ASIAN JOURNAL OF CIVIL ENGINEERING (BUILDING AND HOUSING) VOL. 7, NO. 3 (2006)
PAGES 269-286

NUMERICAL SIMULATION OF CONFINED CONCRETE COLUMNS AND A PARAMETRIC STUDY

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ABSTRACT

The curiosity to understand the behavior of confined concrete columns has led researchers to carry out extensive experimental work over the years. Numerous empirical confinement models have been reported in the literature for the prediction of stress-strain response of the columns behavior under concentric loading, though, nothing significant has been said about the numerical modeling of the problems wherein the nonlinear response of confined concrete columns may be reasonably predicted. In the present study, material models for both concrete and steel have been formulated and nonlinear finite element analysis of three-dimensional confined concrete models has been carried out. The analysis included the provisions for cover spalling. A couple of examples have been presented for the validation of the numerical methodology proposed in this work. A parametric study has also been carried out to find out the effect of different parameters like concrete strength, reinforcement configuration, spacing of lateral ties and type of cross-section on the response of the confined concrete columns. The parametric study revealed some very important observations regarding the behavior of confined concrete columns.

Keywords: Confined concrete columns, finite element modeling, nonlinear analysis, configuration, spacing

1. INTRODUCTION

Deformability of columns can be achieved through proper confinement of the core concrete. The increase in strength and ductility of concrete confined by well-detailed lateral confinement reinforcement is well documented now [1-3]. When concrete is subjected to compressive load, it undergoes volumetric changes with a lateral increase in dimensions due to Poisson's effect. If transverse reinforcement is provided in the form of the spirals, hoops or ties, it resists this tendency of lateral expansion of concrete by developing tensile forces and consequently exerting a compressive reaction force on the concrete core. In this state of

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multi-axial compression, both the deformation capacity and strength of the concrete are enhanced. Such concrete is said to be confined.

Over the last five decades, lots of experiments have been carried out on tied concrete columns by researchers to observe the behavior under concentric loading [1-4]. Numerous models exist in the literature to predict stress-strain law of normal and high strength concrete confined columns [1, 2, 5-7]. Most of these models have limited validity in terms of concrete strength, column geometry, transverse reinforcement yield strength and loading condition. A comparative study of confinement models [8] indicated that the capability of Legeron and Paultre (2003) model to predict the actual test behavior is best among all these empirical models.

In the last three decades, tremendous progress has been made in the modeling of concrete and the numerical analysis of reinforced concrete structures. The rapid developments in the field of structural analysis, however, have not been matched by the developments in material modeling for reinforced concrete. The application of finite element method to tied concrete columns is still at the initial stage [9-12]. The available literature informs that researchers have used different material models to carry out analytical studies on confined concrete columns under eccentric loading. Chen and Mau [1989] simulated the behavior of spirally reinforced circular concrete columns under axial compression using a recalibrated plastic-fracturing formulation originally developed by Bazant and Kim. Abdel-Halim et al., in their analytical study on tie-confined concrete columns under uniaxial loading, used an isotropic nonlinear-elastic model proposed by Cedolin et al. These works were mainly focused on simulating normal strength concrete columns. An analytical study on tie confined concrete columns under eccentric loading was carried out by Xie et al. in 1996, for high strength [59-124 MPa] concrete columns using a fracture energy based plasticity formulation proposed by Pramono and William. An interesting study by Liu and Foster [1998] revealed some important observations on cover spalling. In their study, an axisymmetric FE model based on the explicit micro plane formulation of Carol et al. was used. From their FE analysis they concluded that the early cover spalling in confined high strength concrete columns can be explained by the triaxial state of stress at the cover-core interface. A high tensile strength is observed to exist between the cover-core interfaces at loads significantly below the peak load.

2. RESEARCH SIGNIFICANCE

Experiments have always played a significant role in the research related to concrete structures. But analytical studies have its own importance, as it can give a much better insight to the behavior of the concrete structures. The complex pattern of stress distributions, which experimental studies can not describe properly, can be captured in a much better way with the help of a successful numerical simulation. One can have a better understanding of the behavior and failure mechanism of the tied columns from the successful numerical investigation. And the subsequent parametric study carried out reveals the effects of varying the different parameters. Also, there remains the scope to modify the model so that the parameters remaining uncovered can be included in the simulation, although that is the subject of another work.

3. NUMERICAL SIMULATION

Reinforced concrete exhibits inelastic behavior even at early stages of loading and this effect is more pronounced near ultimate load stages. Therefore it becomes essential to carry out a nonlinear analysis and to carry out a nonlinear analysis, development of proper geometric and material models for both concrete and steel is important.

For concrete applications in general, hexahedral elements are found to be more stable and efficient in convergence than the tetrahedral elements [13]. Therefore eight-noded isoparametric element, having translations in x, y and z directions, have been used for the modeling of concrete. A discrete approach, using two-noded three-dimensional isoparametric bar elements, has been adopted for the modeling of reinforcements.

Concrete is known to be ductile in nature under hydrostatic pressure and to undergo brittle failure in tension. Therefore, a material model has been developed to simulate the behavior of concrete in both tension and compression. The actual behavior of concrete in three-dimensional state of stress is extremely complicated. It is well established that under tensile and low compressive stresses, concrete fails by brittle fracture. On the other hand, it can yield and flow like a ductile material under high hydrostatic pressure. Several failure models have been developed for brittle and ductile behavior of concrete [14, 15]. The most commonly used failure criteria are defined in stress spaces by a number of material constants varying from one to five independent control parameters. Though all these models have certain inherent advantages and disadvantages, yet the William-Warnke five-parameter model, to date, is most versatile and sophisticated criterion for the elasto-plastic modeling of concrete [14]. Another option could have been the Drucker-Prager Model. But, the perfectly-plastic assumption of the Drucker-Prager Model fails to capture the dilation characteristics and over-predicts the strength [16].

