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CONFINEMENT STEEL REQUIREMENTS FOR HIGH STRENGTH CONCRETE COLUMNS

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ABSTRACT

This paper presents results from a comprehensive research program which aims to study various aspects of concrete confinement and its effects on the seismic performance reinforced concrete columns. The current work focuses upon the experimental behavior of reinforced concrete columns made of high strength concrete (HSC) confined by rectilinear ties and subjected to cyclic flexure and shear and moderate to high axial load levels. Each specimen consisted of a 305x305x1473 mm column and a 508x762x813 mm stub. Unconfined concrete strength varied from 72 MPa to 102 MPa, for the work presented here. The concrete strength, steel configuration, axial load level, amount of lateral steel, presence of a heavy stub are the main variables studied. Confinement provisions of current design codes are critically examined in the light of the test results. A recently developed design procedure is checked against the current test data.

KEYWORDS

High strength concrete; concrete confinement; ductility; deformability; columns; strength; plastic hinge

INTRODUCTION

The response of framed concrete structures designed according to the current seismic design philosophy, when subjected to strong ground motion, is not expected to be elastic. Allowing some inelastic deformations to take place and using a reduced base shear force, rather than the base shear corresponding to elastic response has been preferred. Hence formation of plastic hinges while the structure is experiencing large displacement excursions is unavoidable. Most building codes (ACI 318-89, CSA A23.3-94) attempt to ensure beam hinging rather than column hinging in order to ensure stability as well as the vertical load carrying capacity during strong ground motions. However, recent earthquakes and analytical investigations (Pauley 1977, Bayrak 1995) showed that column hinging is still possible during strong ground motions despite the application of "strong column-weak beam" concept, as recommended by aforementioned design codes. As a result design of ductile concrete columns is still an unresolved issue. Tests on NSC confinement have recently been reported (Sheikh and Uzumeri 1980, Sheikh and Khoury 1993) extensively but data on HSC columns is very limited in particular for tests in which large-size specimens are subjected to large inelastic cyclic lateral displacement excursions under high axial load levels. Results from five such tests are reported here. A comparison of current test results with those from earlier studies, which is particularly useful in evaluating the effect of concrete strength, is also provided. Applicability of a recently developed procedure (Sheikh 1995) for the design of confinement steel in concrete with strength up to 55 MPa is also evaluated for high strength concrete.

EXPERIMENTAL PROGRAM

Results from column specimens made of HSC (72 MPa-102 MPa) are presented here and are compared with those from earlier tests (Sheikh and Khoury 1993; Sheikh, Shah, and Khoury 1994) on similar specimens having concrete strengths between 31 MPa and 55 MPa. Each specimen consisted of a 305x305x1473 mm column and a 508x762x813 mm stub. The column represented the part of a column in a regular building frame between the section of maximum moment and the point of contraflexure. The stub represented a discontinuity such as a beam column joint or a footing. The core size measured from the center of the perimeter tie is kept constant at 267x267 mm for all the specimens, giving a core area equal to 77% of the gross area of the column. Table 1 includes details of the test specimens. Each specimen contained 8-20M longitudinal bars having a yield strength of 454 MPa. The volumetric ratio of rectilinear ties to concrete core, measured center-to-center of perimeter ties, ranged from 2.84% to 6.74%.

Table 1. Specimen Details and Section & Member Ductility Parameters

	Specimen		ES-1HT	AS-2HT	AS-3HT	AS-5HT	AS-6HT
	f_c' (MPa)		72.1	71.7	71.8	101.8	101.9
Lateral Steel	size @ spac. (mm)		15M@95	10M@90	10M@90	10M&15M@90	15M@76
	ρ_s (%)		3.15	2.84	2.84	4.02	6.74
	f_{yh} (MPa)		463.0	542.0	542.0	542.0&463.0	463.0
	$A_{sh}/A_{sh(ACI)}$		1.13	1.19	1.19	1.09	1.70
Axial Load	$P/f_c' A_g$		0.50	0.36	0.50	0.45	0.46
	P/P_o		0.50	0.36	0.50	0.48	0.49
	$R_{A/P}$		2.3	3.3	2.4	2.3	3.5
Ductility Factors	μ_Δ @	$0.8\,P_{max}$	4.6	6.2	5.0	4.0	6.3
	μ_ϕ @	$0.8\,M_{max}$	6.6	15.8	10.1	9.6	14.0
		$0.9\,M_{max}$	5.9	13.6	9.1	5.6	10.4
Ductility Ratios	$N_{\Delta 80}$		15	18	15	14	23
	$N_{\Delta t}$		20	61	28	35	68
	$N_{\phi 80}$		19	53	20	27	54
	$N_{\phi t}$		25	113	42	49	114
Energy Indicators	W_{80}		33	41	36	23	49
	W_t		57	313	102	98	230
	E_{80}		80	631	161	144	450
	E_t		105	1412	396	311	1243

