the flexural cracks and shear cracks occurred at both ends of the wall by drift angle 1/200rad. The shear cracks occurred before the maximum strengths were observed and the shear cracks progressed when the drift angle increased. The width of the flexural cracks did not increase. The final failure mode was diagonal shear failure type. In the low strength concrete columns with wing wall on both-side, the bond splitting failure occurred along the vertical reinforcing bars in the wing wall at drift angle 1/200rad. The splitting cracks then occurred along the boundary between the column and wing walls. When the diagonal shear cracks in the wing walls did not progress, spalling and crushing of the concrete cover was observed at the top and bottom ends of the walls. The failure mode at the final stage was bond slip failure after the crushing of the concrete of the wing walls. The concrete cover near both ends of the column parts was also entirely demolished and buckling of the main bars was observed.

3.2 Shear force-drift angle response

Shear force-drift angle hysteretic responses are shown in Figure 4. Broken lines and solid lines show the calculated strength obtained by equation (1) and (2) respectively. M_u is the flexural strength assuming the yielding of the main bars in the column parts and Q_{su} is the shear strength in the Standard for Seismic Evaluation of Existing Reinforced Concrete Buildings (2001). The critical drift angle of 80% of the maximum strength is inserted into the figures.

$$M_u = 0.8 a_t \cdot \sigma_y \cdot D + 0.5 N \cdot D \left(1 - \frac{N}{b \cdot D \cdot F_c} \right) \tag{1}$$

$$Q_{su} = \left\{ \frac{0.053 \cdot k_p \left(18 + \sigma_{\mathrm{B}}\right)}{M/Qd + 0.12} + 0.85 \sqrt{p_w \cdot \sigma_{wy}} + 0.1 \sigma_0 \right\} b \cdot j \tag{2}$$

a_t : Sectional area of main bar (mm^2) b : Column width (mm) D : Column depth (mm)
d : Effective depth of column (mm) F_c : Specified concrete strength (N/mm^2)
j : Internal lever arm (mm) N : Axial force (N) M/Q : Shear span (mm)
p_t : Ratio of main bar (%) p_w : Ratio of hoop reinforcement σ_0 : Axial load level (N/mm^2)
σ_B : Concrete strength (N/mm^2) σ_y : Yield strength of main bar (N/mm^2)
σ_{wy} : Yield strength of hoop reinforcement (N/mm^2)

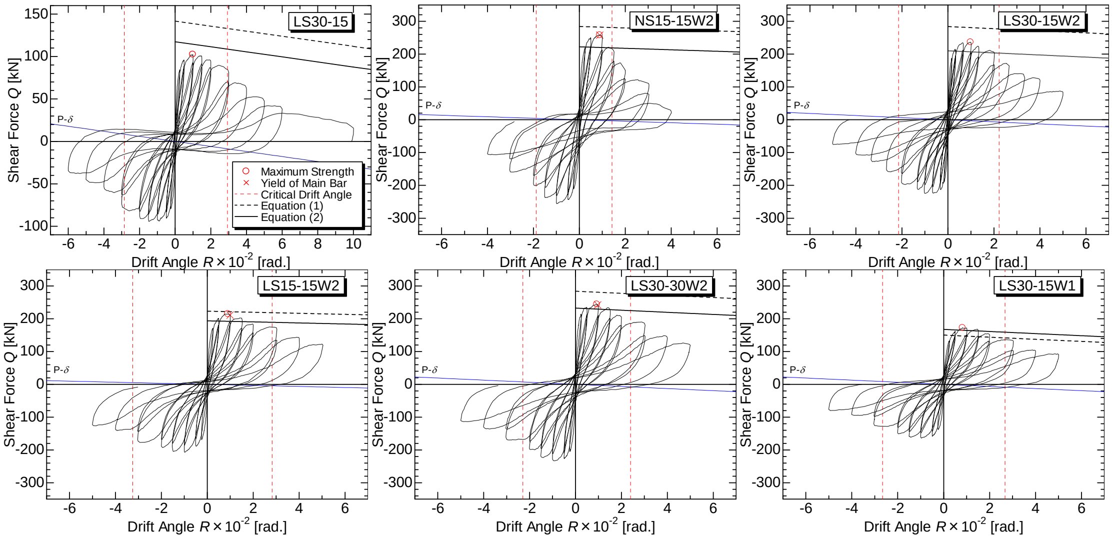
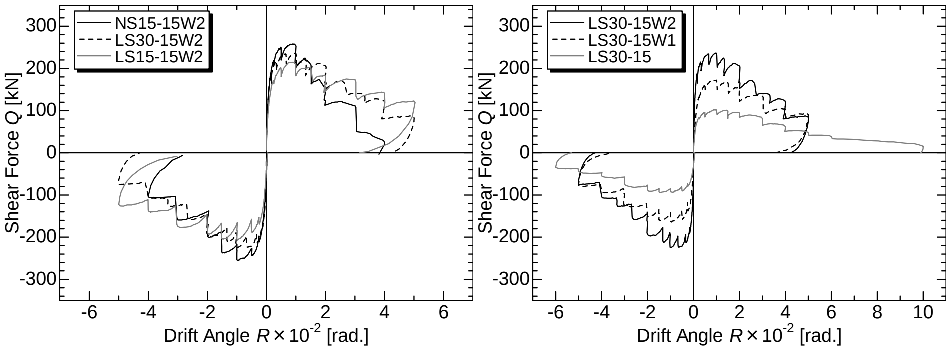