For modeling concrete in compression, inelastic constitutive relations have been specified by defining yield criterion with the hardening rule and the uniaxial strength. The William-Warnke five-parameter model has been combined with isotropic hardening rule (to describe the changing of the yield surface with progressive yielding) and an associated flow rule (to indicate that the plastic strain would occur in a direction normal to the yield surface) to formulate the inelastic constitutive relations. Thus, the model assumes that, concrete behaves elastically as long as the stress state lies within an initial yield surface. When loading progresses beyond the initial yield surface, plastic flow occurs and the yield surface hardens isotropically up to a failure surface. In this range, the plastic strain rate is governed by the yield function.

A smeared crack approach has been adopted to simulate the cracking phenomenon in concrete. A concrete-based model for the consideration of tension stiffening has been used in the present study to simulate the post-cracking behavior of concrete.

For reinforcement, the constitutive relations have been formulated by defining yield criterion with an isotropic hardening rule and associative flow rule. The von Mises criterion describes the yielding of steel in the present study.

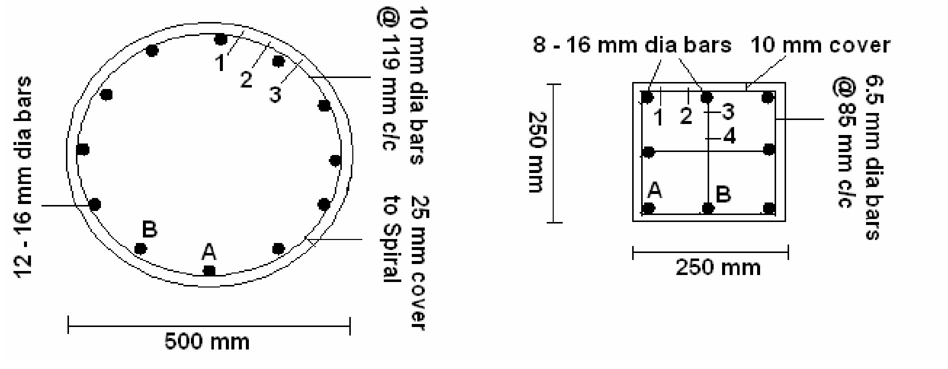
The numerical procedure involved geometric modeling of the confined concrete columns and carrying out nonlinear finite element analysis. In the geometric modeling, complete strain compatibility has been assumed between the reinforcement and the concrete elements. The boundary conditions applied at the ends ensured that, translations in two directions in the horizontal plane are restrained and the translation in the vertical direction is permitted.

The load has been applied at the ends by imposing incremental vertical displacements, with higher displacement increments in the pre-peak region and refined and much smaller displacement increments in the peak range. The solution scheme has been the Newton-Raphson iterative scheme with a force based convergence criteria and a tolerance limit of 0.001.

The spalling of cover in concrete columns is quite established phenomena, though it is more explicitly noticed in high strength concrete columns [3, 4]. Therefore, it needs to be incorporated in the computational model for more realistic analysis. It was included in the analysis by setting the cover elements to a low stiffness once a threshold tensile strain value in the range of 0.0007 to 0.00075 is attained [12].

4. VALIDATION EXAMPLES

In this section an attempt has been made to validate the finite element model formulated, by comparing the numerical predictions with experimental results for a couple of confined column sections. Two test specimens M4 (circular section) [1] and CS17 (square section) [6] representing normal strength and the high strength concrete respectively have been chosen for this purpose. The geometry and reinforcement arrangement of these columns are given in Fig 1 and the material properties are briefed in Table 1.



(a) (b)

Figure 1. Geometry and reinforcement arrangement for (a) M4 (b) CS17

The finite element modeling has been carried out for the test zones, which for M4 is 900 mm and for CS17 is 935 mm. The analysis has been carried out for the test zone lengths as specified for each problem.

Table 1. Material properties for test specimens

Specimen	Material Properties							
	Concrete				**Reinforcements**			
	Compressive Strength (Mpa	**Tensile Strength (MPa)**	**Modulus of Elasticity (MPa)**	**Poisson's Ratio**	**Yield Strength**		**Modulus of Elasticity**	**Poisson's Ratio**
					Longitudinal Steel	**Tie Steel**		
M4	29	3	26000	0.2	295	320	200000	0.3
CS17	68.9	5.2	42037	0.2	450	400	200000	0.3

From the comparison of the stress-strain curves [Figure. 2], obtained both numerically and experimentally for the two specimens, it is evident that the curves follow each other closely and the predicted values for the stress and strain parameters match reasonably well with the observed ones [Table 2].

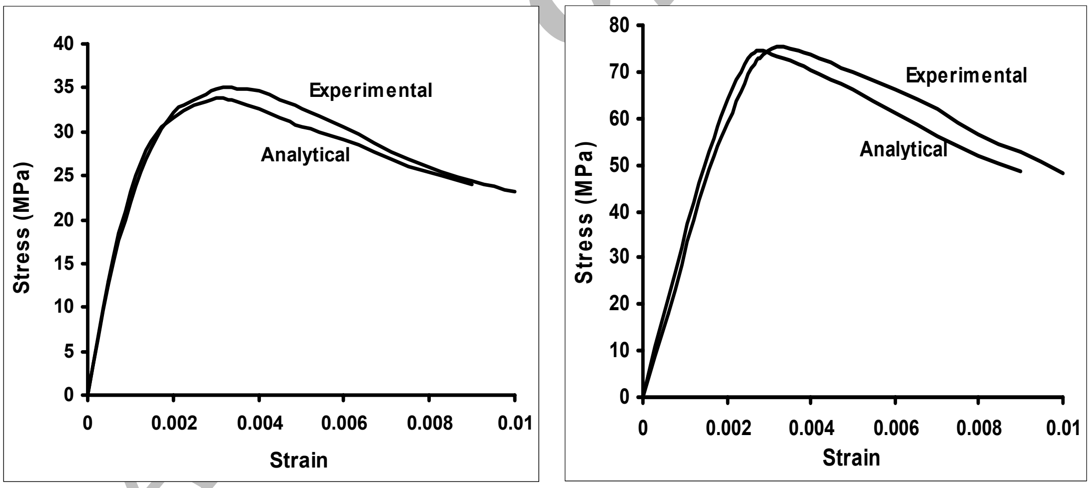
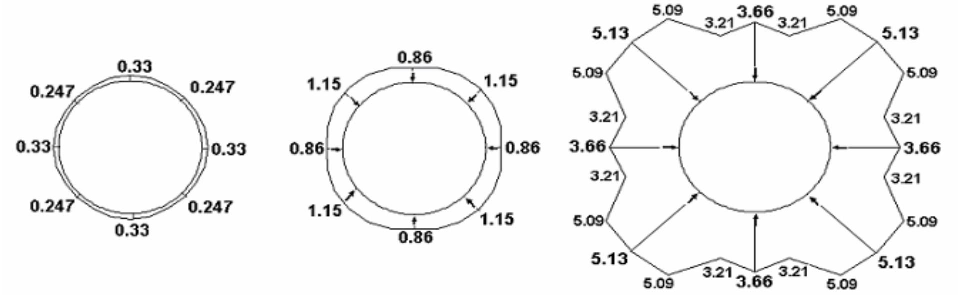


