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# Effect of external restraint on the behavior of confined concrete columns

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ABSTRACT: Results from a select group of specimens tested under a variety of loading and geometric conditions are presented here. Test variables included the presence of stub adjacent to the critical section, lateral support provided to the longitudinal bars, level of axial load and the amount of lateral steel. Due to the confinement provided by lateral reinforcement and stub restraint, an increase as high as 70% in the section's flexural capacity was observed in some specimens. Stub restraint causes the failure to move away from the adjacent critical section which will result in higher than expected shear force. Quality of the lateral support provided to the longitudinal bars has a very significant effect on section and member ductility. Increase in lateral steel contents can result in a proportional increase in the energy absorption capacity of a well-confined member if premature buckling of bars can be avoided.

## INTRODUCTION

Failure of an entire structure can be triggered by the failure of columns as has been observed in the after math of several earthquakes ("Reducing" 1986). It is precisely this type of total collapse that various design codes aspire to avoid. In the ACI Code ("Building" 1989) seismic design approach contains strong column-weak beam concept and it is recommended that the sum of design flexural strengths of columns is at least 20% larger than that of girders in a certain plane at a joint. Recent research (Paulay 1986) has indicated that to prevent the column hinging during inelastic displacements of a frame, the ratio between the nominal flexural strengths of columns to those of beams at a joint may need to be in the range of 2.0 to 2.5. Recognizing this uncertainty of column hinging, the code recommends the use of confining steel when the axial load on the columns is larger than $0.1f_c'Ag$ where $f_c'$ is the concrete compressive strength as measured from a 150 x 300 mm cylinder and Ag is the gross cross sectional area of the column. The amount of lateral steel required is independent of the level of axial load, distribution of steel, tie spacing, column performance and any external restraint to the critical region of a column.

In general the critical regions of the columns, where confinement steel is used, are in the vicinity of beam column joints and at the base of a structure. In a well-designed structure, the joints and the foundations should not experience excessive inelastic deformations during a severe earthquake. Heavy undamaged elements adjacent to the column critical regions have very significant effect on the column section properties. The external restraint causes an increase in concrete strength and hence an increase in the flexural capacity of the section particularly when the column axial load is large. In a capacity design approach underestimation of the section flexural capacities would result in an underestimation of the design shear force and may result in a brittle shear failure.

## EXPERIMENTAL WORK

In an extensive experimental program, a large number of confined concrete specimens with varied sizes and shapes were tested under a variety of loading and geometric conditions (Sheikh and Yeh 1990, Khoury and Sheikh 1991, Shah and Sheikh 1991). Variables in the overall test program included distribution of longitudinal and lateral steel, spacing and amount of lateral steel, amount of longitudinal steel, concrete type and strength (lightweight vs. normal weight and normal strength vs. high strength), characteristics of lateral steel, type of loading and presence of stub adjacent to the critical section. Results from a few of these tests are discussed here along with the details of the relevant specimens with a view to examine a select group of variables such as level of axial load in case of specimens subjected to axial load, shear and flexure, steel configuration, amount of confining steel and the restraint provided by a stub to the adjacent section.

Specimens selected for presentation here include 305 x 305 x 2,740 mm prismatic columns and 305 x 305 x 1,450 mm columns with 510 x 760 x 810 mm stubs at one end. The prismatic specimens were tested under monotonic lateral loads while simultaneously subjected to constant axial load such that the critical test region was subjected to flexure and axial load with almost no shear (Fig. 1). The test region in the non-prismatic stub specimens was subjected to a constant axial load and cyclic shear and flexure. Fig. 2 shows the idealization of a stub specimen. The test region of the column is in the vicinity of the stub. Table 1 lists details of the specimens that are discussed in this paper. Standard cyclic loading consisted of one cycle of deflection to $0.75\Delta_o$ followed by two cycles each to a displacement of $\Delta_o$, $2\Delta_o$, $3\Delta_3$, --- until the specimen could not maintain the axial load. In case of monotonic lateral loading, the test was continued until the specimen could not maintain axial load or the lateral load dropped to zero on the descending part of the lateral load - displacement curve.

