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SEISMIC BEHAVIOR OF REINFORCED CONCRETE COLUMNS WHICH USED ULTRA-HIGH-STRENGTH CONCRETE

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ABSTRACT

The object of this study is to assess the seismic behavior of reinforced concrete columns which used urtra-high-strength concrete. Five specimens of 120 MPa concrete with lateral reinforcement of ultra-high-strength steel bars (yield strength of 1,380 MPa) were tested under reversed cyclic loading. Test variables were 1) axial stress level and 2) capacity of lateral reinforcement in terms of the product of its yield strength and area ratio. The effect of these variables on failure mode, ultimate strength and ductility of the column was discussed including results of the author's previous tests of columns using 40 - 80 MPa concrete. Based on regression analysis using all the available column test data of high strength concrete, empirical equations to evaluate ultimate displacement were proposed.

KEY WORDS

reinforced concrete, column, high-strength concrete, high-strength steel, ultra-high-strength material, lateral reinforcement, ductility, ultimate displacement, axial force, regression analysis

INTRODUCTION

It is recognized that the use of high strength concrete makes taller reinforced concrete buildings possible even in high seismic regions. This trend has been increasing in Japan and concrete compressive strength of 60 MPa is already used in several tall buildings. Use of concrete strength higher than 100 MPa is considered to be achieved in near future. The object of this study is to assess the seismic behavior of reinforced concrete columns which used urtra-high-strength concrete. Five specimens of 120 MPa concrete laterally reinforced with ultra-high-strength steel bars were tested under reversed cyclic loading. Including results of author's previous tests of columns using 40 - 80 MPa concrete, the effect of axial stress level and the capacity of lateral reinforcement on failure made, maximum strength and ductility was discussed. Regression analysis on displacement ductility using all the available column test data of high strength concrete was carried out.

EXPERIMENTAL PROGRAM

Details of Specimens and Testing Procedure

Table 1 shows property of specimens. All the specimens had 225 x 225 mm square section and shear span/ depth ratio of 2.0 as shown in Fig.1. Specified compressive strength of 120 MPa concrete was cast vertically. The maximum size of coarse aggregate was 20 mm. Silica fume was added to increase strength. Measured concrete compressive strength and modulus of elasticity at the test were 118 MPa and 47,070 MPa, respectively. Ultra-high-strength deformed bars (nominal diameter of 5.1 and 6.4 mm) with yield strength of 1,380 MPa were used for lateral reinforcement. Spiral type outer square ties and sub-ties with 135° hooks extending with the length 8 times the bar diameters were provided. Deformed bars with nominal diameter of 10 mm (D10) were used for longitudinal reinforcement. Table 2 shows mechanical characteristics of steel bars.

Test variables were 1) ratio of axial stress to concrete compressive strength (σ_0 / f_c') ; 0.60 and 0.35, and 2)

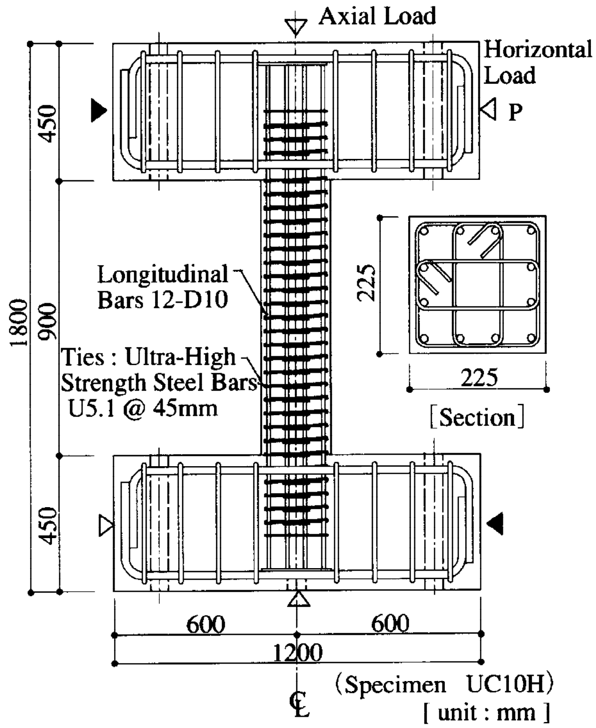


Fig. 1 Test column

Table 1 Properties of specimen

Specimen	Axial Stress Ratio σ_o/fc	Area Ratio of Ties $P_w(\%)$	$P_w \cdot f_yh$ (MPa)	Ties
UC10H	0.62	0.77	11.0	4-U5.1@45
UC15H	0.62	1.19	16.9	4-U6.4@45
UC20H	0.62	1.52	21.7	4-U6.4@35
UC15L	0.36	1.19	16.9	4-U6.4@45
UC20L	0.36	1.52	21.7	4-U6.4@35