Figure 2. Comparison of analytical and experimental stress-strain curves for (a) M4 (b) CS17

The progressive development of confining stresses at a given section of the computational domain has also been examined in the present study. It is evident from the literature that the distribution of the laterally restrained longitudinal bars and the resulting tie configuration influences significantly the pattern of these stresses at the periphery of core [1, 2, and 4]. The reported experimental and theoretical studies suggest that at the tie level cross section the confining pressure is maximum at the laterally supported longitudinal bar and it is minimum between the two such adjacent bars. It is encouraging to note that finite element analysis results for the two columns specimens provide the variation of lateral confining pressure of the same kind as specified in the earlier studies [Figures. 3 and 4].

Table 2. Comparison of experimental and analytical stress-strain parameters

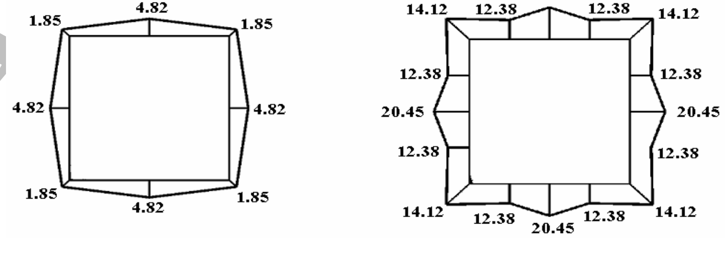
	Experimental			**Analytical**		
Specimen	**Peak Stress Mpa**	**Peak Strain**	**Strain at 85% of Peak Stress in Post Peak**	**Peak Stress Mpa**	**Peak Strain**	**Strain at 85% of Peak Stress in Post Peak**
M4	35	0.003	0.008835	33.93	0.003	0.0082
CS17	75.2	0.0033	0.0068	74.6	0.0028	0.0066



(a) (b) (c)

Note: All the stress values are in MPa.

Figure 3. Confining stresses for M4: (a) Before peak (b) At peak (c) At the end of solution



(a) (b)

Note: All the stress values are in MPa

Figure 4. Confining stresses for CS17: (a) At peak (b) At the end of solution

Table 3. Stress levels in longitudinal and transverse reinforcement

Specimen	**M4**					**CS17**					
Reinforcement Type & Location	**Longitudinal Steel**		**Transverse Steel**			**Longitudinal Steel**		**Transverse Steel**			
	A	**B**	**1**	**2**	**3**	**A**	**B**	**1**	**2**	**3**	**4**
Stress at the Peak (MPa)	295*	295*	174.52	168.21	166.44	353.69	450*	98.37	200	110	122.31
Stress at the End of Solution (MPa)	295*	295*	320*	320*	313	400	450*	142.8	322	188	212

*** Indicates yielding of Reinforcement**

The study indicates that the longitudinal reinforcement yielded invariably by the time peak of stress-strain curves was reached in both the cases. This can be endorsed on the basis of many earlier studies, which have established that longitudinal steel yields prior to lateral ties and this normally happens before or at the peak of stress-strain curve of columns [1, 4]. Also it is apparent from Table 3 that the yielding of lateral ties for specimen M4 coincided with the attainment of the confined concrete peak strength. However, it did not happen so for the specimen CS17 even beyond the peak strength point; the observation that did not match with the experimental findings. Quite a few studies in the past did infer that the lateral ties may not always yield at the peak of axial stress-strain curve [2, 3, and 7]. In fact these studies have shown that the level of stress in the ties at the peak depends upon the available degree of confinement for the given yield strength of lateral ties. To this end, the analytical model proposed by Legeron and Paultre (2003) has been employed to estimate the level of actual stress in the lateral confining steel at the peak of axial stress-strain curve. A comparative study of the various confining models has clearly illustrated the superiority of Legeron and Paultre (2003) model in predicting the actual test behavior for both normal and high strength concrete columns [8]. The model predicted the value of tie stress at the peak of CS 17 to be 206 MPa, which is considerably less than the yield strength of ties and very close to the values estimated in the present study.

5. PARAMETRIC STUDY

Non-linear finite element analysis is a time-consuming and complicated process for routine use in assessing or designing confined concrete columns. Therefore, these techniques have been directed towards the development of the design concepts, which, in turn, enables the researchers and designers to have better insight of the problem. To this end, an extensive parametric study based upon finite element code proves to be a significant tool and the findings of it can be easily summarized to evolve a rational basis of confined concrete column design. A parametric study has been carried out, in the present work, to find out the effect of varying strength of concrete, reinforcement configuration, spacing of tie steel and the column cross section on the stress-strain response of confined concrete columns. The various details of all the columns are given in Table 4. The parametric study has been carried out for a 750 mm long column. Both square (300 mm x 300 mm) and circular sections (535 mm diameter) of columns have been studied. A clear cover of 40 mm has been provided for the columns.