Figure 4. Shear force-drift angle respose

In all columns the hysteresis loops show slip type as bond slip failure and the shear force decreased when the drift angle increased. In the column without wall LS30-15, the maximum strength did not reach the calculated shear strength. In the column with wing wall on both-side NS15-15W2 (normal strength concrete), the maximum strength reached 257.9kN at drift angle R=1/100rad, and exceeded the calculated shear strength. The shear force decreased rapidly after the maximum strength. The critical drift angle was approximately 1/70rad, which was the minimum value in all columns. In the column with wing wall on both-side LS30-15W2 with a high axial force level, the shear force reached the maximum at the drift angle R=1/50rad. and the critical drift angle was 1/45rad. The shear force after the maximum strength decreased more gradually than that of the column with wing wall on both-side LS15-15W2 with a low axial force level. The maximum strength of LS15-15W2 was 214.8kN, which was the minimum value in all columns with wing wall on both-side. However, the maximum strength was approximately the same value as the calculated flexural strength because yielding of the main bars was observed at that time. In the column with wing wall on both-side LS30-30W2 with a large amount of hoop reinforcement, the shear force did not reach the calculated flexural strength although yielding of the main bars occurred. The maximum strength was 243.9kN, which was the maximum value in the low strength concrete columns. In the column with wing wall on one-side LS30-15W1, the maximum strength was 172.6kN.

3.3 Experimental variable

Comparisons of the envelope curves of shear force-drift angle relations are shown in Figure 5. To investigate the effect of the axial force level, three test columns with the same section of the column and the wing wall were compared. The subjected axial force for the column with wing wall on both-side NS15-15W2 was 227kN (η=0.15) and the axial forces for columns LS30-15W2 and LS15-15W2 were 243kN (η=0.3) and 161kN (h=0.15) respectively. The axial force N were estimated by using the concrete strength σ_B obtained from the material tests. There were no significant differences between the two low strength concrete columns. The envelope of the column with the lower axial force showed a little more ductile behavior than that of the column LS30-15W2 with the higher axial force. The maximum strength of the normal strength concrete column was the highest strength. The shear force decreased more rapidly than the other two low strength concrete columns. Concerning the arrangement of wing walls, three test columns subjected to the same axial force were compared. Apparent differences were observed in the



(a)Axial force level (b)Number of walls

Figure 5 Envelope curves of shear force drift angle

three test columns. The maximum strength of the column with wing wall on both-side LS30-15W2 was 1.37 times that of the columns with wing wall on one-side LS30-15W1 and 2.31 times that of the columns without wing wall LS30-15. It was confirmed that the wing walls significantly affected the maximum strength and ductility. The envelopes of the columns with wing wall decreased rapidly and approached that of the column without wing wall. This was due to the severely damaged wing walls when the drift angle increased.

4 DISCUSSIONS

4.1 Maximum strength

The maximum strengths of all test columns are listed in Table 3. Calculated flexural and shear strength were obtained theoretically by two equations (1) and (2). The section of the column with wing wall was assumed to be rectangular in the equations. In the calculations for the column with wing wall on one-side LS30-15W1, the strength was the average of the calculated strength both ignoring and considering the wing wall. In the column without wall LS30-15, the calculated shear strength was overestimated due to the bond slip failure of the main reinforcement of the column part. The maximum strength of the columns with wing wall could be approximately predicted by Eq. (2). The ratio between the observed and calculated strength of the column with wing wall was 1.02~1.14, however, the observed failure modes were different from diagonal shear failure except for the normal strength concrete column NS15-15W2. It is necessary to develop a prediction method corresponding to the failure mechanism.