σ_o : Axial stress, f_{yh} : Yield strength of ties
Common factors for all the specimens
f_c : Compressive strength of concrete cylinder = 118 Mpa
Shear span / Depth = 2.0

Table 2 Mechanical properties of reinforcing bars

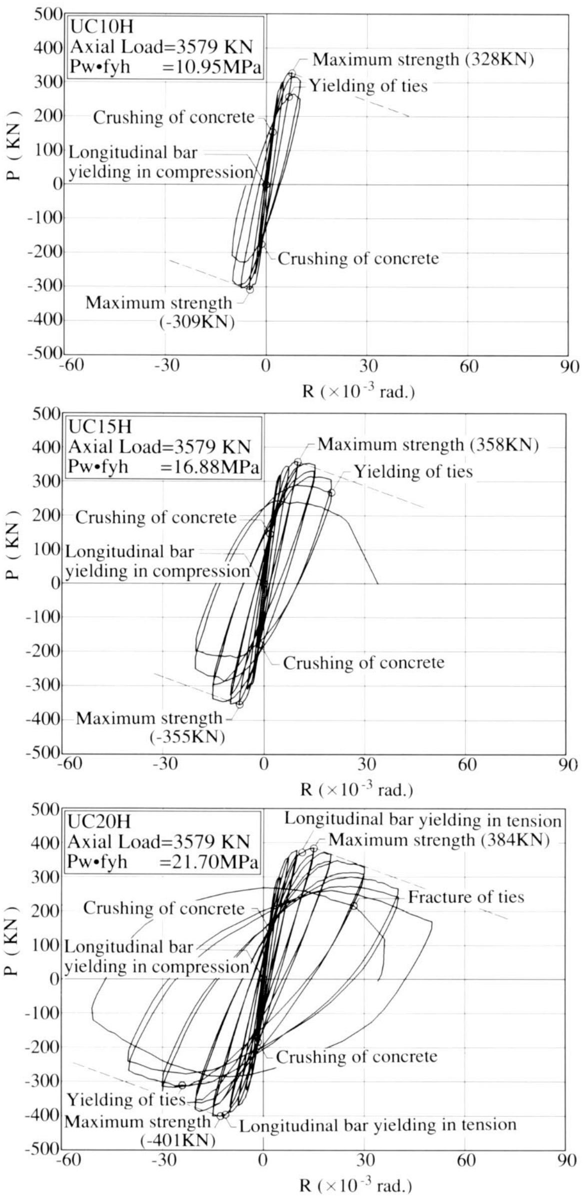
Bar Type	Bar Diameter (mm)	Yield Strength f_y (MPa)	Ultimate Strength f_u (MPa)	Modulus of Elasticity E_s (MPa)	Elongation ε_u (%)
Main Bar (D10)	10.0	393	543	2.00×10^5	19.5
Tie Bar (U5.1)	5.1	1415	1476	2.04×10^5	9.2
Tie Bar (U6.4)	6.4	1424	1515	2.18×10^5	9.5

capacity of lateral reinforcement in terms of the product of area ratio of ties (P_w) and yield strength (f_{yh}) ; 11.0, 16.9 and 21.7 MPa. The factor (P_w) is defined as As/(B· s) in which As= total area of lateral reinforcement ; B= column width and s= center to center spacing of lateral reinforcement. In specimen identification for UC15L, for example, "15" means the capacity of lateral reinforcement ($P_w \cdot f_{yh}$) of 15 MPa and "L" shows the lower axial stress level (0.35 of concrete compressive strength). Reversed cyclic horizontal load under double curvature was applied to each specimen while axial compression was held constant. Inflection point was kept at midheight of the column using loading apparatus which was designed to move top stub parallel with bottom stub. Loading was controlled by displacement angle and the amplitude was increased gradually. Loading program consisted of each one cycle at displacement angle of 0.2, 0.33, 0.5, and 0.75 % followed by each two cycles at displacement angle of 1.0, 1.5, 2.0, 3.0, 4.0 and 5.0 %.

Test results

General behavior Figure 2 shows the relationships between load and displacement and Fig.3 shows final appearances of specimens. Test results are summarized in Table.3. Two different failure modes were observed. Specimens UC15L and UC20L under lower axial compression and UC20H with largest capacity of lateral reinforcement under high axial load failed in flexture while UC10H and UC15H subjected to high axial stress showed brittle compression failure. UC15L and UC20L under lower axial stresses exhibited stable hysteresis loops at the second cycle of 5 % and deterioration of load carrying capacity was not observed until displacement angle of 8 % at which loading was terminated. Horizontal load in UC20H dropped during the second cycle of 5 % due to fracture of lateral reinforcement. Yielding of longitudinal reinforcement was not observed in UC10H and UC15H which failed in compression. They failed at their end portion by diagonal slippage of concrete together with buckling of longitudinal reinforcement and fracture of lateral reinforcement during the second cycle of 1 % and the first cycle of 2 % displacement angle, respectively.