5.1 Concrete Strength

The effect of varying the concrete strength on the stress-strain response of the confined concrete columns has been studied for two different reinforcement configurations, C and D, with the concrete strength varying from 30 to 90 MPa. The tie spacing has been kept constant at 75 mm c/c and the strength of reinforcements at 415 MPa.

5.2 Reinforcement Configuration

The reinforcement configuration determines the effectively confined concrete area, which increases with a better distribution of longitudinal bars around the column concrete core. The larger the effectively confined concrete area, the higher would be the confinement efficiency. The effect of reinforcement configuration on stress-strain behavior of confined concrete columns has been studied for four different unconfined concrete strengths – 30 MPa, 60 MPa, 90 MPa and 120 MPa. The spacing of lateral reinforcements has been taken as 75 mm c/c. The yield strength of both longitudinal and lateral steel has been taken as 415 MPa. Three dimensional nonlinear finite element analyses, for each of the four concrete compressive strengths (unconfined), have been carried out for four different reinforcement configurations – A, B, C and D [Figure 5].

The total reinforcement percentages in all the columns have been kept almost same (around 2.37 %), so as to have a rational comparison [Table 4].

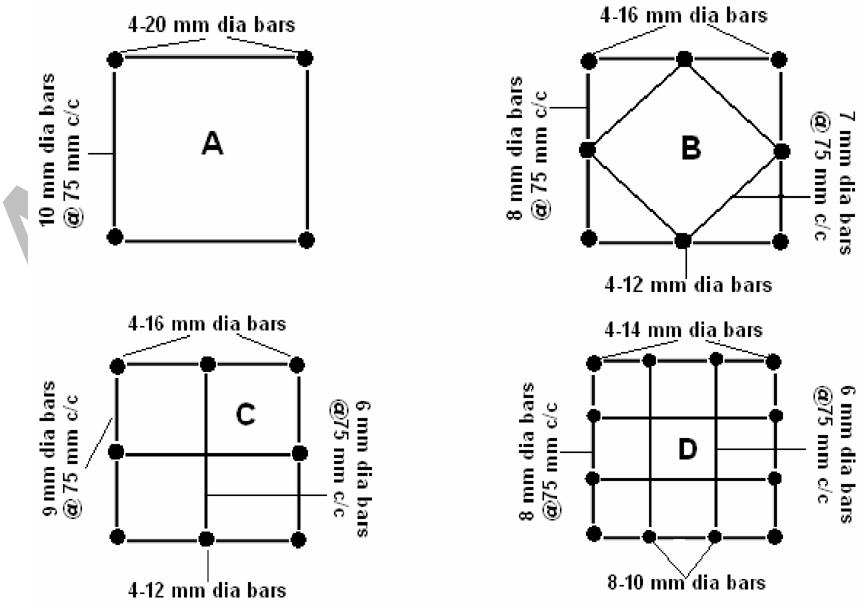


Figure 5. Reinforcement configurations considered for parametric study

Table 4. Details of the specimens considered for the parametric study

Specimen	Configuration	Longitudinal Reinforcement		Lateral Reinforcement				Concrete Strength (MPa)	Ke	Percentage Reinforcement		
		Dia (mm)	Yield Strength (MPa)	Dia (mm)		Yield Strength (MPa)	Spacing (mm)			Longitudinal	Lateral	Total
				Internal	External							
A1	A	20 (4 nos)	415	-	10	415	75	30	0.2413	1.396	0.977	2.373
B1	B	16 (4 nos)+12 (4 nos)	415	7	8	415	75	30	0.4807	1.396	0.973	2.369
C1	C	16 (4 nos)+12 (4 nos)	415	6	9	415	75	30	0.4836	1.396	0.972	2.368
D1	D	14 (4 nos)+10 (8 nos)	415	6	8	415	75	30	0.5613	1.382	0.987	2.369
A2	A	20 (4 nos)	415	-	10	415	75	60	0.2413	1.396	0.977	2.373
B2	B	16 (4 nos)+12 (4 nos)	415	7	8	415	75	60	0.4807	1.396	0.973	2.369
C2	C	16 (4 nos)+12 (4 nos)	415	6	9	415	75	60	0.4836	1.396	0.972	2.368
D2	D	14 (4 nos)+10 (8 nos)	415	6	8	415	75	60	0.5613	1.382	0.987	2.369
A3	A	20 (4 nos)	415	-	10	415	75	90	0.2413	1.396	0.977	2.373
B3	B	16 (4 nos)+12 (4 nos)	415	7	8	415	75	90	0.4807	1.396	0.973	2.369
C3	C	16 (4 nos)+12 (4 nos)	415	6	9	415	75	90	0.4836	1.396	0.972	2.368
D3	D	14 (4 nos)+10 (8 nos)	415	6	8	415	75	90	0.5613	1.382	0.987	2.369
C4	C	16 (4 nos)+12 (4 nos)	415	6	9	415	37.5	30	0.4836	1.396	1.944	3.340
C5	C	16 (4 nos)+12 (4 nos)	415	6	9	415	112.5	30	0.4836	1.396	0.648	2.044
C6	C	16 (4 nos)+12 (4 nos)	415	6	9	415	37.5	90	0.4836	1.396	1.944	3.340
C7	C	16 (4 nos)+12 (4 nos)	415	6	9	415	112.5	90	0.4836	1.396	0.648	2.044
SC1	E	16 (4 nos)+12 (4 nos)	415	-	10.5	415	75	30	0.8911	1.396	1.007	2.403
SC2	E	16 (4 nos)+12 (4 nos)	415	-	10.5	415	75	60	0.8911	1.396	1.007	2.403

5.3 Lateral Steel Spacing

Varying the lateral steel spacing, results in increment or decrement of the percentage of lateral steel. The lateral confining pressure imposed on concrete core is directly related to the percentage of lateral steel. A larger percentage of lateral steel would result in a larger confining pressure being applied on the concrete core and would therefore increase the confinement efficiency. To observe the effect of spacing of lateral reinforcement on strength and ductility of confined concrete columns, reinforcement configuration C has been chosen for the analysis purpose. The yield strength of the reinforcements has been taken as 415 MPa. For each of two different concrete strengths, M30 and M90, nonlinear finite element analysis has been carried out three times for the three different tie spacing: one 37.5 mm (SP1), one 75 mm (SP2) and the other 112.5 mm (SP3).