Each specimen was tested under constant axial load and reversed cyclic lateral displacement excursions until it was not able to maintain the axial load. The lateral load was applied at the stub near the stub-column interface (Figure 1). Hence the column test region near the stub was subjected to constant axial force and cyclic shear and moment. In the first cycle the specimen was subjected to 75% of the elastic or yield displacement (Δ_1), which can be defined as the lateral deflection corresponding to the estimated lateral load carrying capacity (V_{max}) on a straight line joining origin and a point about 65% of V_{max} on the lateral load-displacement curve. It should be realized that, both Δ_1 and V_{max} were calculated using the theoretical sectional response of the unconfined column and integrating curvatures along the length of the specimen. Subsequent displacement excursions consisted of two cycles each at Δ_1, $2\,\Delta_1$, $3\,\Delta_1$, $4\,\Delta_1$ and so on.

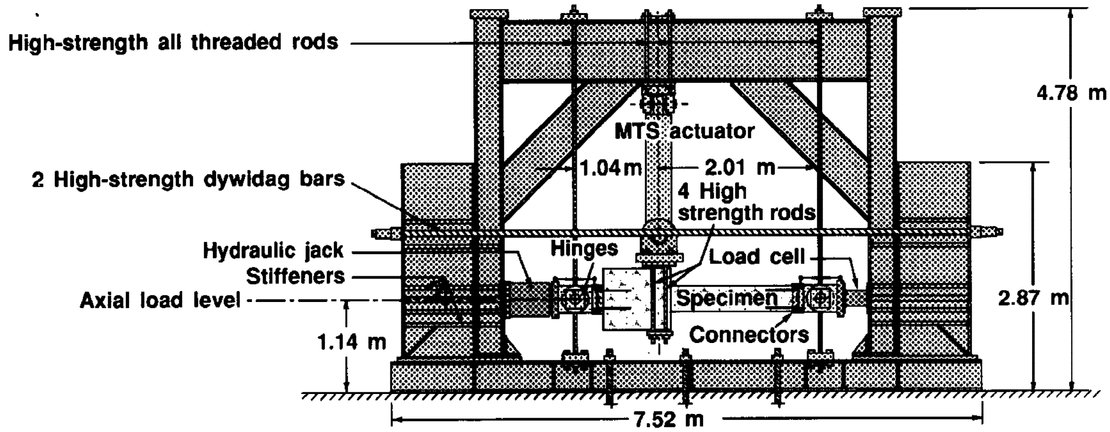


Figure 1. Schematic of Test Setup

RESULTS

Results are presented graphically in the form of column shear force-versus-tip deflection and moment-versus-curvature relationships of the failed sections. Figure 2 illustrates the idealization of a specimen and the definitions of shear force V, and tip deflection Δ_t used in Figures 3 to 7. It should be recognized that the deflection at the failed section was determined from the measured deflected shape of the column, and was also used to calculate secondary moment caused by the axial load. Curvatures were calculated from the deformation readings measured by the upper and lower LVDTs located in the extensively damaged region within the hinging zone. Furthermore, Figure 2 illustrates the definitions of section and member ductility parameters used to evaluate the performances of the specimens. Using these parameters the effects of concrete strength, axial load and steel configuration on the behavior HSC columns are evaluated.