The effect of capacity of lateral reinforcement ($P_W \cdot f_{yh}$) on load-displacement relationships in specimens under high axial load (UC10H, UC15H and UC20H) was more significant than that under lower axial load (UC15L and UC20L). Larger capacity of lateral reinforcement gave increase of maximum strength and ductility especially in specimens with high axial compression. It could be because the maximum strength was determined by yielding of longitudinal reinforcement or crushing of concrete depending on capacity of lateral reinforcement when an axial load was high. Yielding of longitudinal reinforcement was observed in UC20H while not observed in UC15H and UC10H. In specimens with lower axial compression (UC20L and UC15L), maximum strengths were almost the same values and tension bars reached yielding. UC15L showed slightly larger load drop after the maximum strength than UC20L.



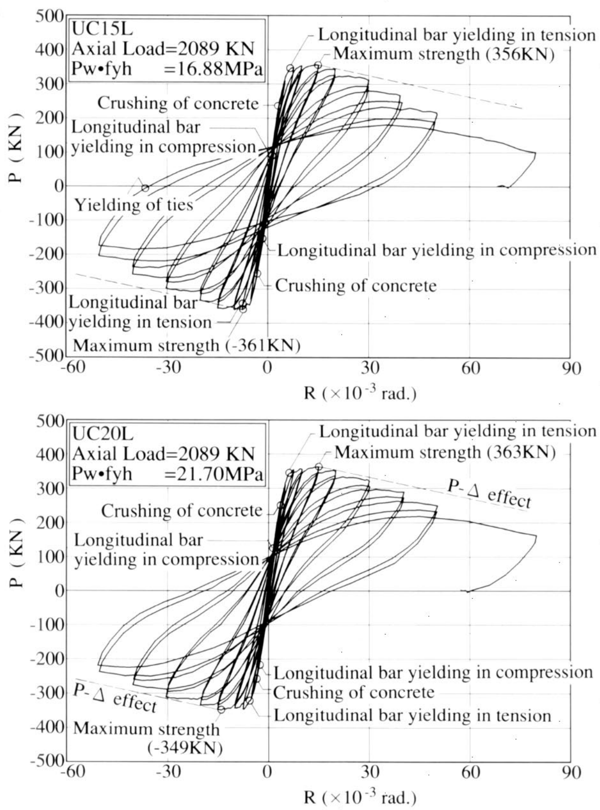
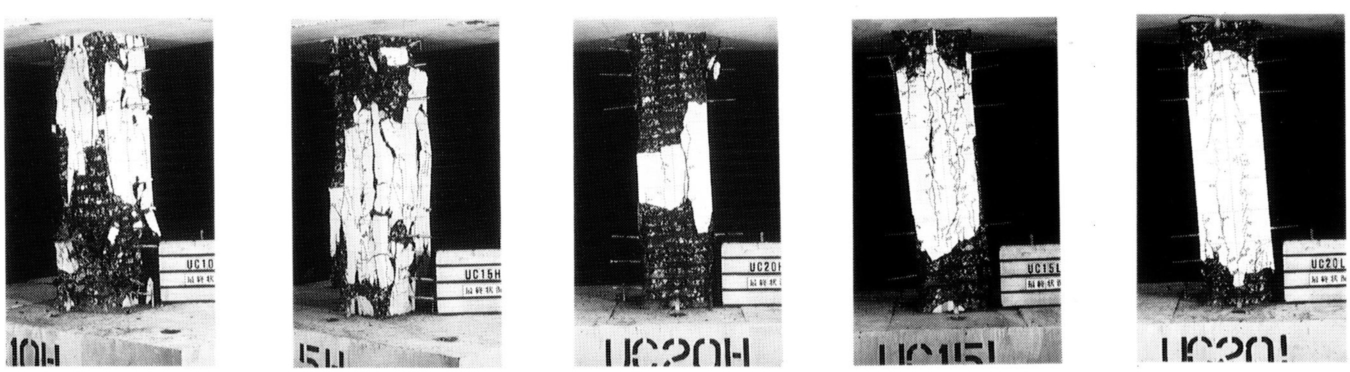


Fig. 2 Load-displacement curves



UC10H	UC15H	UC20H	UC15L	UC20L
$\sigma_0/f'_c=0.62$	$\sigma_0/f'_c=0.62$	$\sigma_0/f'_c=0.62$	$\sigma_0/f'_c=0.36$	$\sigma_0/f'_c=0.36$
$P_w \cdot f_{yh}=11.0$MPa	$P_w \cdot f_{yh}=16.9$MPa	$P_w \cdot f_{yh}=21.7$MPa	$P_w \cdot f_{yh}=16.9$MPa	$P_w \cdot f_{yh}=21.7$MPa