5.4 Column Cross-Section

The effect of varying the cross-section of confined concrete columns has also been studied in the present study. A circular cross-section [configuration E], equivalent to the square section [configuration C], has been chosen for comparison, keeping all the other parameters (e.g. concrete strength, reinforcement percentage, lateral reinforcement percentage, spacing of lateral steel and strength of reinforcements) constant [Fig 6]. The study has been carried out for two different concrete strengths – 30 and 60 MPa.

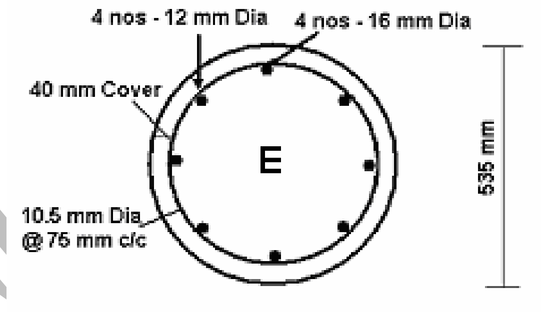
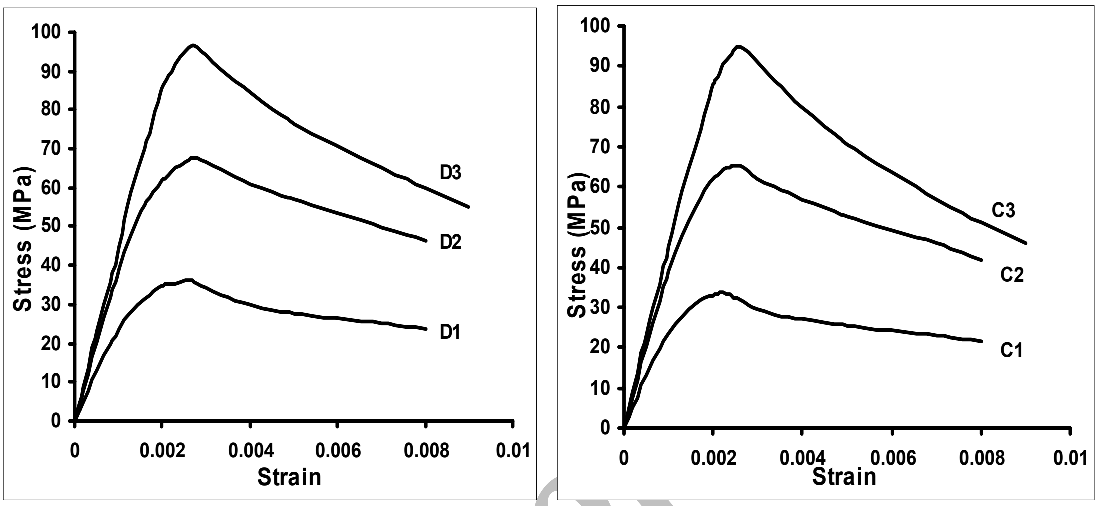


Figure 6. Column with circular section

6. DISCUSSION OF THE RESULTS OF THE PARAMETRIC STUDY

6.1 Effect of Concrete Strength

The concrete compressive strength was the most important and one of the primary variables investigated extensively in the study. The behavior of columns with same volumetric ratio of reinforcement, lateral steel spacing spacing, reinforcement configuration and yield strength of lateral steel, but with different concrete strengths was compared to quantify the effects of this parameter (Figure 7). The stress-strain responses clearly suggest the brittle nature of high strength concrete. For same reinforcement configuration, increase in concrete strength, results in a steeper post-peak region and therefore lesser ductility. The decreasing trend of strength enhancement and ductility for columns from lower strength concrete to higher strength concrete columns show that effectiveness of confinement decreases as the concrete strength increases. Therefore, if the same levels of strength and ductility enhancements are desired, higher strength concrete columns shall require more confinement than lower strength concrete columns.



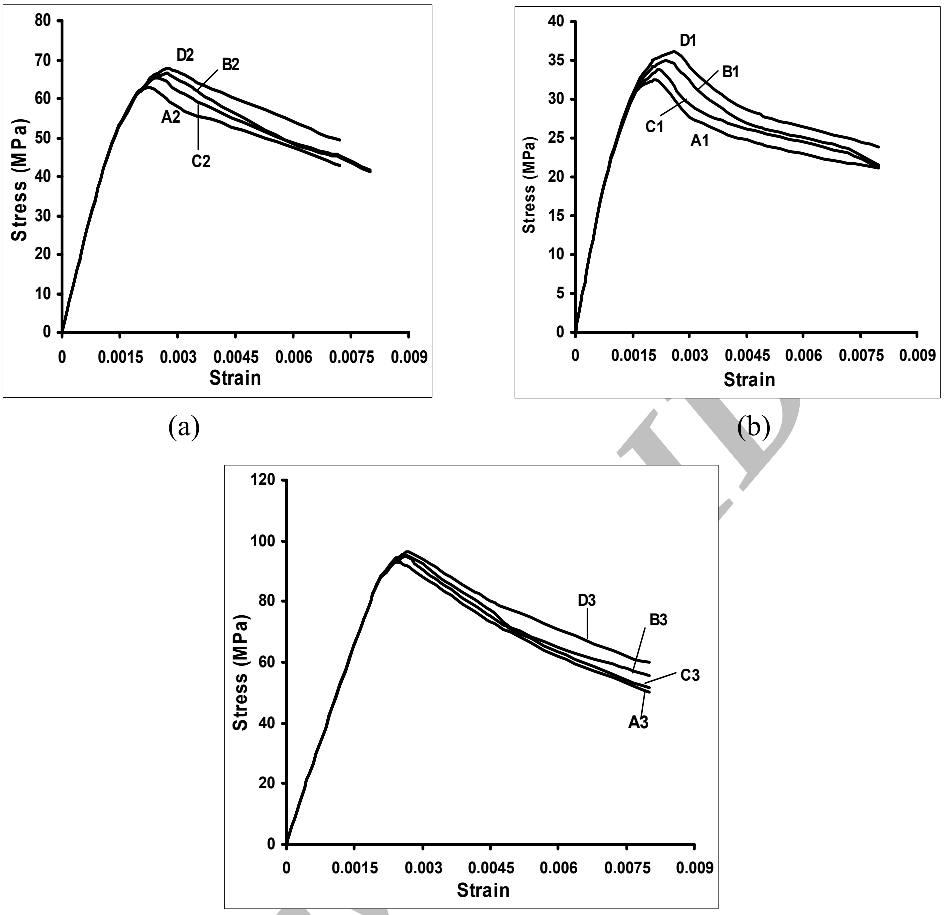
(a) (b)