For normal strength concrete ($f_c' \approx 30$ MPa) specimens, the lateral reinforcement ratio ($\rho_s$) required according to the seismic provisions of North American design codes ("Building" 1989, "Code" 1984) is approximately 1.6%. With the limited size of available steel bars and the maximum allowable spacing of 100 mm, the usual amount of lateral steel used in the columns that require confinement would be somewhat larger than that recommended in the codes.

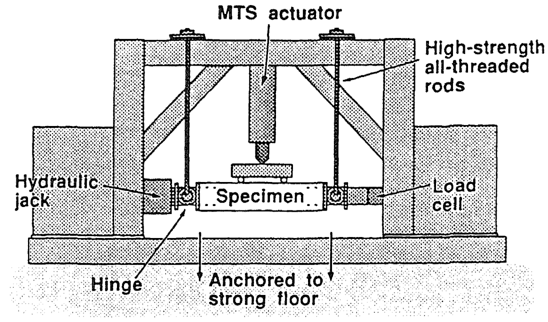



Figure 1. Test setup

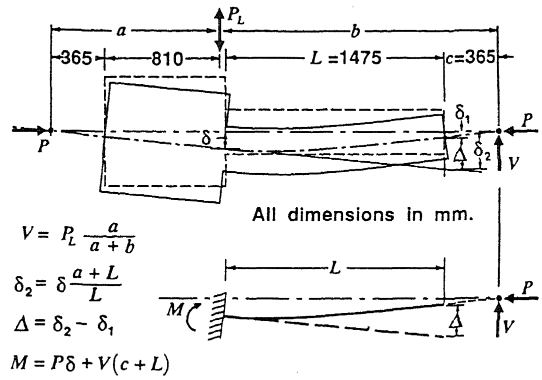
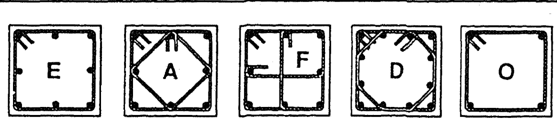



Figure 2. Idealization of stub specimen

Table 1 - Details of the tested specimens

| Specimen | Concrete strength (MPa) | Longitudinal steel No. & Size | Longitudinal steel $\rho$ (%) | Transverse steel Size & Spacing (mm) | Transverse steel $\rho_s$ (%) | $f_{yh}$ (MPa) | $\frac{P}{f_c' A_g}$ | $\frac{M_{max}}{M_{ACI}}$ |
|---|---|---|---|---|---|---|---|---|
| Prismatic Specimens | | | | | | | | |
| A-3 | 31.8 | 8-19 mm | 2.44 | 9.5 @ 108 | 1.68 | 490 | 0.61 | 1.23 |
| F-9 | 26.5 | 8-19 mm | 2.44 | 9.5 @ 95 | 1.68 | 490 | 0.77 | 1.25 |
| E-13 | 27.2 | 8-19 mm | 2.44 | 12.7 @ 114 | 1.69 | 483 | 0.74 | 1.01 |
| Non-Prismatic Specimens | | | | | | | | |
| ES-13 | 32.5 | 8-19 mm | 2.44 | 12.7 @ 114 | 1.69 | 464 | 0.76 | 1.40 |
| FS-9 | 32.4 | 8-19 mm | 2.44 | 9.5 @ 95 | 1.68 | 507 | 0.76 | 1.37 |
| AS-3 | 33.2 | 8-19 mm | 2.44 | 9.5 @ 108 | 1.68 | 507 | 0.60 | 1.37 |
| AS-17 | 31.3 | 8-19 mm | 2.44 | 9.5 @ 108 | 1.68 | 507 | 0.77 | 1.53 |
| AS-18 | 32.8 | 8-19 mm | 2.44 | 12.7 @ 108 | 3.37 | 464 | 0.77 | 1.70 |
| AS-19 | 32.3 | 8-19 mm | 2.44 | 9.5 @ 108 □ | 1.30 | 507 | 0.47 | 1.32 |
| | | | | 6 @ 108 ◇ | | 469 | | |



## RESULTS

Whereas failure in prismatic specimens occurred at the sections which carried maximum moment, the sections undergoing maximum moment did not fail in stub columns. The restraint provided by the stubs strengthened the adjacent critical sections and pushed the failure away from the stub. Fig. 3 shows the sketches of the extensively damaged regions in six stub specimens. In all the specimens, the failure initiated at a distance ranging from 225 to 325 mm away from the stub face and later extended toward the stub. It is believed that restraint from the stub reduced gradually due to the reduced stiffness of the region after the failure is initiated. At the section where the failure initiates the external restrain from the stub is minimal and the applied moment first exceeded the capacity at that location. It then follows that the moment capacity of that section should correspond to the section capacity in a companion prismatic specimen.