Fig. 3 Specimens after test

Strain of Lateral Leinforcement Strain distribution of lateral reinforcement along column height are shown in Fig.4. The specimens with larger ($P_w \cdot f_{yh}$) exhibited less strain at a given displacement angle when same axial load was applied. If ($P_w \cdot f_{yh}$) is the same value, larger axial load gave lager strain. In UC10H and UC15H failed in compression, there were little difference between strains of lateral reinforcesments in loading direction and in perpendicular direction, and strain distribution tended to be uniform from the end of column to its midheight. On the other hand, in specimens failed in flexture (UC20H, UC15L and UC2L), strain at the height ranging from 0.5D to 1.0D (D ; column depth) from the column end tended to be higher than others and the strain in loading direction seemed to be higher than that in perpendicular direction. It could be because the lateral reinforcement in loading direction carried both shear and compression forces.

Axial Strain of Column Figure 5 shows axial strain (shrinkage) of column versus displacement angle of each specimen. Figure 6 compares axial strains at given displacement angles for all specimens. Shrinkages of specimens with high axial compression progressed faster than those with lower axial compression. In comparisons between specimens with same axial load, larger ($P_w \cdot f_{yh}$) provided less amount of axial shrinkage. Axial shrinkages of UC10H, UC15H and UC20H at which the specimens failed to sustain axial load were 1 %. 2 % and 3 %, respectively. The specimens UC15L and UC20L with lower axial load exhibited axial shrinkages of less than 1 % and 0.5 %, respectively, even at a large displacement angle of 8 %.

