Seismic Performance of High-Strength Concrete Columns

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ABSTRACT

This experimental investigation was conducted to examine the behaviour of eight one-third scale columns made of high-strength concrete (HSC). The columns were subjected to a constant axial load corresponding to 30 per cent of the column axial load capacity and a cyclic horizontal load-inducing reversed bending moment. The variables studied in this research are the volumetric ratio of transverse reinforcement, tie configuration and tie yield strength. Columns with 42 per cent higher amounts of transverse reinforcement than that required by seismic provisions of ACI 318-02 showed ductile behaviour. Relationships between the calculated damage index and the observed damage such as initial crack, spalling of concrete, buckling of longitudinal bar, and crushing of concrete are propose

1. Introduction

Sakai and Sheikh[1)] have summarized major research conducted on the subject of confinement of concrete columns constructed using NSC. However, information on the ductility of HSC columns has been limited[2-3)], with most of the available information being based on experimental testing of small-scale columns subjected to concentric axial loads only. ACI-ASCE Committee 441[4)]pointed out that columns subjected to axial loads of less than 20 per cent of column axial-load capacity exhibited a good level of ductility when confined according to current ACI confinement requirements. The scientific community has not yet reached a consensus on required confinement reinforcement for ductile HSC columns.

This experimental investigation was conducted to examine theseismic performance of eight one-third scale HSC columns. The columns were subjected to a constant axial load corresponding to 30 per cent of the column axial-load capacity and a cyclic horizontal load-inducing reversed bending moment. The variables studied in this research are the volumetric ratio of transverse reinforcement, tie configuration, and tie yield strength.

2. Experimental program

The dimensions and steel-bar-reinforcement layout of the reinforced concrete column are shown in Fig. 1. The core size measured from the centre of the perimeter tie was kept constant at 174×174 mm for all specimens. Table 1 includes details of the test

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specimens. Ready-mix normal-weight concrete with an average slump of 210 mm was used. The 7-day strength of concrete was about 75 per cent of the 28-day concrete strength, and afterwards, the 28-day concrete strength increased by about10 per cent in the following six months. Two different types of reinforcing steel were used to construct specimens. Important properties of steel are also listed in Table 1. Several electrical strain gauges were placed in the specimens on both the longitudinal and transverse bars (Fig. 1). Test setup and lateral load sequence are shown schematically in Fig. 2 and 3.

Table 1 Properties of Specimens

Specimen	Transverse reinforcement							Longitudinal Bar			f_{ck} (MPa)	$\frac{P}{f_{ck} \cdot A_g}$	Set
	Bar	S (mm)	Detail	ρ_s (%)	$\frac{\rho_s}{\rho_{s(ACI)}}$	f_{yh} (MPa)	$\frac{\rho_s \cdot f_y}{f_{ck}}$	Bar	f_{yh} (MPa)	ρ_l (%)			
C-S	Ψ6	57	C	1.58	1.00	779.10	17.94	8-D13	430.71	2.54	68.60	0.3	S-series
D-S	Ψ6	65	D	1.58	1.00	779.10	17.94	8-D13	430.71	2.54	68.60	0.3	
H-S	Ψ6	38	H	1.58	1.00	779.10	17.94	8-D13	430.71	2.54	68.60	0.3	
C-A	Ψ6	40	C	2.25	1.42	779.10	22.55	8-D13	430.71	2.54	68.60	0.3	A-series
D-A	Ψ6	46	D	2.25	1.42	779.10	22.55	8-D13	430.71	2.54	68.60	0.3	
H-A	Ψ6	27	H	2.25	1.42	779.10	22.55	8-D13	430.71	2.54	68.60	0.3	
L-C-S	Ψ6	40	C	2.25	1.00	548.80	18.00	8-D13	430.71	2.54	68.60	0.3	L-series
L-D-S	Ψ6	46	D	2.25	1.00	548.80	18.00	8-D13	430.71	2.54	68.60	0.3	

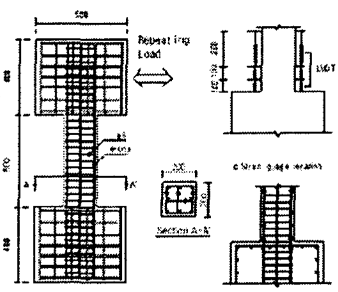


Fig. 1 Detail of specimens and gaging arrangements(unit:mm)