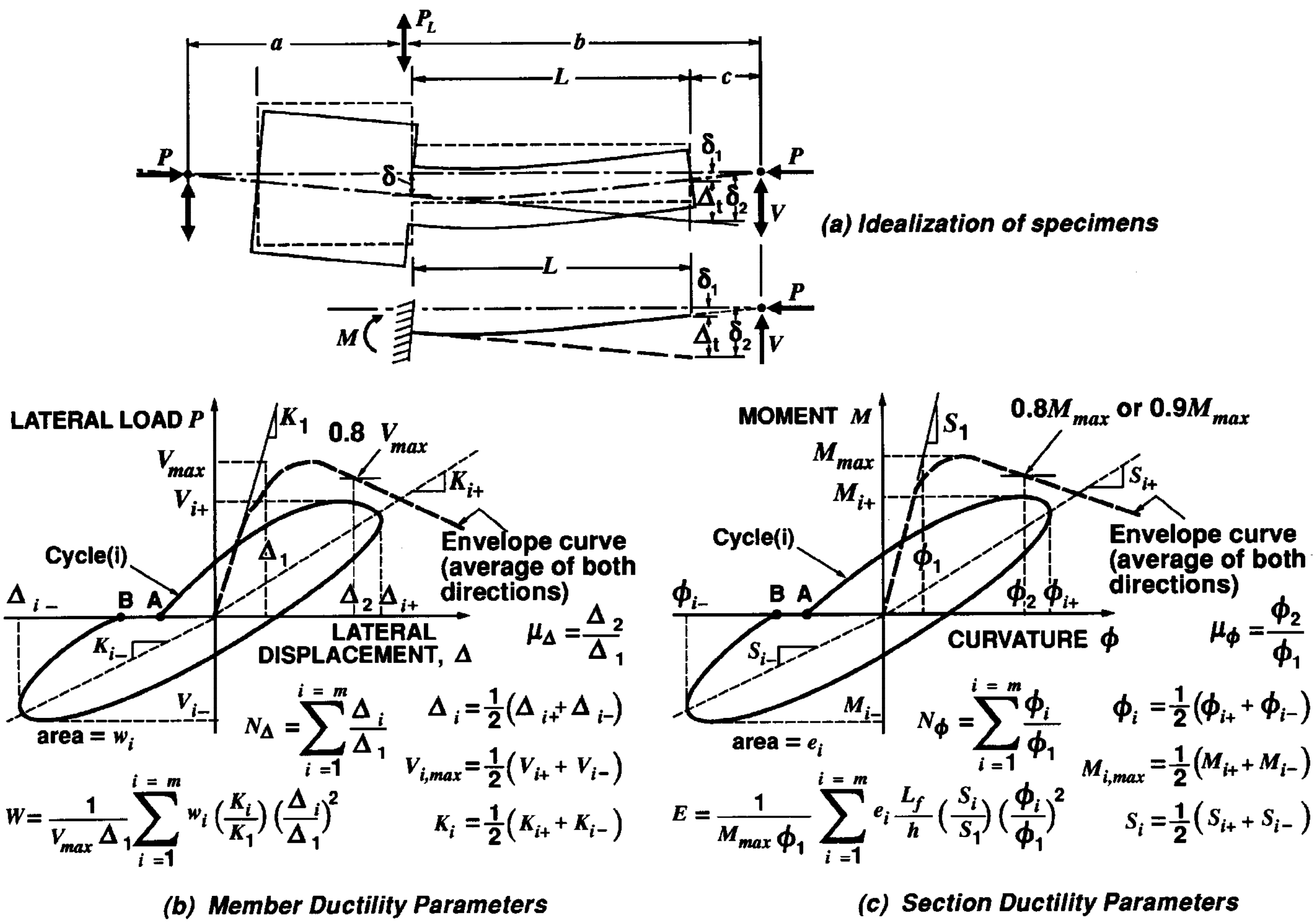
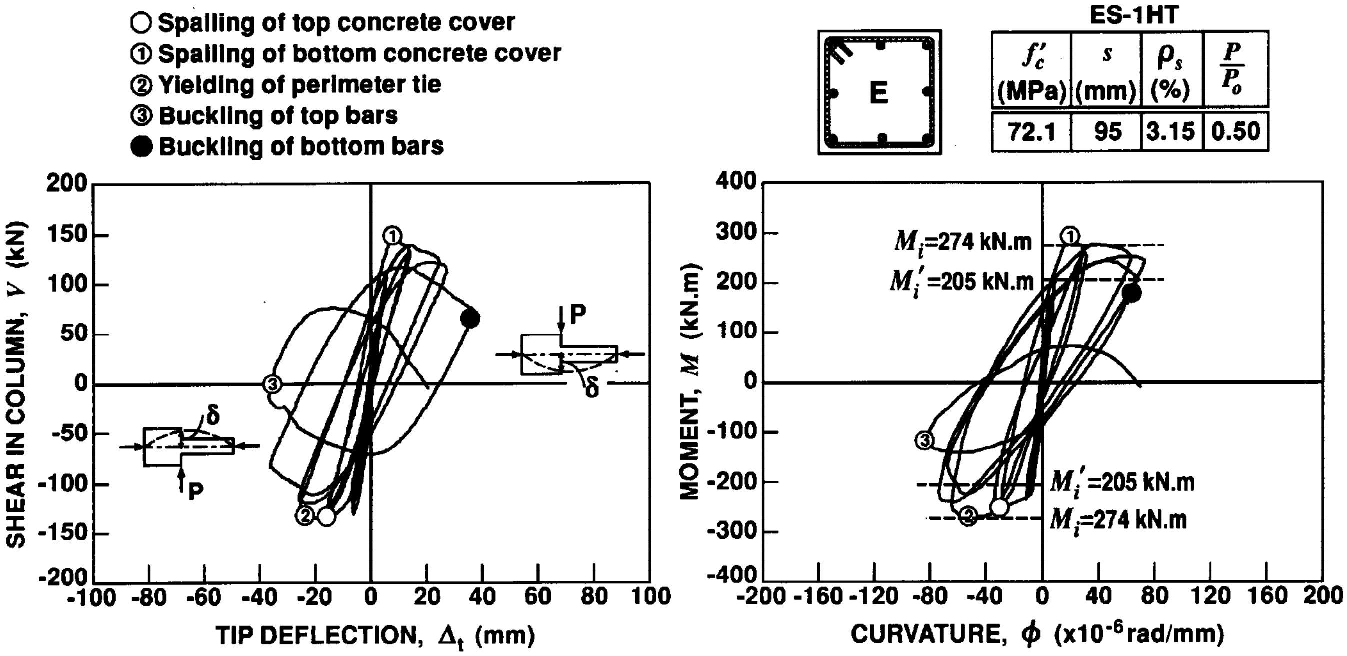