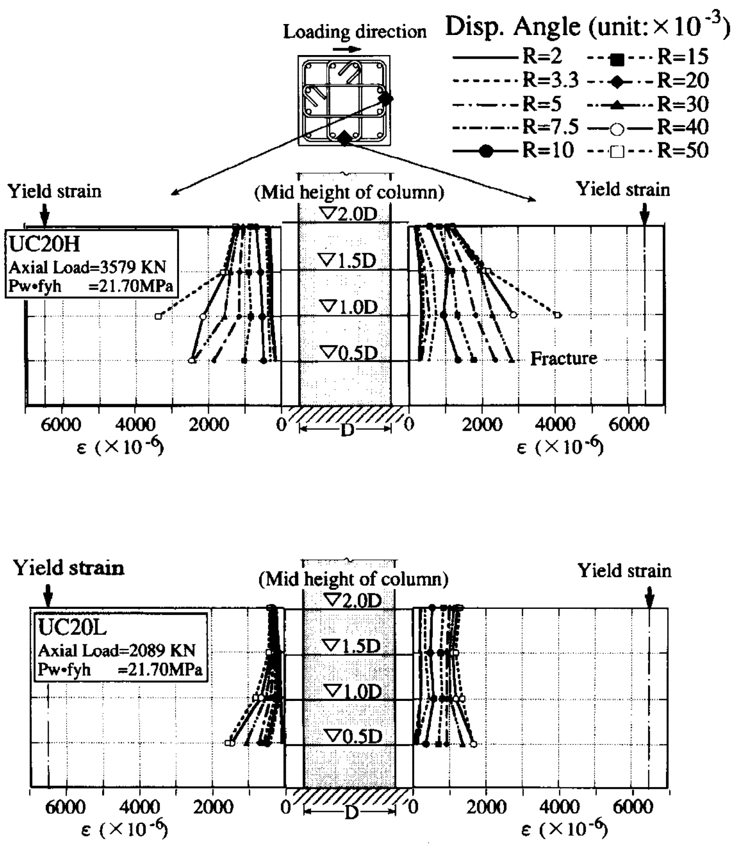


Fig. 4. Strain of lateral reinforcement

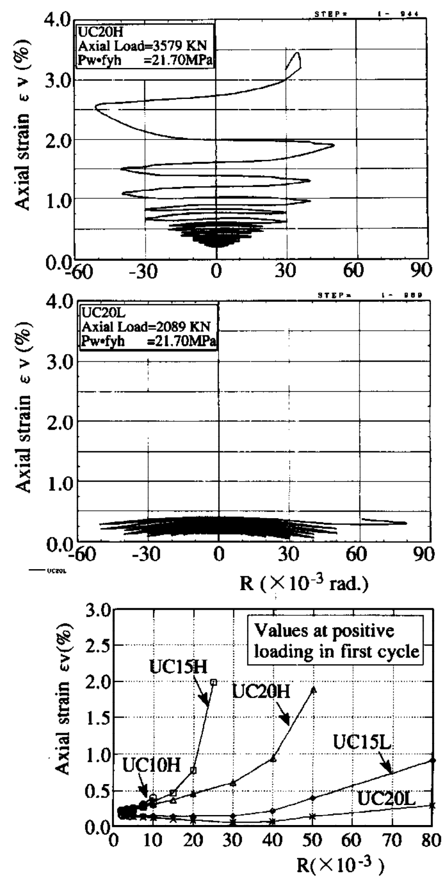


Fig.5. Axial strain

Discussion of Results

Maximum strength Comparisons of measured and calculated maximum strengths are given in Table 3. AIJ (Architectural Institute of Japan) and ACI (American Concrete Institute) equations for flexural strength were used excluding the upper limit of the concrete strength. Following remarks were drawn from the comparisons.

1) Two equations gave good agreement for the measured maximum strengths when the axial stress level was approximately 30 % of the concrete strength.
2) Evaluation of maximum strength by AIJ and ACI equations gave fairly conservative values for measured ones when axial stress level was relatively high (approximately 60 % of concrete strength).

The ratios of measured values to calculated ones by AIJ and ACI equations were from 1.31 to 1.74.

Displacement ductility To evaluate displacement ductility, the ultimate displacement (Ru) which was defined as the displacement angle at which 80 % of the maximum strength was sustained in load versus displacement

Table 3 Test results

Specimen	Ultimate displacement Ru (1/1000rad)	Failure Mode	Maximum Strength (KN)				
			Measured	Calculated		Measured / Calculated	
			P_{max}	Q_{AIJ}	Q_{ACI}	P_{max} / Q_{AIJ}	P_{max} / Q_{ACI}
UC10H	10.0	C	354	257	283	1.38	1.25
UC15H	20.0	C	394	257	283	1.52	1.39
UC20H	30.0	F	438	257	283	1.71	1.55
UC15L	30.0	F	386	377	346	1.02	1.12
UC20L	40.0	F	394	377	346	1.05	1.14

Pw : Area ratio of ties　　fyh : Yield strength of ties

σo : Axial stress　　f'c : Compressive strength of concrete cylinder

C(Compression failure) : Brittle failure due to crushing of concrete at whole cross section with buckling of longitudinal bars.

F(Flexural failure) : Maximum strength is determined by yielding of tension steel.

FC(Flexural compression failure) : Maximum strength is determined by crushing of concrete after yielding of tension steel.

Pmax*: Value excluded P-Δ effect

QACI : Strength by ACI equation

QAIJ : Strength by AIJ equation

$Q_{AIJ}=2M_u/h_o$

When $N_{max} \geqq N > N_b$

$$M_u=[0.5a_g \cdot f_y \cdot g_1 \cdot D+0.024(1+g_1)(3.8-g_1)b \cdot D^2 \cdot f'_c]\times\left(\frac{N_{max}-N}{N_{max}-N_b}\right)$$

When $N_b \geqq N \geqq 0$

$$M_u=0.5a_g \cdot f_y \cdot g_1 \cdot D+0.5N \cdot D\left(1-\frac{N}{b \cdot D \cdot f'_c}\right)$$

When $0 > N \geqq N_{min}$

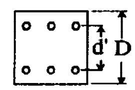
$$M_u=0.5a_g \cdot f_y \cdot g_1 \cdot D+0.5N \cdot g_1 \cdot D$$