Fig. 4 shows a comparison of the moment-curvature behavior of similar sections in prismatic and non-prismatic stub specimens. Moment is normalized with respect to the theoretical capacity of the unconfined section ("Building" 1989) to facilitate comparison. Assuming the validity of the envelope curve concept, it is obvious that behaviors of similar sections in the two companion specimens are similar and that the sections where failure initiated are mostly free of the restraint from the stubs. In the three stub specimens shown in Fig. 4 the moment at the stub-column interface is approximately 14% to 17% higher than the moment at failed sections and the moment at the failed section in turn is approximately 16% to 20% higher than the unconfined moment capacity. With an increase in the lateral reinforcement ratio, the enhancement of moment capacity due to lateral steel in

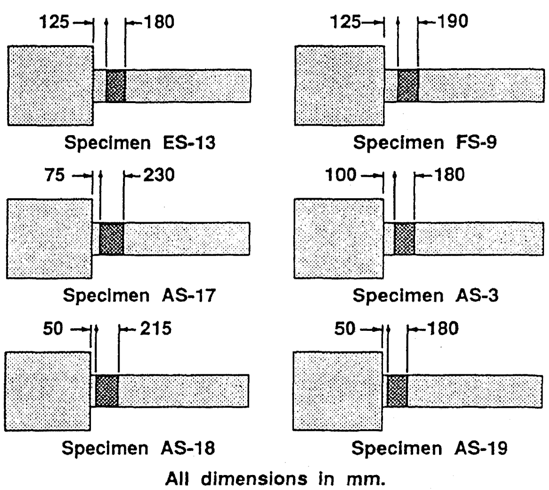



Figure 3. Extensively damaged regions in specimens

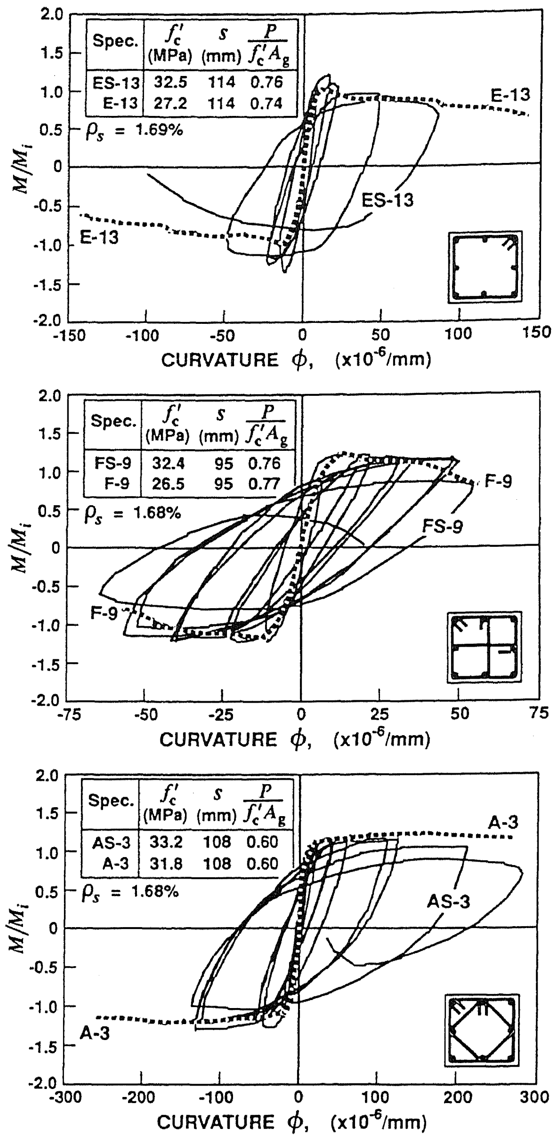



Figure 4. Effect of stub restraint

well-confined columns would be of a much larger magnitude. It should be noted that the higher moment at the critical sections adjacent to stub did not cause failure. The capacities of these sections are therefore larger than the maximum moment values listed in Table 1 and are unknown. It is believed that the moment enhancement due to stub restraining effect is dependent upon the relative sizes of column and stub. However, the region over which this effect is significant is approximately equal to the column section dimension. For a safe estimate of the design shear force calculated from end moment capacities, appropriately reduced column length should be used. For a typical column in a building, it is recommended that the column length should be conservatively reduced by two times the column section dimension for this purpose.