Figure 7. Effect of varying concrete strength for (a) Configuration C and (b) Configuration D

6.2 Effect of Configuration

The stress-strain responses for the four different configurations for different concrete strengths are given in Fig 8. The comparison clearly indicates that the tie configuration D is the most efficient and A is the poorest in increasing the strength of confined concrete. Comparing the curves one can also see the pattern of improved behavior for type B configuration over type C configuration, for both normal and high strength concrete.

Besides increasing the strength, enhancement of ductility is another important reason for providing confining reinforcements. Ductility can be presented in different ways such as strain ductility, curvature ductility, displacement ductility etc. In our present study strain ductility is considered as the measure of ductility. The ratio of strain corresponding to 85% of peak stress and strain corresponding to the peak stress is considered as strain ductility. To observe the variation in ductility for different configurations, a factor called confinement effectiveness coefficient (given by Mander et al., 1988) is calculated for each configuration [Table 4]. The comparison of variation in ductility [Fig 9] with confinement effectiveness coefficients for different grade of concrete indicates that configuration D is the superior among the four. Interestingly, column with configuration B has greater ductility than that with configuration C, though B and C has almost same confinement effectiveness co-efficient.



(a) (b)

(c)

Figure 8. Effect of varying reinforcement configurations for (a) M30 grade concrete (b) M60 grade concrete and (c) M90 grade concrete

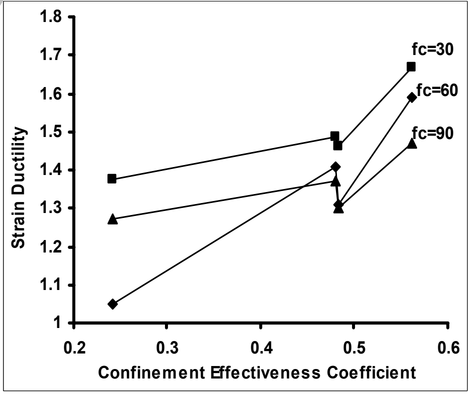


Figure 9. Comparison of variation in ductility with confinement effectiveness coefficient for different reinforcement configurations for different grades of concrete

A close inspection of the stress levels in the lateral and longitudinal reinforcements [Table 5a and 5b] leads to some important findings. The yielding of longitudinal reinforcements occurred near the peak and much before the ties yield. The longitudinal bars in the corner regions are less stressed than the bars in the middle region of the cross section and they also reach the yield stress later. This is indicative of the fact that the corner regions are more effective in confining the core and consequently the bars at the middle region of the cross section are more likely to cause the failure of the column. From the observation of the stress levels in the ties it can be seen that the stresses in the ties for configuration A, B and C increases in the post-peak region for normal strength concrete, but decreases in case of high strength concrete. The reason for such behavior is due to the fact that high strength concrete exhibits less lateral expansion under axial compression than normal strength concrete due to its higher modulus of elasticity and lower internal micro cracking and consequently the confining reinforcements come into play later in the process and the efficiency of the passive confinement gets reduced.

But, in case of configuration D, the stresses in ties increase in the post-peak region even for higher strength concrete. Moreover, the ties of configuration D have comparatively lower stress levels for normal strength concrete columns, but results in greater increase in strength and ductility. All these clearly suggest that, configuration D, with its better lateral steel distribution, is the most efficient one among the four considered in producing a compact core and thereby, enhancing the strength and ductility for both normal and high strength concrete. On the other hand, configuration A, in spite of having stress levels in ties similar to that of the other configurations, proves to be the least efficient in increasing the strength and ductility. A comparison between configuration B and C indicate that, though the stress levels in peripheral ties for both configuration B and C are similar, the cross ties in C are more stressed than the internal ties of B. However, configuration B enhances the strength and ductility more than C. The inclined ties in configuration B, not only prevents the lateral displacements of the peripheral ties, but also strengthens the corner regions and produces a more compact core by preventing the internal cracking due to its better organization. On the other hand, the cross ties in configuration C, though prevents the lateral displacement of peripheral ties are not better organized to prevent the internal cracking. The above-mentioned reasons result in better performance of configuration B compared to C. Interestingly; in all the cases the peripheral ties are less stressed than the internal ones.