$N_{max}=b \cdot D \cdot f'_c+a_g \cdot f_y$

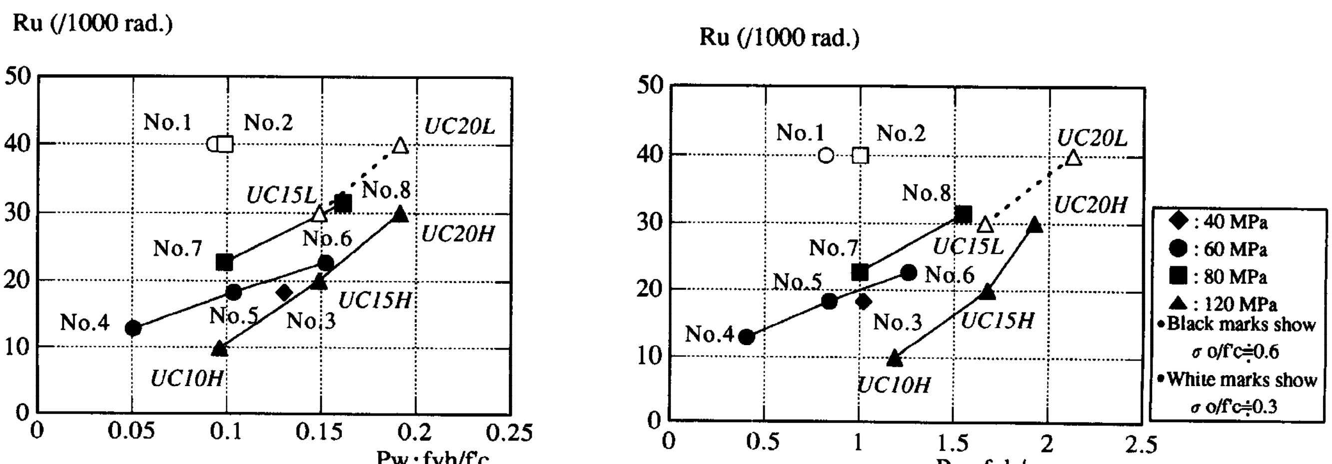
$N_{min}=-a_g \cdot f_y$

$N_b=0.22(1+g_1)b \cdot D \cdot f'_c$

ho: Column height (cm)
N : Axial force (kg)
at : Total area of tension reinforcement (cm²)
ag : Total area of longitudinal reinforcement (cm²)
b : Column width (cm)
D : Column depth (cm)
fy : Yield strength of reinforcing bar (kg/cm²)
f'c: Concrete compressive strength (kg/cm²)
g1 : Ratio of distance between center of gravity of tension steel and compression steel to column depth



$g_1=d'/D$



(a) (Ru) – ($P_w \cdot f_{yh} / f'_c$) relationship　　(b) (Ru) – ($P_w \cdot f_{yh} / \tau_{max}$) relationship

Fig. 6 Ultimate displacement and capacity of lateral reinforcement

angle curve included P-Δ effect was used. Figure 7(a) shows the relationship between ultimate displacement angle (Ru) and capacity of lateral reinforcement normalized by concrete strength ($P_w \cdot f_{yh} / f'_c$). The (Ru) increased with ($P_w \cdot f_{yh} / f'_c$) in specimens with same concrete strength and same axial stress level (σ_0 / f_c). The specimens with same ($P_w \cdot f_{yh} / f'_c$) tend to have less (Ru) when concrete strength and axial stress level (σ_0 / f_c) increased. The 120 MPa concrete specimens showed less (Ru) than the specimens with 60 and 80 MPa concrete when the axial stress level (σ_0 / f_c) and ($P_w \cdot f_{yh} / f'_c$) were same.

Figure 7(b) shows the relationship between ultimate displacement angle (Ru) and capacity of lateral reinforcement normalized by measured maximum nominal shear stress ($P_w \cdot f_{yh} / \tau_{max}$). When the ($P_w \cdot f_{yh} / \tau_{max}$)

was around 1.0, the specimens with 60 and 80 MPa concrete and with axial stress level (σ_0 / f_c) of 0.3 showed large (Ru) of 4 %. The specimen with 120 MPa concrete and with σ_0 / f_c =0.3 needed twice as much ($P_w \cdot f_{yh} / \tau_{max}$) as comparable specimens with 60 and 80 MPa concrete to develop a (Ru) of 4 %. When the ($P_w \cdot f_{yh} / \tau_{max}$) was around 1.0 and the (σ_0 / f_c) was equal to 0.6, the specimens with concrete strength ranging from 40 to 80 MPa showed a (Ru) of 2 %, however, those with 120 MPa concrete exhibited a (Ru) of 1 %.

EVALUATION OF DISPLACEMENT DUCTILITY USING AVAILABLE TEST DATA

Outline of Data Used

To assess the effects of various parameters associated with displacement ductility of columns, regression analysis was performed using available experimental column data including those presented in this paper. Data of specimens with square cross section lager than 200 x 200 mm and tested under reversed cyclic loading were used. Lateral reinforcement included ties of normal strength rebar with 135° hooks, normal strength welded wire fabrics, high strength closed type ties by butt-welding and ultra-high-strength spiral hoops. In most cases, all the longitudinal reinforcements were confined with ties and subties. Total number of the data was 118, concrete strength ranged from 24 to 130 MPa (number of specimen with concrete strength higher than 40 MPa was 108) and shear span to depth ratio iwas 1.0 through 2.5.

Previously defined ultimate displacement (Ru) was used in this analysis. Employed principle parameters associated with the (Ru) were 1) compressive strength of concrete (f'_c), 2) ratio of axial stress to concrete compressive strength (σ_0 / f_c), 3) shear span to depth ratio (a/D), 4) measured maximum shear stress ($\tau_{max} = Q_{max}$ / Bj : B=column width, j=7/8d, d = effective depth), 5) capacity of lateral reinforcement ($P_w \cdot f_{yh}$), 6) existence of sub-ties and 7) failure mode.