Other factors that are generally ignored in the design of confining steel in North American design codes are the level of axial load and steel configuration. Figs. 5 to 7 show the effects of these two variables on the behavior of specimens. Lateral load in Fig. 5 has been normalized with respect to $V_i$, the load (V in Fig. 2) required to cause the theoretical failure moment for unconfined section at the stub-column interface in the absence of axial load. The only significant difference between the two columns compared in Figs. 5 and 6 is the level of axial load. Higher axial load results in a somewhat larger moment enhancement over the theoretical capacity and a very significant reduction in ductility and energy absorption capacity. With a 28% increase in the axial load the section ductility and energy absorption capacity of the column is reduced by about 35%. Both the columns contained slightly higher lateral steel than that recommended by the ACI and CSA Codes ("Building" 1989, "Code" 1984). The axial loads on specimens AS-3 and AS-17 were 88% and 111% of the allowable loads ("Building" 1989). Considering the lateral support provided to the longitudinal steel bars, and tie spacing, these two columns can be classified as well-confined. Under an axial load exceeding the code limits the confined section and the member as a whole were capable of providing fairly ductile behavior which indicates that for lower levels of axial load, the amount of lateral steel in well-configured columns can be significantly reduced without inviting brittle failure.

Effect of steel configuration on column behavior is shown in Fig. 7. The three specimens compared in this figure contained almost equal amounts of lateral steel and exactly the same amount and distribution of longitudinal steel. Level of axial load and tie spacing are also similar in the three specimens. The type of lateral support provided to the middle bars is the only difference between the three specimens. The 90° hooks in specimen FS-9 are alternated along the column length as recommended by the codes ("Building" 1989, "Code" 1984). The poorest behavior among the three specimens was shown by specimen