6.3 Effect of Spacing of Lateral Reinforcement

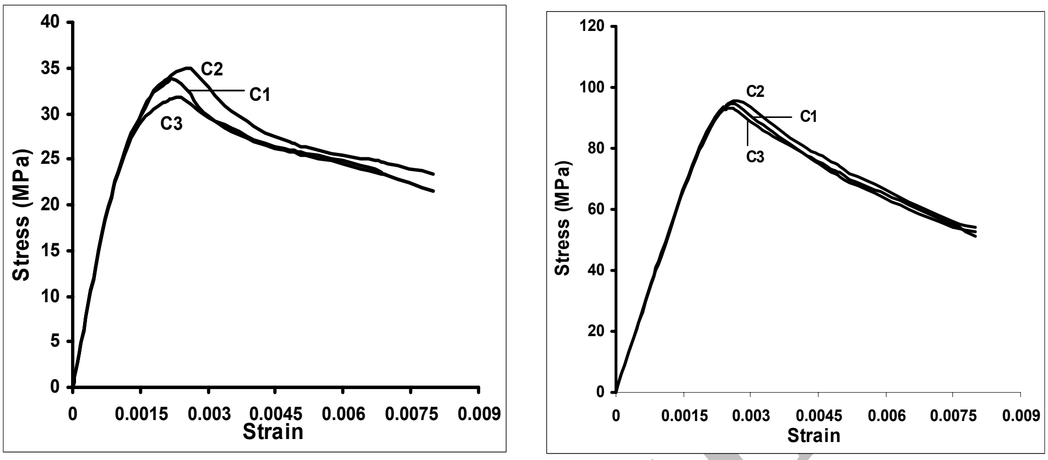
The importance of the amount of lateral confining steel as a factor that affects the behavior of confined concrete is well recognized. A decrease in the spacing of lateral steel and hence an increase in volumetric ratio of confining reinforcement may be directly translated into a proportional increase in lateral confining pressure. The spacing of transverse reinforcement is an important parameter that affects the distribution of confinement pressure on the confined core in addition to the stability of longitudinal bars. The stress-strain responses for different tie spacing for different concrete strengths are illustrated in Figure 10.

Table 5a. Stress in ties for different configurations

Configuration			**A**		**B**			**C**				**D**					
Tie Locations			**1**	**2**	**1**	**2**	**3**	**1**	**2**	**3**	**4**	**1**	**2**	**3**	**4**	**5**	**6**
Stress Level	At Peak	M30	67	110	76	113	105	71	110	101	128	60	99	112	85	111	122
		M60	77	113	85	116	109	80	114	104	125	69	104	114	92	115	120
		M90	79	114	87	117	109	84	116	106	123	75	110	121	97	118	125
	At End	M30	104	260	116	268	240	112	272	169	415	63	205	272	116	332	354
		M60	119	379	125	316	287	122	392	246	415	87	185	220	106	282	245
		M90	97	202	110	218	209	100	203	133	287	101	215	279	112	318	340

Table 5b. Stresses in longitudinal reinforcements for different configurations

Configuration			**A**	**B**		**C**		**D**	
Longitudinal Steel Locations			**a**	**a**	**b**	**a**	**b**	**a**	**b**
Stress Level	At Peak	M30	348	346	363	341	353	341	357
		M60	387	378	404	377	393	380	396
		M90	415	415	415	415	415	415	415
	At End	M30	415	415	415	415	415	415	415
		M60	415	415	415	415	415	415	415
		M90	415	415	415	415	415	415	415



(a) (b)

Figure 10. Effect of varying lateral steel spacing for (a) M30 grade (b) M90 grade concrete

The comparison clearly shows that the column with 37.5 mm spacing enhances the strength and post-peak deformability more than the others for both normal and high strength concrete. The comparison of variation in ductility with confinement effectiveness coefficients [Table 4] for different grade of concrete [Fig 11] confirms this.

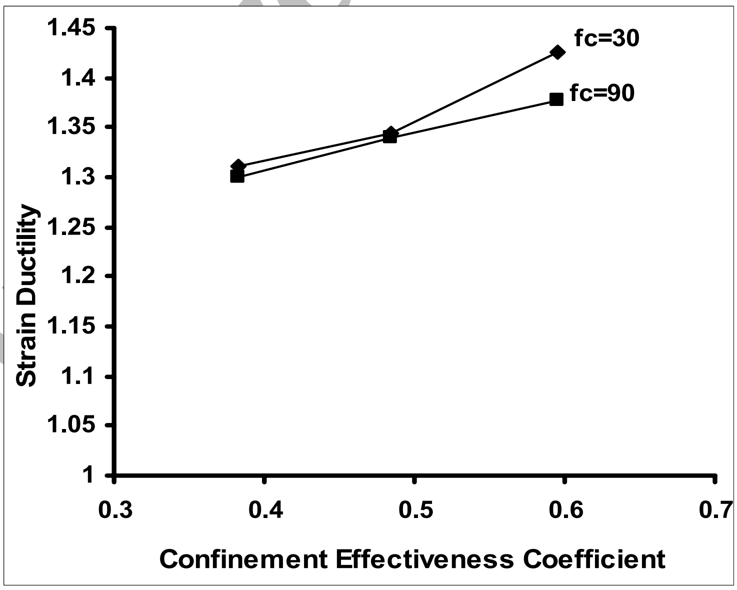
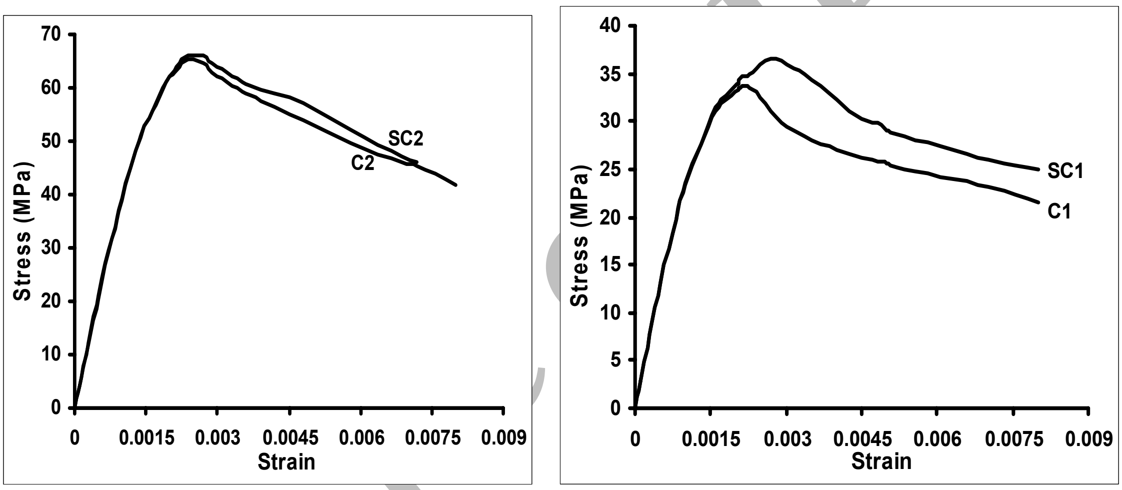