Discussions and Results

Parameters 1) through 4) above did not show clear correlation with (Ru) independently. On the other hand, capacity of lateral reinforcement ($P_w \cdot f_{yh}$) exhibited clear positive correlation with (Ru). Figures 8 and 9 show the (Ru) as functions of ($P_w \cdot f_{yh}$) and ($P_w \cdot f_{yh}$) / (f'_c). Linear least squares fit lines and correlation coefficients (R) are given in the figures. The correlation coefficient (R) increased from 0.49 to 0.60 when ($P_w \cdot f_{yh}$) was normalized by compressive strength of concrete (f'_c), which means ($P_w \cdot f_{yh}$)/(f'_c) has stronger correlation with (Ru) than ($P_w \cdot f_{yh}$). The ($P_w \cdot f_{yh}$)/(f'_c) which eliminate the effect of concrete strength was employed for primary factor of the analysis. In the relationship between (Ru) and ($P_w \cdot f_{yh}$)/(f'_c), lower bound of (Ru) can be given by following equation (1). Regardless of failure mode and existence of sub-tie, all measured values of (Ru) are larger than calculated ones by equation (1) except for one data.

$$Ru = 0.1(P_w \cdot f_{yh})/(f'_c) \qquad (1)$$

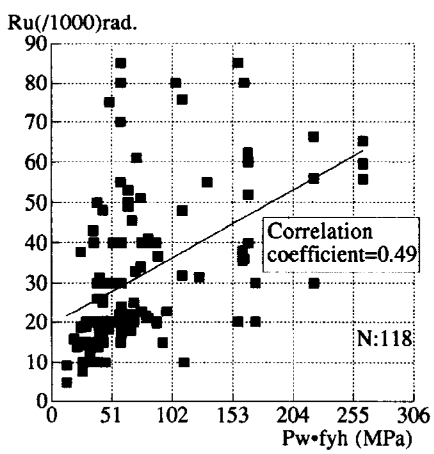


Fig.7 : (Ru) - ($P_w \cdot f_{yh}$) relationship

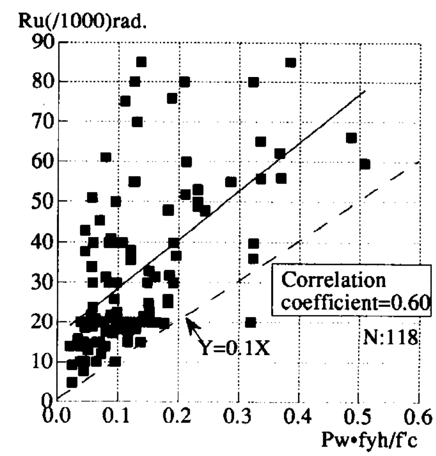


Fig.8 : (Ru) - ($P_w \cdot f_{yh} / f'_c$) relationship

Relationship between (Ru) and $(P_w \cdot f_{yh})/(f'_c)$ was classified by failure mode (shear failure and flexural failure). The data numbers of shear failure and flexural failure are 19 and 99, respectively. The (Ru) for almost all the data of shear failure are equal to or less than 2 % and the correlation between (Ru) and $(P_w \cdot f_{yh})/(f'_c)$ was not so strong as shown in Fig.10. The data of which failed in shear were omitted from the analysis. The rest of the data (99 data) was classified by the cases with and without sub-tie. Data numbers with and without sub-ties are 94 and 5, respectively.

The 94 specimens with sub-ties failed in flexture is classified by axial stress to concrete strength ratio (σ_0 / f_c) as shown in Figs.9~11. Linear least squares fit lines and correlation coefficients (R) are given in the figures. The fit line equations for given axial stress levels (σ_0 / f_c) were as follows.

$$Ru = 0.160(P_w \cdot f_{yh})/(f'_c)+0.020 \quad (\text{for } \sigma_0 / f'_c < 0.4) \tag{2}$$

$$Ru = 0.149(P_w \cdot f_{yh})/(f'_c)+0.010 \quad (\text{for } 0.4 \leq \sigma_0 / f'_c < 0.6) \tag{3}$$

$$Ru = 0.127(P_w \cdot f_{yh})/(f'_c)+0.006 \quad (\text{for } 0.6 \leq \sigma_0 / f'_c) \tag{4}$$

The inclination and the constant value of the fit line equations decrease as (σ_0 / f_c) increases, which means that the displacement ductility decreases as axial stress level (σ_0 / f_c) increases when $(P_w \cdot f_{yh}) / (f'_c)$ is the same value. The correlation coefficient (R) increase as (σ_0 / f_c) increases. Classification of relationship between (Ru) and $(P_w \cdot f_{yh}) / (f'_c)$ by concrete strength and shear span to depth ratio did not give a clear correlation. Based on the study, multiple linear regression analysis of the ultimate displacement (Ru) on $(P_w \cdot f_{yh}) / (f'_c)$ and axial stress level (σ_0 / f_c).was carried out. The 94 data of the specimens with sub-ties failed in flexture were used. The following equation (5) on the (Ru) was acquired. The correlation coefficient (R) of the measured and calculated (Ru) by equation (5) is 0.70.