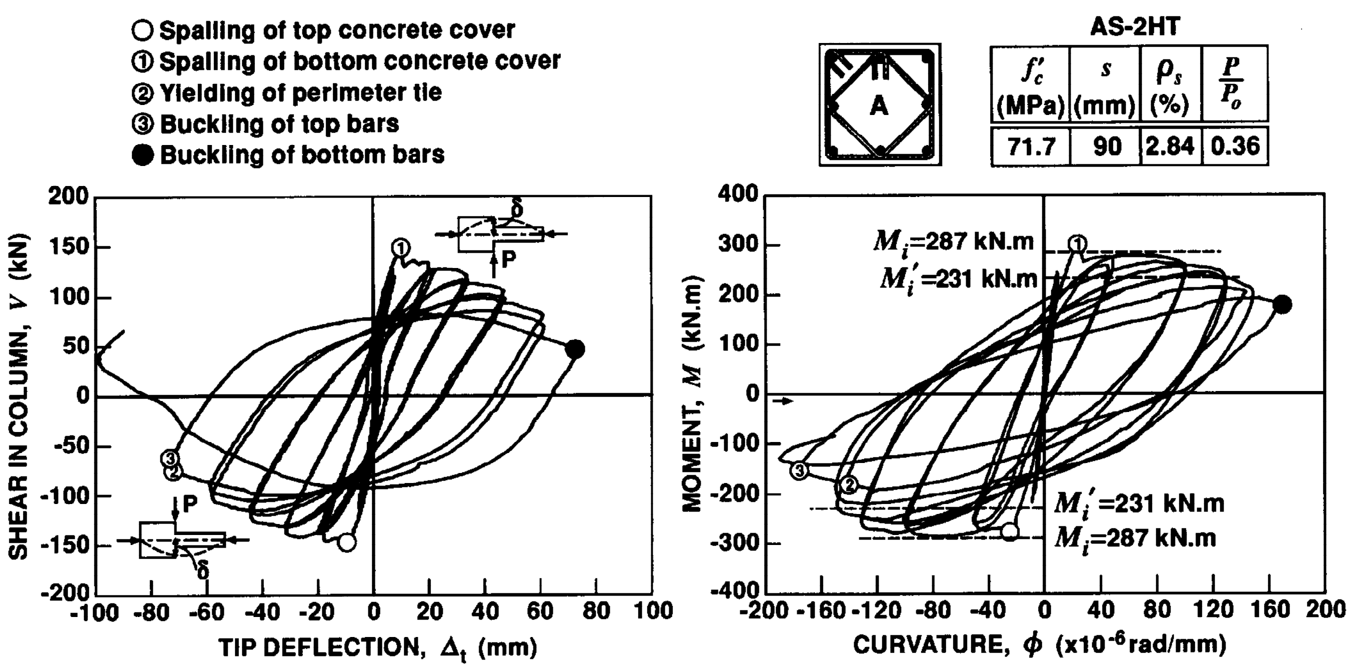
Figure 2. Idealization of Specimens and Definitions of Ductility Parameters