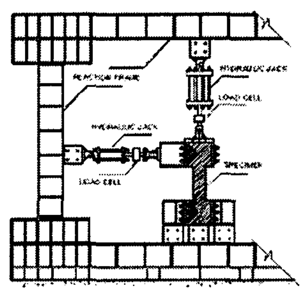


Fig. 2 Test setup and loading condition

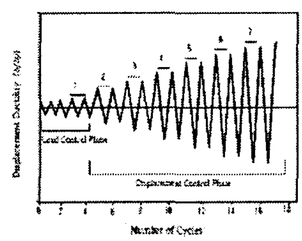


Fig. 3 Lateral displacement sequence

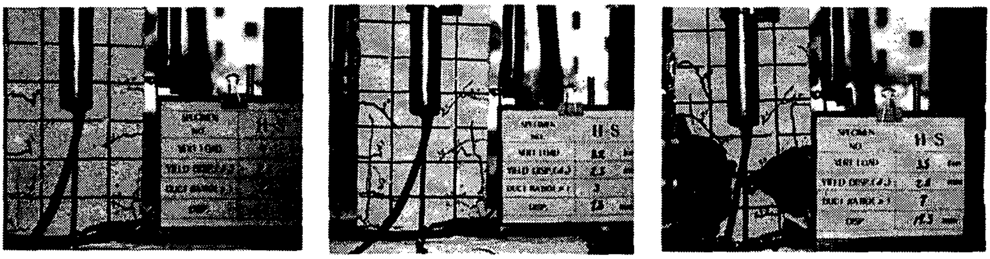
3. Test Results

3.1Test observations

As lateral force increased, flexural cracking spread to 50 per cent of the distance from the critical sections between the bottom end and the lateral loading point. Afterward, the longitudinal bars yielded in tension at a displacement ductility of μ_Δ = 1, and diagonal shear cracks occurred. Incipient spalling of concrete developed in the plastic hinge region at a displacement ductility of μ_Δ = 3. Spalling of cover extended as displacement increased. The failure mode for all specimens was dominated by flexural effects. The final appearances of Column H-S is shown in Fig 4.

3.2 Hysteretic loops

Lateral force-displacement hysteresis loops for Column H-A and H-S are shown in



(a) after 2 cycles to μ_Δ = 2 (b) after 2 cycles to μ_Δ = 3 (c) at the end of testing

Fig 4 Column H-S at each stage of testing

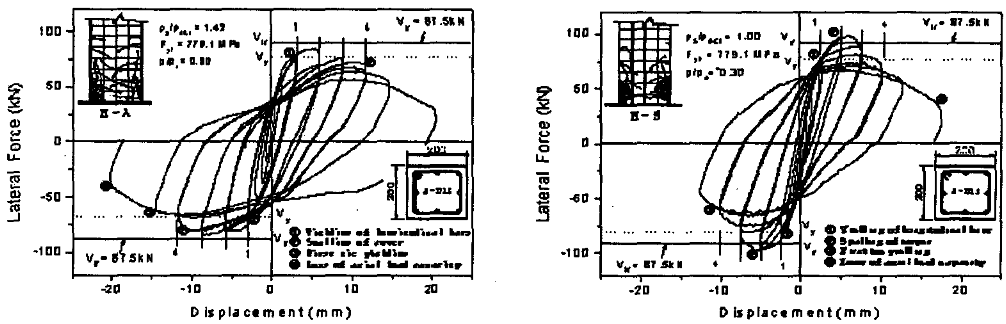


Fig 5 Column H-S at each stage of testing

Fig. 5; this includes lateral force V_{if} at ideal flexural strength based on ACI 318-02 provisions, assuming rectangular concrete stress blocks, and shear force V_y at final yield of longitudinal reinforcement.