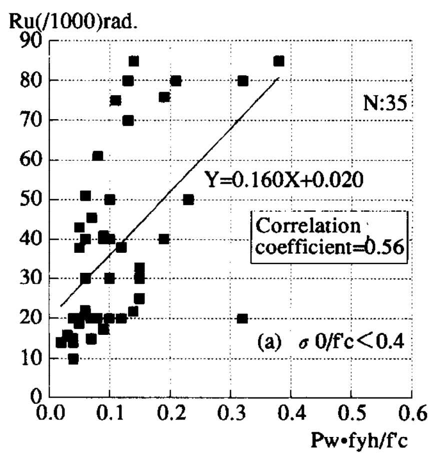


Fig.9 : (Ru) – $(P_w \cdot f_{yh} / f'_c)$ relationships (1)

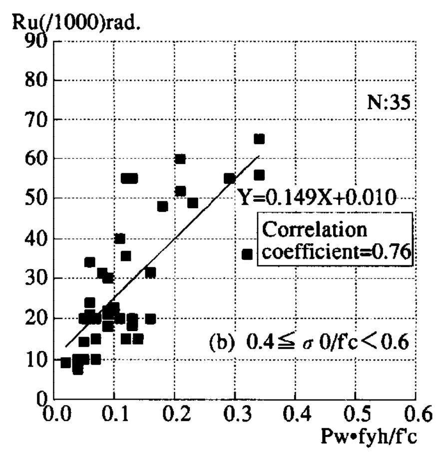


Fig.10 : (Ru) – $(P_w \cdot f_{yh} / f'_c)$ relationships (2)

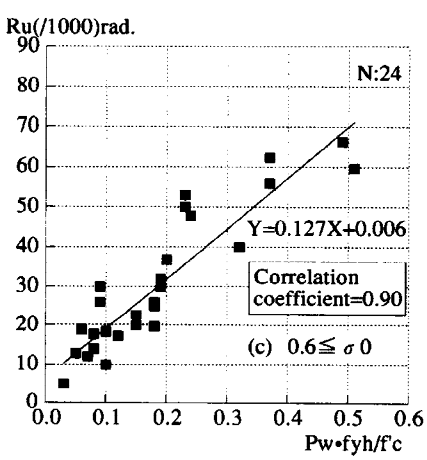


Fig.11 : (Ru) – $(P_w \cdot f_{yh} / f'_c)$ relationships (3)

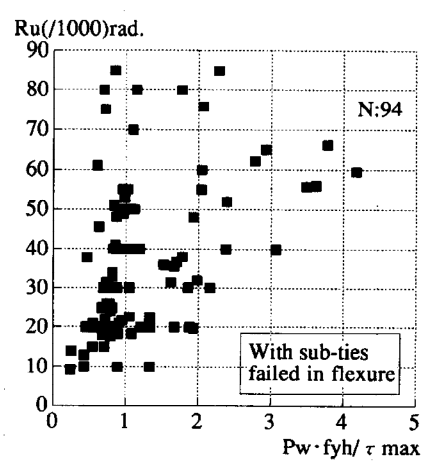


Fig.12 : (Ru) – $(P_w \cdot f_{yh} / \tau'_{max})$ relationships

$$Ru = 0.127 (P_w \cdot f_{yh}) / (f_c') - 0.052(\sigma_0 / f_c) + 0.041 \tag{5}$$

The relationship between (Ru) and ($P_w \cdot f_{yh}$) normalized by measured maximum nominal shear stress (τ_{max}) which meant the redundancy of the lateral reinforcement for shear was also examined. The relationship between (Ru) and ($P_w \cdot f_{yh}$) / (τ_{max}) for the 94 specimens with sub-ties failed in flexture is shown in Fig.12. The figure shows that the specimens with ($P_w \cdot f_{yh}$) / (τ_{max}) equal to or larger than 1.0 develop a (Ru) equal to or larger than 2 % displacement angle.

CONCLUSION

1) Ductility of columns of ultra-high-strength concrete was strongly affected by both the level of axial compression and the capacity of lateral reinforcement.

2) The brittle ultra-high strength concrete could still be well confined by using high-or ultra-high-strength lateral reinforcement, however, relatively more capacity of lateral reinforcement was required to provide sufficient ductility than the case with lower strength concrete.

3)The capacity of lateral reinforcement normalized by concrete strength was appropriate index to evaluate the ductility. Based on the regression analysis using all the available column data of high strength concrete, empirical equations to evaluate the displacement ductility were proposed.

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