(a)
Shear Force vs. Tip Deflection Behaviour

(b)
Moment vs. Curvature Behaviour

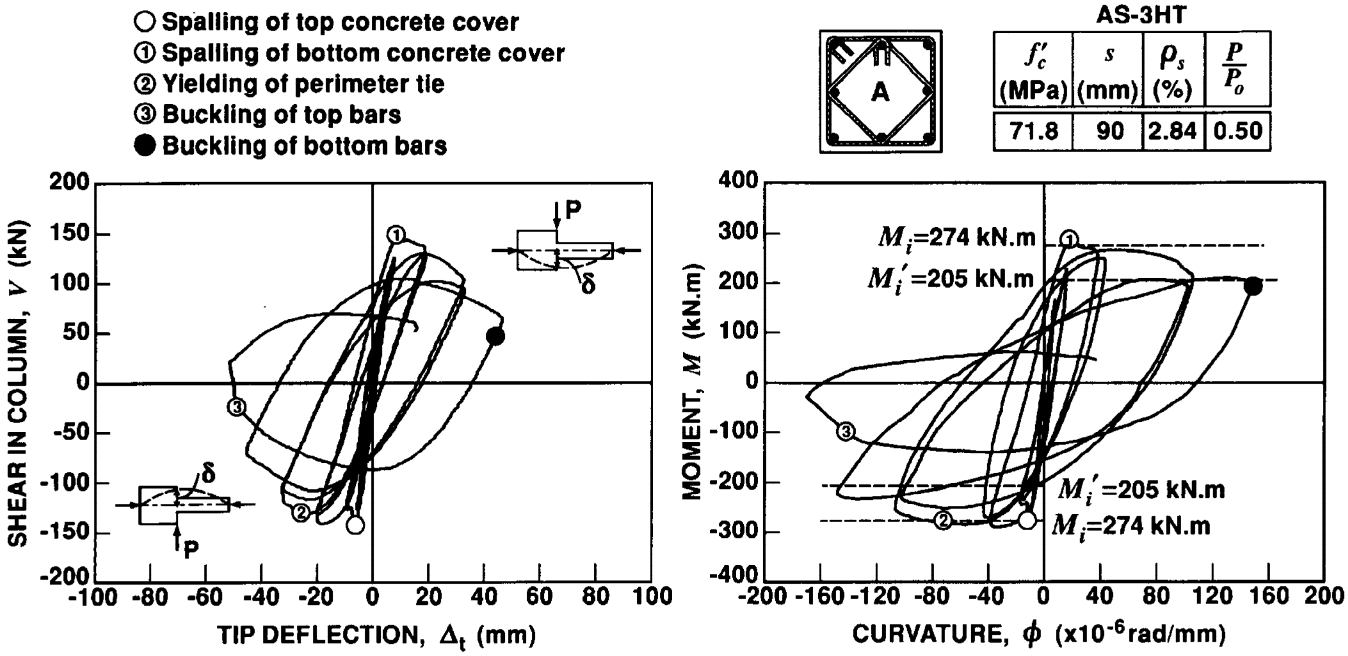
Figure 3. Behavior of Specimen ES-1HT



(a)
Shear Force vs. Tip Deflection Behaviour

(b)
Moment vs. Curvature Behaviour

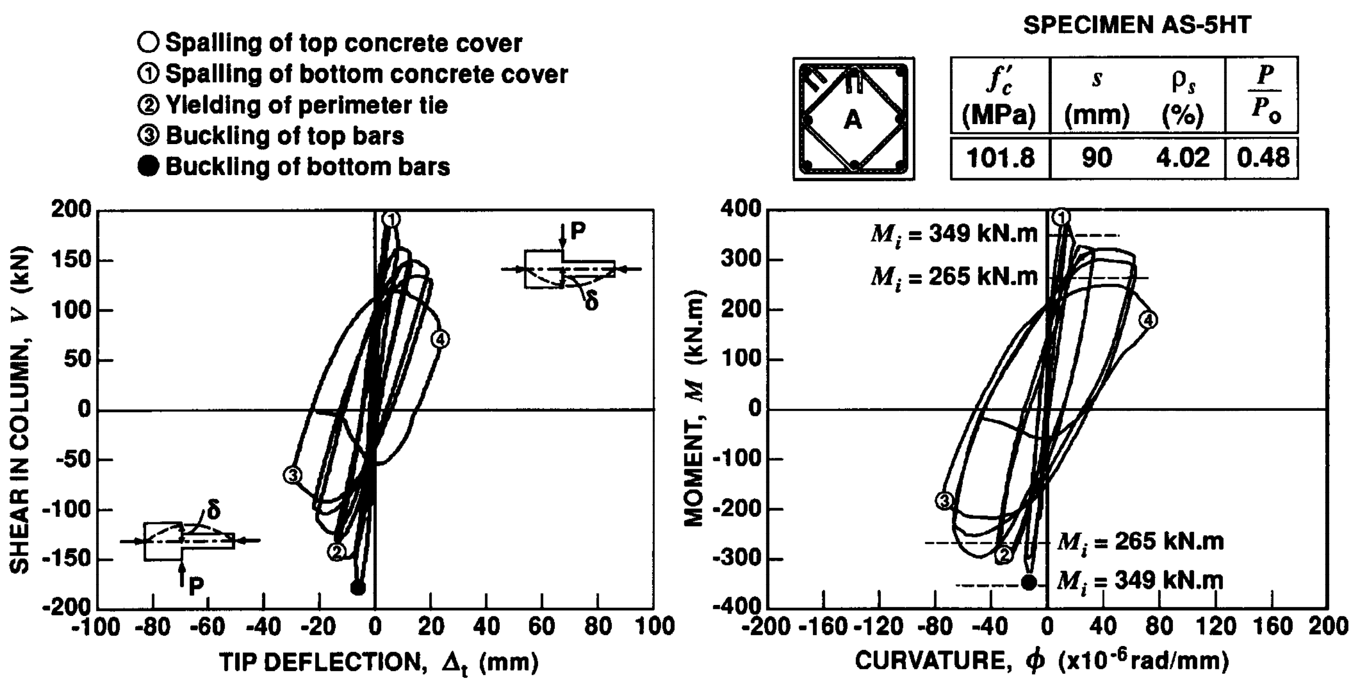
Figure 4 Behavior of Specimen AS-2HT



(a)
Shear Force vs. Tip Deflection Behaviour

(b)
Moment vs. Curvature Behaviour

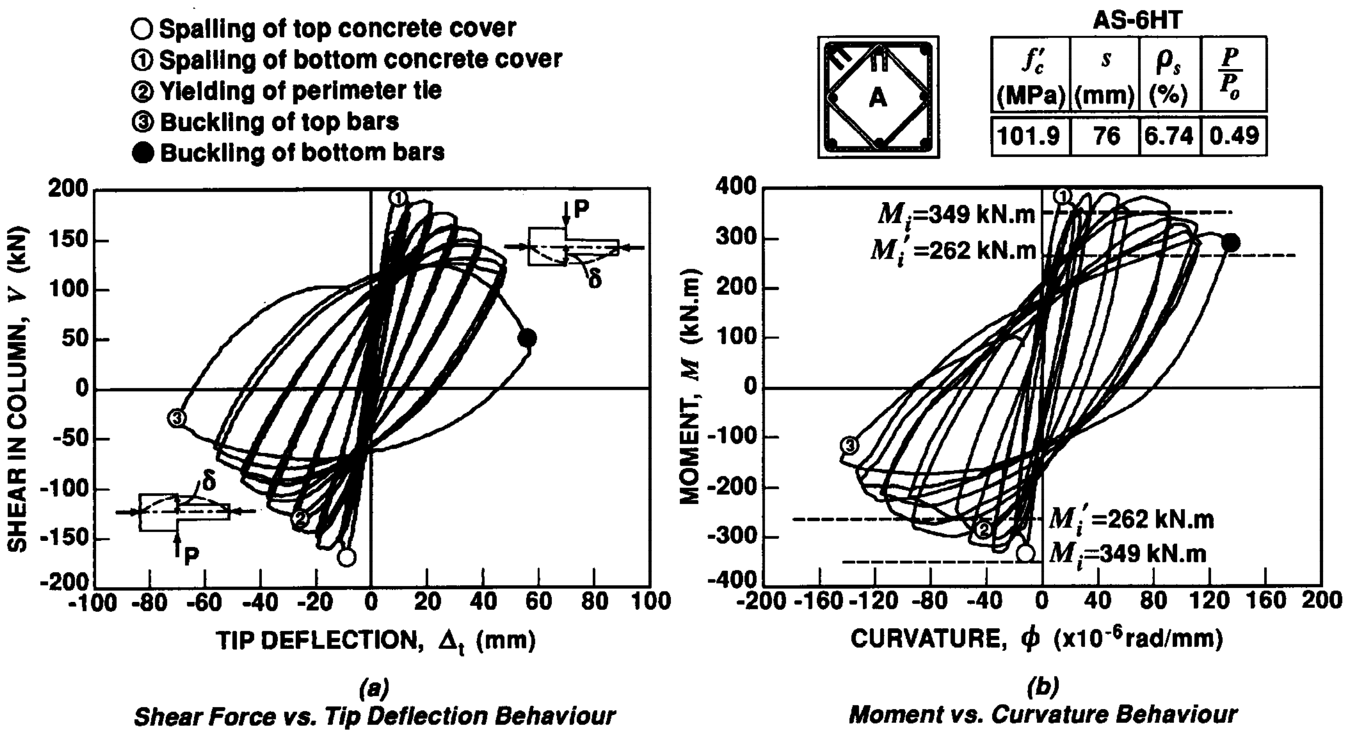
Figure 5. Behavior of Specimen AS-3HT



(a)
Shear Force vs. Tip Deflection Behaviour

(b)
Moment vs. Curvature Behaviour

Figure 6. Behavior of Specimen AS-5HT



f'_c (MPa)	s (mm)	ρ_s (%)	$\frac{P}{P_o}$
101.9	76	6.74	0.49

Figure 7. Behavior of Specimen AS-6HT

Test Observations : First indications of distress in all of the tested specimens were the cracks in the top and bottom concrete cover. Both the number of cracks formed and their lengths increased in the first three cycles as the number of displacement excursions to which specimens were subjected, increased. For Specimens AS-2HT, AS-3HT, AS-5HT and AS-6HT top concrete cover spalled off suddenly at the first downward peak of fourth cycle ($\Delta = 2\Delta_1$), and the bottom concrete spalled off in the second peak of the same cycle. For Specimen ES-1HT , on the other hand, top concrete spalled off suddenly at the first peak of the fourth cycle ($\Delta = 2\Delta_1$), and bottom concrete cover severely cracked in the same cycle and spalled off in the next cycle. In most cases, during the last cycle buckling of the longitudinal bars was observed after the yielding of the perimeter ties, which was an indication of the commencement of failure. The failure of the specimen was accompanied by extensive buckling of the longitudinal bars in all the specimens. In specimen AS-3HT some of the buckled bars fractured.