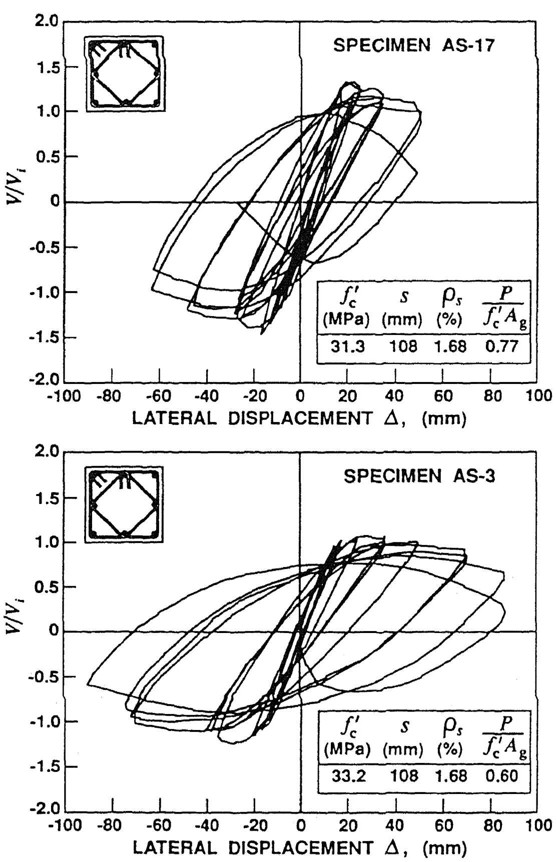



Figure 5. Effect of axial load on load-deflection behavior

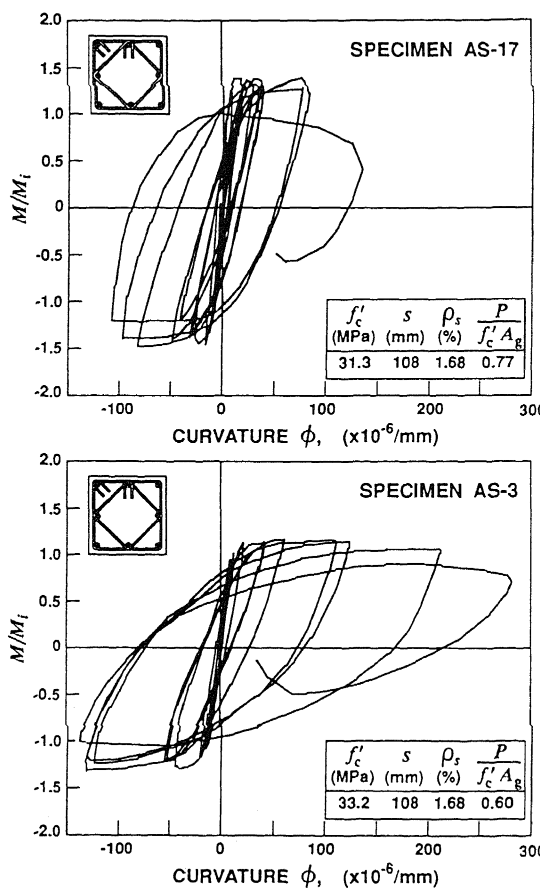



Figure 6. Effect of axial load on moment-curvature behavior

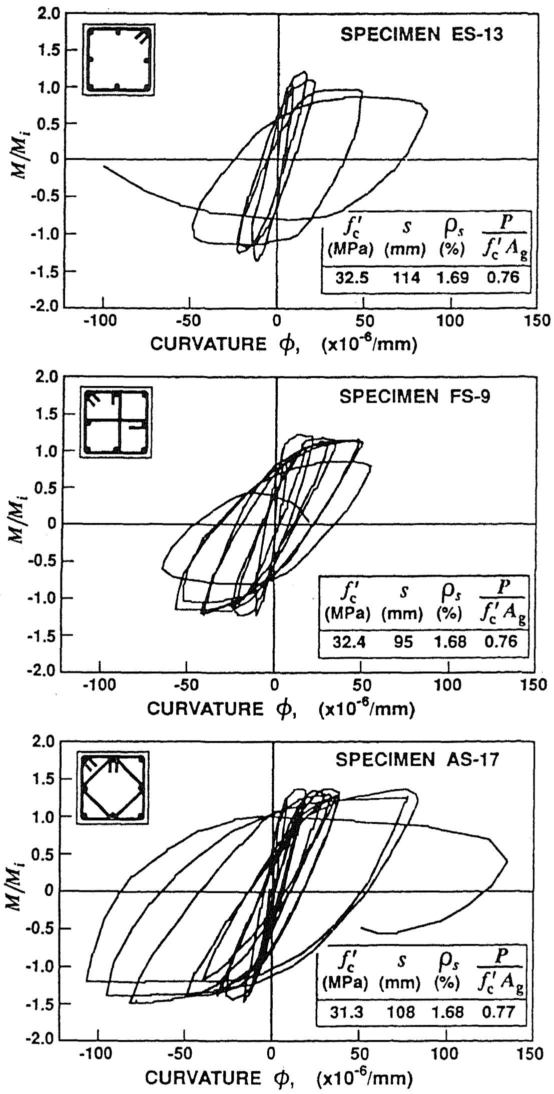



Figure 7. Effect of steel configuration

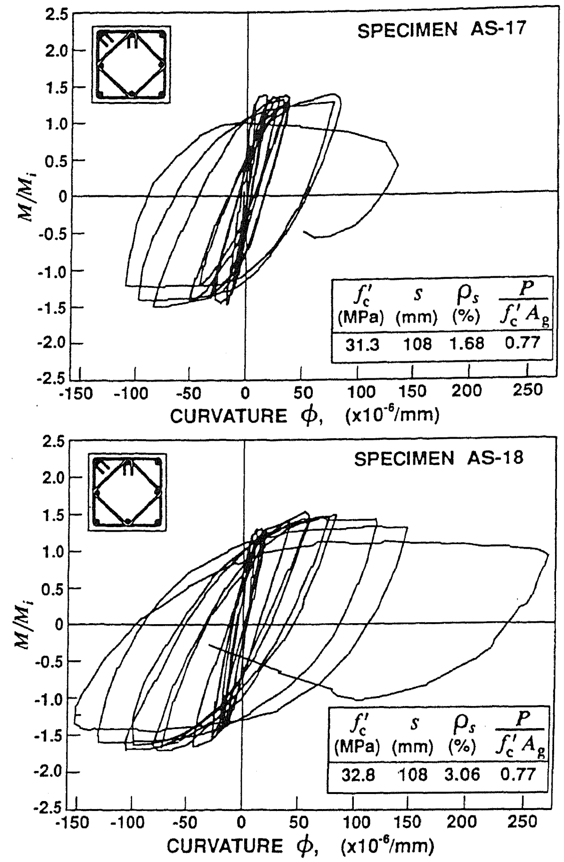



Figure 8. Effect of amount of lateral steel

ES-13 which could undergo only five complete cycles before it lost its ability to maintain the axial load. In contrast, specimen AS-17 underwent 12 complete load cycles before it lost its ability to maintain the axial load. Behavior of specimen FS-9 was quite different compared with that of specimen AS-17 although all the bars in both the specimens were supported by tie bends. For the first four cycles specimen FS-9 behaved in a reasonably ductile and stable manner but as the stress in a crosstie increased beyond about 60% of yield stress, the 90° hook started to open out in the absence of concrete cover in the eighth cycle followed by a quick buckling of the middle bar. Behavior of the section beyond this point deteriorated rapidly as a result of a loss of confinement. Based on several ductility parameters such as section and member ductility factors and energy absorbed by the section, the relative deformabilities of specimens ES-13, FS-9 and AS-17 are approximately in the order of 1, 2 and 3, respectively. The performance of 90° hooks under high axial load level is obviously questionable. Under low axial loads their performance has been observed to be quite satisfactory (Rabbat et al 1986, Khoury and Sheikh 1991).

Effect of lateral steel on column behavior can be evaluated from Fig. 8 in which two specimens similar in all respects except the amount of lateral steel are compared. An increase of about 82% in the amount of lateral steel improved the section's ductility and energy absorption capacity by about 100%. Enhancement in the moment capacity, however, is not as directly affected by an increase in lateral steel content. It should be noted that the level of axial load in both the specimens exceeded the allowable limit by about 12% and specimen AS-17 contained the amount of lateral steel just exceeding that required by the codes ("Building" 1989, "Code" 1984)