3.3 Ductility factor and energy dissipation

In seismic design, the inelastic deformation is generally quantified by ductility parameters and by energy dissipation capacity. The energy dissipation capacity also accounts for the history of loadings in addition to the maximum displacement attained. Both types of indicators are computed in this paper to compare the column behaviour on a rational basis. Table 4 provides the values of $\mu_{\Delta u}$, $\mu_{\phi u}$, and E_N for each column.

Table 2 Comparisons of ductility, energy dissipation, and damage index

Specimen	Ductility ratio		Energy dissipation	Initial crack		Spalling of cover		Shear crack		Cruching concrete		Failure	
	$\mu_{\Delta u}$	$\mu_{\phi u}$	E_N	μ_Δ	Damage Index	μ_Δ	Damage Index	μ_Δ	Damage Index	μ_Δ	Damage Index	μ_Δ	Damage Index
C-S	3.25	9.75	7.8	1	0.020	3	0.046	3	0.050	4	0.062	5	0.077
D-S	3.16	9.00	7.7	1	0.026	3	0.057	3	0.058	4	0.064	5	0.103
H-S	3.55	14.40	7.9	1	0.022	3	0.048	3	0.048	4	0.065	5	0.081
C-A	3.69	15.00	9.5	1	0.020	3	0.040	3	0.044	4	0.062	5	0.68
D-A	4.38	17.80	9.6	1	0.025	3	0.042	3	0.052	4	0.052	5	0.70
H-A	4.85	19.80	9.7	1	0.030	3	0.052	3	0.055	4	0.062	5	0.75
L-C-S	3.64	14.70	10.0	1	0.028	3	0.043	3	0.043	4	0.063	5	0.65
L-D-S	3.70	15.00	11.4	1	0.023	3	0.045	3	0.050	4	0.055	5	0.069

3.4 Strain distribution

Figures 6 shows the typical strain distribution for two test columns at different displacement ductility ratios. The yield strain of transverse reinforcement was approximately 2,100 microstrains. As noted in Fig 6, for specimen H-A, which used higher-yield-strength steel for transverse reinforcement, the yield strain in ties was reached at relatively high displacement (11.55 mm). However, when lower-yield-strength steel for transverse reinforcement was used (see Column L-C-S), yielding was observed at a horizontal low displacement of 7.91 mm.

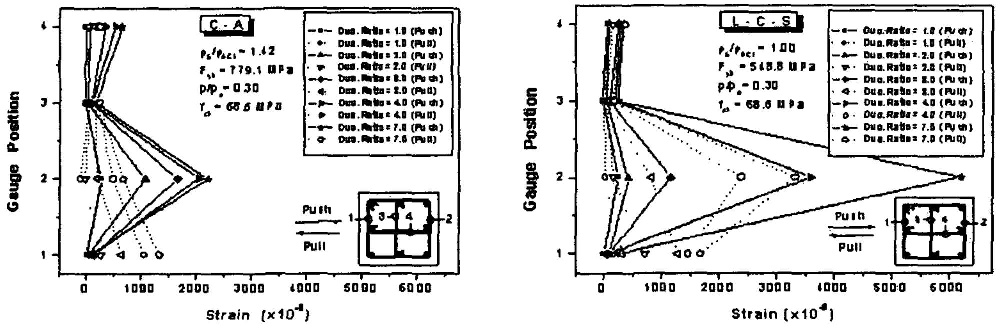


Fig. 6 Strain in transverse reinforcement in critical region

3.5 Damage index

The index used in this study is a modified form of the Park and Ang index, as given by Kunnath et al.[5] Table 2 shows the calculated damage index and the observed damage.

4. Conclusions

(1) Specimens made of high-strength concrete with a nominal strength of 68.6 MPa, designed using seismic provisions of ACI 318-99, possess adequate curvature and displacement ductility under 30 per cent of the column's axial load capacity.

(2)For axial load levels below 30 per cet of the column's axial capacity, increasing the yield strength of transverse reinforcement would have little influence on seismic performance.

(3) For specimens with the same volumetric ratio of transverse reinforcement, specimens with type-H ties display more ductile behaviour than specimens with type-C or type-D ties.

(4) The damage index at first flexural crack, at the time of spalling of cover concrete, and at crushing of core concrete was 0.013 to 0.030, 0.045 to 0.58, and 0.065 to 0.103, respectively.

References

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