Table 3. Summary of Maximum Strength

<table>
<tr><th rowspan="2">Test Column</th><th rowspan="2">Observed [kN]</th><th colspan="2">Calculated [kN]</th><th rowspan="2">Failure Mode</th></tr>
<tr><th>Flexural Strength Eq.(1)</th><th>Shear Strength Eq.(2)</th></tr>
<tr><td>LS30-15</td><td>102.5</td><td>141.7 (0.72)</td><td>117.4 (0.87)</td><td>SB</td></tr>
<tr><td>NS15-15W2</td><td>257.9</td><td>284.3(0.91)</td><td>226.5(1.14)</td><td>S</td></tr>
<tr><td>LS30-15W2</td><td>236.6</td><td>284.0(0.83)</td><td>214.3(1.10)</td><td rowspan="4">SB</td></tr>
<tr><td>LS15-15W2</td><td>214.8</td><td>223.1(0.96)</td><td>197.9(1.09)</td></tr>
<tr><td>LS30-30W2</td><td>243.9</td><td>284.0(0.86)</td><td>235.8(1.03)</td></tr>
<tr><td>LS30-15W1</td><td>172.6</td><td>150.2(1.41)</td><td>168.7(1.02)</td></tr>
</table>

( ) : Observed/Calculated SB : Splitting failure after bond slip failure S : Shear failure

4.2 Proposed Method

It is important to evaluate the bond strength of the plain round bar in the columns because the main failure patterns were bond slip failure in the tests when the shear force reached the maximum strength. In previous studies, Wakabayashi & Minami (1985) proposed to cumulate the strength of the column and the strength of the wing walls to obtain the maximum strength of the columns with wing wall as shown in Equation (3).

$$V_u = V_{uc} + V_{uw} \tag{3}$$

V_{uc} : Shear strength of column (N) V_{uw} : Shear strength of wall (N)

Shear strength of the low strength column was calculated considering the bond slip mechanism proposed by Araki & Iki (2011) based on the truss and arch theory. Shear strength of the walls was calculated using the arch theory shown in the ultimate strength concept of the Architectural Institute of Japan (1990).

$$V_{uc} = V_t + V_a \tag{4}$$

V_t : Shear strength from truss mechanism (N) V_a : Shear strength from arch mechanism (N)

$$V_t = min\left\{\Sigma\left(\tau_b \cdot \psi\right) j_t,\ b \cdot j_t \cdot p_w \cdot \sigma_{wy}\right\} \tag{4-1}$$

$$V_a = \left\{\nu \cdot \sigma_B - \frac{2.5V_t / j_t}{b}\right\} \frac{b \cdot D}{2} \cdot \tan\theta \tag{4-2}$$

$$\tan\theta = \sqrt{\left(L/D\right)^2 + 1} - L/D \qquad \nu = 0.7 \text{-} \sigma_B/200 \qquad \sigma_B \le 13.5 \Rightarrow \nu = 1$$

j_t : Distance between tension and compression bars (mm) L: Clear span length of column (mm)
p_w : Ratio of hoop reinforcement b : Column width (mm) D : Column depth (mm)
ν : Reduction factor for concrete σ_B : Concrete strength (N/mm^2)
σ_{wy} : Yield strength of hoop reinforcement (N/mm^2) θ : Angle of compression strut
ψ : Circular length (mm) τ_b : Bond stress (N/mm^2) (=0.09σ_B, 1.98)

$$V_{uw} = t_w \cdot l_{wb} \cdot p_s \cdot \sigma_{sy} \cdot \cot\phi + (1-\beta)\nu \cdot \sigma_B \frac{t_{wb} \cdot l_{wa}}{2} \cdot \tan\theta \tag{5}$$

$$\beta = \left(1 + \cot^2\phi\right) p_s \cdot \sigma_{sy} \Big/ (\nu \cdot \sigma_B) \qquad \tan\theta = \sqrt{\left(h_w/l_{wa}\right)^2 + 1} - h_w/l_{wa}$$

b : Column width (mm) D : Column depth (mm)
σ_{sy} : Yield strength of wall reinforcement (N/mm^2) t_w: Thickness of wall (mm)
p_s : Ratio of wall reinforcement ϕ : Angle of compression strut in truss mechanism
h_w: Height of wall (mm) L_{wa}, L_{wb} : Equivalent wall length in truss and arch mechanism (mm)

Considering bond deterioration of the main bars the predicted maximum strengths were in agreement with the observed values regardless of the failure mode shown in Table 4.

Table 4. Comparison between observed and calculated strength by proposed method

Test Column	Observed [kN]	Calculated [kN]	Observed/Calculated
NS15-15W2	257.9	245.2	1.05
LS30-15W2	236.6	210.2	1.13
LS15-15W2	214.8	210.2	1.02
LS30-30W2	243.9	210.2	1.16
LS30-15W1	172.6	167.4	1.03

5 CONCLUSIONS

Based on the investigation of the test results of the columns with wing wall, the following conclusions can be made.

1) Final failure modes of the low strength concrete column with wing wall were the splitting failure between the column part and the wing wall part after bond slip failure in the column.
2) The wing wall contributed to the increase of the maximum strength of the columns with wing wall. The shear strength of the column with wing wall approached that of the column without wing walls after the wing walls were severely damaged.
3) It is possible to predict the maximum strength cumulating the strength of the column and the wing wall. When calculating the strength of the column, it is necessary to evaluate the bond deterioration of the main bars in the low strength concrete column.

6 ACKNOWLEDGEMENTS

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