Figure 11. Comparison of variation in ductility with confinement effectiveness coefficient for different lateral steel spacing for different grades of concrete

6.4 Effect of varying the Cross-Section

It is well established experimentally now, that circular spirals are more effective in confining concrete than rectilinear ties owing to their better uniform distribution of lateral confining pressure around the core compared to the case of square or rectangular ties. In the present study this fact has been tried to be investigated analytically. The comparison of the stress-strain curves [Fig 12] clearly indicates that columns with circular cross-section perform better than those with square ones. The columns with circular cross-sections had greater peak stress and strain values than the square ones for both the cases with concrete strengths 30 and 60 MPa. Therefore, if the same percentages of strength and ductility enhancements are desired, square or rectilinearly confined columns are required to be confined more vigorously than circular columns.



(a) (b)

Figure 12. Effect of varying cross-section for (a) M30 grade and (b) M60 grade concrete

7. CONCLUSIONS

This paper presents the analytical results of a detailed finite element simulation study carried out on confined concrete columns subjected to concentric axial compression. The non-linear behavior of concrete in compression has been idealized by employing an elasto-plastic strain-hardening model. The William-Warnke five-parameter model has been used to define the initial and the subsequent yield surfaces. A fixed crack smeared approach has been employed for the modeling of the tensile behavior of the concrete material. The resulting model is shown to validate well the actual test behavior of confined concrete. The findings of a detailed theoretical parametric study are also reported. The variables are concrete compressive strength, lateral steel configuration, spacing of transverse reinforcement and shape of cross section.

A consistent decrease in strength enhancement and deformability of columns is observed with increasing concrete strength. Therefore, a higher degree of confinement is required in columns with higher concrete strength than in a column with lower concrete strength to achieve similar advantages. The configuration and spacing of lateral steel have significant effect on the behavior of confined concrete. For the same volumetric ratio of lateral steel, decrease in spacing of confining steel results into an increase in strength and ductility. The strength and ductility of columns also improve significantly if lateral steel is distributed properly around the longitudinal steel in an efficient configuration. The results show that circular confinement is better than the rectilinearly confined sections.

REFERENCES

1. Mander J.B., Priestly M.J.N. and Park R., Theoretical stress-strain model for confined concrete, *ASCE Journal of Structural Engineering,* No. 8, **114**(1988) 1804-1826.
2. Sheikh S.A. and Uzmeri S.M., Strength and ductility of tied concrete columns, *ASCE Journal of Structural Division*, No. 5, **106**(198) 1079-1102.
3. Cusson, D., and Paultre, P., High Strength Concrete Columns Confined by Rectangular Ties, *J. Struct Engg. Div., ASCE*, No. 3, **120**(1994) 783-804.
4. Razvi S. and Saatcioglu M., Tests of high strength concrete under concentric loading, *Report No. OCEERC 96-03, Carleton Earthquake Engineering Research Centre, Ottawa, Canada*, 1996.
5. Cusson, D., and Paultre, P., Stress-strain Model for Confined High Strength Concrete, *J. Structural Engg. ASCE,* No.3, **121**(1995) 468-477.
6. Razvi, S., and Saatcioglu, M., Confinement Model for High Strength Concrete, *J. Structural Engg., ASCE*, No. 3, **125**(1999) 281-289.
7. L'egeron, F., and Paultre, P., Uniaxial Confinement Model for Normal- and High-Strength Concrete Columns, *J. Struct. Engg. Div., ASCE*, No. 2, **129**(2003) 241-252.
8. Sharma, U., Bhargava, P., and Kaushik, S.K., Comparative Study of Confinement Models for High Strength Concrete Columns, *Magazine of Concrete Research,* No. 2, **57**(2005) 185-197.
9. Chen, B., and Mau, S. T., Recalibration of a Plastic-Fracturing Model for Concrete Confinement, *Cement and Concrete Research,* **19**(1989) 143-154.
10. Abdel-Halim, M. A. H., and Abu-Lebdeh. T. M., Analytical Study for Concrete Confinement in Tied Columns, *J. Struct. Engg. Div., ASCE,* No.11, **115**(1989) 2810-2828.
11. Xie, J., Elwi, A. E., and MacGregor, J. G., Numerical Investigation of Eccentrically Loaded High-Strength Concrete Tied Columns, *ACI Structural J.,* No. 4, **93**(1996) 449-461.
12. Foster, S. J., and Liu, J., Cover Spalling in HSC Columns Loaded in Concentric Compression, *J. Struct Engg. Div., ASCE,* No. 12, **124**(1998) 1431-1437.
13. Chen, W. F., and Lan, Y. M., Finite Element Study of Confined Concrete, *Proc. Int. Symp. On Confined Concrete, Changsha, China*, 2004, 21-34.
14. Chen, W.F., *Plasticity in Reinforced Concrete*, McGraw-Hill, New York, 1982.
15. Chen, W.F. and Saleeb, A.F., Constitutive Equations for Engineering Materials, Vol, I.,

Elasticity and Modeling, Wiley & Sons, 1994.

16. Mirmiran, A., Zagers, K., and Yuan, W., Nonlinear Finite Element Modleing of Concrete Confined by Fiber Composites, *Finite Elements in Analysis and Design*, No. 1, **35**(2000) 79-96.