## CONCLUSIONS

1. Heavy stub provides additional confinement to the adjacent section and in the tests reported here increased section moment capacity by more than 20%. The failure is thus moved to section away from the stub where the stub restraining effect is minimal. The design shear should therefore be calculated based on the length between plastic hinges which may be approximately 2h smaller than the clear column length where h is the section depth.

2. Increased axial load reduces section and member ductility significantly. A 28% increase in axial load from $0.6f_c'Ag$ to $0.77f_c'Ag$ reduced section ductility by about 40%.

3. Distribution of longitudinal and lateral steel and the lateral support provided to the longitudinal bars play an important role in column behavior. Sections with unsupported middle bars behave very poorly compared with the sections in which the middle bars are supported by tie bends that are anchored inside the core. The 90° hooks not anchored in the core confine concrete and support longitudinal bars effectively only at small deformations. At large deformations these hooks open out and cause brittle failure particularly in column supporting large axial loads.

4. An increase in the amount of lateral reinforcement increases strength and ductility of a column section. Enhancement in ductility and energy absorption capacity was observed to be almost proportional to the increase in lateral steel contents but increase in the moment capacity was less than proportional to the increase in the amount of lateral steel.

5. In tests reported here an increase in the moment capacity of the order of 70% was observed due to the combined effects of stub restraint and lateral steel confinement. A similar increase in the design shear force should be considered for capacity design approach.

6. Since the North American design codes do not consider factors such as steel configuration, level of axial load and presence of stub adjacent to potential plastic hinge regions in the design of confining steel, the columns designed according to code provisions may under certain circumstances fail in a brittle manner when subjected to large inelastic deformations. In other situations code provisions are unnecessarily conservative.

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