Effect of Concrete Strength : Four specimens of Configuration A (columns with outer ties and inner ties) can be compared to evaluate the effect of concrete strength on the seismic performance of tied columns. Force-deformation behaviors of these specimens are illustrated in Figures 4 to 7; and various ductility parameters evaluated for these specimens are included in Table 1. The variable $R_{A/P}$ used in this table is defined (Bayrak 1995) as follows;

$$R_{A/P} = \frac{A_{sh}/A_{sh(ACI)}}{P/P_o} \tag{1}$$

It is believed that specimens having the same $R_{A/P}$ ratios, and the same type of lateral steel configuration are comparable. In this expression the level of axial load is presented by the index variable P/P_o rather than the index $P/f_c'A_g$. For columns with similar f_c', both these indices provide similar comparison; but for different f_c' values in columns the comparison does not remain valid with index $P/f_c'A_g$ (Sheikh et al. 1994). Specimen AS-2HT contained 19% more steel than the Code's (ACI 318-89) requirements, and tested under a moderate level of axial load ($P/P_o = 0.36$). Specimen AS-6HT, on the other hand contained 70% more steel than the Code's requirements and tested under a higher level of axial load ($P/P_o = 0.49$). As a result , the $R_{A/P}$ ratios are very similar in these two specimens. An examination of the force-deformation behaviors of these specimens (Figures 4 and 7) and a comparison of their ductility parameters indicate that despite the difference in their concrete strength they displayed very similar behavior. Similar conclusions can be drawn for specimens AS-3HT and AS-5HT which have $R_{A/P}$ ratios of 2.4 and 2.3 respectively. From an examination of $\mu_{\phi 90}$, $N_{\phi 80}$, E_{80} and $N_{\Delta 80}$, W_{80}, it appears that the higher strength concrete specimens have lower deformability and energy absorption capacities initially, but during the later part of the displacement excursions, these properties improve rapidly with increasing effectiveness of confinement due to excessive microcracking in the core concrete; and hence the total values are comparable to those of lower strength concrete specimens.

Effect of Axial Load : Specimens AS-2HT and AS-3HT are very similar in every respect except that P/P_0 is 0.36 for Specimen AS-2HT and it is 0.50 for Specimen AS-3HT. Force deformation relationships of theses two specimens are provided in Figures 4 and 5; and their section and member ductility parameters are included in Table 1. An increase in the axial load from 0.36 P_0 to 0.50 P_0 caused significant decreases in all section ductility parameters similar to what was observed in normal strength concrete columns (Sheikh and Khoury 1993). Reductions of 36%, 62%, and 74% were observed in curvature ductility factor $\mu_{\phi 80}$, cumulative curvature ductility ratio $N_{\phi 80}$ and energy damage indicator E_{80}, respectively. A higher axial load resulted in an increase in the rate of stiffness degradation with every load cycle and adversely effected the seismic performance of HSC columns.

Effect of Steel Configuration : Specimens ES-1HT and AS-3HT contained 13% and 19% more lateral steel than ACI 318-89 requirements and were tested under the same level of axial load. In AS-3HT all the longitudinal bars were laterally supported by tie bends, whereas in ES-1HT only four corner longitudinal bars were laterally supported by tie bends. Due to more efficient confinement mechanism of Configuration A, curvature ductility factors, $\mu_{\phi 80}$ and $\mu_{\phi t}$, of Specimen AS-3HT are approximately 50% larger than those of ES-1HT. Early buckling of middle longitudinal bars of Specimen ES-1HT and subsequent loss of confinement are main reasons behind its relatively brittle behavior. Specimen ES-1HT displayed very poor energy absorption and dissipation characteristics and behaved in a very brittle manner, although it contained 13% more steel than the Code's requirements.

CORROBORATION WITH A PERFORMANCE BASED DESIGN PROCEDURE

Khoury and Sheikh (1991) proposed a procedure for the design of confinement steel, for a given ductile performance which takes into account parameters such as concrete strength, distribution of lateral and longitudinal steel, tie spacing and level of axial load. The spacing was not explicitly included in the design procedure, but the maximum spacing is limited to about 6 times longitudinal bar diameter to avoid premature buckling. The following formulation for the design of confinement steel is suggested:

$$A_{sh} = [A_{sh(ACI)}]\, \alpha\, Y_P\, Y_\phi \tag{2}$$

where α, Y_P, Y_ϕ are factors to account for steel configuration, axial load level and section performance respectively. For columns having "A" type steel configuration α is suggested to be equal to 1.0 and for "E" type steel configuration $\alpha = 2.5$ is suggested. Y_P and Y_ϕ are given by the following equations.

$$Y_P = 1 + 13\left(\frac{P}{P_o}\right)^5 \quad ; \quad Y_\phi = \frac{\mu_\phi^{1.15}}{29} \tag{3}$$

The following simpler and slightly conservative expressions for Y_P and Y_ϕ were also provided.

$$Y_P = 6\frac{P}{P_o} - 1.4 \geq 1.0 \quad ; \quad Y_\phi = \frac{\mu_\phi}{18} \tag{4}$$

It should be recognized that the procedure suggested by Khoury and Sheikh (1991) was suggested for columns with concrete strength up to 55 MPa and its application to HSC is currently being evaluated. Observed curvature ductility factors are compared with the analytical predictions in Figure 8. Curvature ductility ratios, μ_ϕ, predicted by equations 3 and 4 are reasonably close to experimental observations.

CONCLUDING REMARKS

The following conclusions can be drawn from the work reported here.

-Behavior of HSC columns subjected to constant axial load and reversed cyclic lateral load, is substantially improved by confinement provided by rectilinear ties. HSC columns can be made to behave in a ductile manner under high levels of axial load, provided that sufficient amount of confining steel is used in an efficient configuration.

-Overall behavior of HSC columns was observed to be only slightly less ductile compared with that of NSC columns. However, during the stage of loading immediately beyond peak, HSC columns displayed significantly lower deformation and energy dissipation capacities which improved during the later part of successive displacement excursions.

-An increase in axial load decreases column's deformability & ductility, and accelerates stiffness & strength degradation with every displacement cycle. Larger amount of confining steel is required to compensate for this effect.

-Columns with only four corner bars laterally supported by tie bends and designed according to the current seismic design codes' requirements displayed brittle behavior under axial load P equal to 0.50 P_o.

-Since the Code procedure does not consider axial load level and steel distribution in the design, columns such designed can display a wide range of behavior from very ductile to brittle.

-Design procedure suggested by Khoury and Sheikh (1991) provided reasonable estimations for observed curvature ductility ratios for most of the HSC specimens.

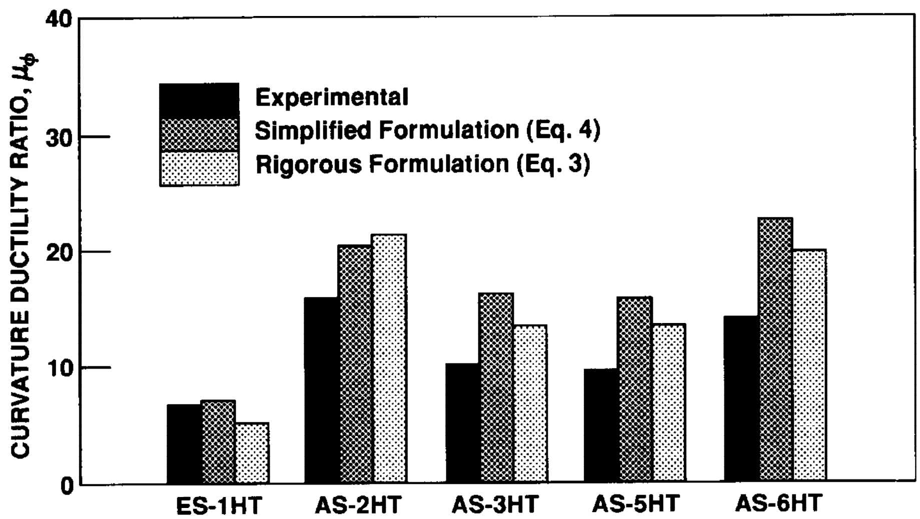


Figure 8. Comparison of Observed and Computed Ductility Ratios

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REFERENCES

ACI Committee 318, "Building Code Requirements for Reinforced Concrete and Commentary (ACI 318-89/ACI 318R-89)", American Concrete Institute, Detroit, 1989, 353p.

Bayrak O., "High Strength Concrete Columns Subjected to Earthquake Type Loading", M.A.Sc. Thesis, Department of Civil Engineering, University of Toronto, Toronto, Ontario, 1995, 239 p.

Code for Design of Concrete Structures for Buildings (CAN3-A23.3-M94)", Canadian Standards Association, Rexdale, Ontario, 1994, 199 p.

Khoury, S.S., and Sheikh, S.A., "Behavior of Normal and High Strength Confined Concrete Columns with and without Stubs", Research Report No.UHCEE 91-4, Department of Civil and Environmental Engineering, University of Houston, Texas, December 1991, 345 p.

Sheikh, S.A., and Uzumeri, S.M., "Strength and Ductility of Tied Concrete Columns", Journal of the Structural Division, ASCE, V.106, ST5, May 1980, pp.1079-1102.

Sheikh, S.A., Khoury, S.S., "Confined Concrete Columns with Stubs", ACI Structural Journal, V.90, No.4, July-August 1993, pp.414-431.

Sheikh, S.A., Shah, D.V., and Khoury, S.S., "Confinement of High-Strength Concrete Columns", ACI Structural Journal, V.91, No.1, January-February 1994, pp.100-111.

Sheikh, S.A., "Design of Confinement Steel for Reinforced Concrete Columns", 5th U.S. National Conference on Earthquake Engineering, Chicago, Illinois, July 10-14 1994, V.2, pp.609-618.