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Flexural behavior of confined concrete columns subjected to high axial loads

C.C.Yeh & Shamim A.Sheikh
University of Houston, Texas, USA

ABSTRACT: Results from an experimental program involving sixteen confined concrete columns are reported in this paper. Experimental results are compared with the predictions from an analytical model proposed earlier. All the specimens were 12 inches (35 mm) square and 9 feet (2.74 m) long except one, which was 13 inches (330 mm) square. The specimens were tested to large inelastic deformations under flexural loads while subjected to constant axial loads in the range of 0.4 $f'_c A_g$ to 0.78 $f'_c A_g$. The tests were continued until after the flexural loads were reduced to zero on the descending part of the load-deflection behavior. In addition to the level of axial load, variables investigated for their effects on the behavior of the columns are: Detailing of longitudinal and lateral steel, including overlapping hoops and crossties; spacing; and the amount of lateral steel.

1 INTRODUCTION

In seismic design of framed structures, a prime consideration is the need to have the structure capable of deforming in a ductile manner to dissipate energy. Various building codes (ACI 318-83; CSA 1984; SEAOC 1980) aim at having plastic hinges form in the beams rather than in the columns by restricting the ratio of the sum of moment strengths of the columns to that of the beams at a connection in each principal plane [ΣM (cols)/ΣM (beams)]. This requirement, though it reduces the possibility of column hinging, does not ensure that plastic hinges would always form in beams (Park and Pauley 1975). Observations from various earthquakes indicated several failures in columns designed according to the code requirements. In the recent Mexico earthquake of 1985 (Rosenblueth and Meli 1986), one of the most common failure modes was the deterioration of concrete in columns and subsequent loss of load-carrying capacity. In an attempt to prevent column hinging, the New Zealand design code (NZ 1982) requires the ratio ΣM (cols)/ΣM (beams) to be larger than 2 at a joint, which is more stringent than most other codes. The corresponding ratio for the ACI Code is 1.2. Column hinging at the base of structures is, of course, relied upon to dissipate energy. It is, therefore, necessary to design columns such that they are capable of behaving in a ductile manner. It is well-established (Sheikh and Uzumeri 1980, Sheikh and Uzumeri 1982, Sheikh 1982, Sheikh and Yeh 1986) that confinement of core concrete with appropriate detailing of both longitudinal and lateral reinforcement can increase strength and ductility of concrete. Most of the previous experimental work involved testing of columns under concentric compression based on which an analytical model was proposed (Sheikh and Uzumeri 1980, Sheikh and Uzumeri 1982). The model was further extended to include both axial and flexural loads (Sheikh and Yeh 1986). Research reported here describes results from an experimental study in which columns were tested to large inelastic deformations under combined axial and flexural loads. Experimental results are also compared with the predictions from the analytical model.

2 TEST PROGRAM

The test program was designed to study the effects of several variables on the mechanism of confinement and the behavior of columns. Sixteen columns were tested

Table 1. Details of specimens.

Specimen	Concrete Strength (ksi)	Longitudinal Steel	ρ_1 (%)	Transverse Steel	ρ_s (%)	f_{yt} (ksi)	$\frac{P}{f'_c A_g}$
E4MM-1	4.45	8 #6	2.08	#4 @ 4"	1.74	70	0.40
E4MH-2	4.55	8 #6	2.44	#4 @ 4 1/2"	1.69	70	0.61
A3MH-3	4.61	8 #6	2.44	#3 @ 4 1/4"	1.68	71	0.61
F3MH-4	4.67	8 #6	2.44	#3 @ 3 3/4"	1.68	71	0.60
D3MM-5	4.53	12 #5	2.58	#3 @ 4 1/2"	1.68	71	0.46
F4MH-6	3.95	8 #6	2.44	#4 @ 6 13/16"	1.68	70	0.75
D1MH-7	3.80	12 #5	2.58	6 mm @ 2 1/8"	1.62	68	0.78
E4SH-8	3.76	8 #6	2.44	#4 @ 5"	0.84	70	0.78
F3MH-9	3.84	8 #6	2.44	#3 @ 3 3/4"	1.68	71	0.77
E1MH-10	3.81	8 #6	2.44	#3 @ 2 1/2"	1.68	71	0.77
A3SH-11	4.05	8 #6	2.44	6 mm @ 4 1/4"	0.77	68	0.74
F2SM-12	4.85	8 #6	2.44	6 mm @ 3 1/2"	0.82	67	0.60
E3MH-13	3.95	8 #6	2.44	#4 @ 4 1/2"	1.69	70	0.74
D3SH-14	3.90	12 #5	2.53	6 mm @ 4 1/4"	0.81	67	0.75
D3MH-15	3.80	12 #5	2.53	#3 @ 4 1/2"	1.68	71	0.75
A3SH-16	4.92	8 #6	2.44	6 mm @ 4 1/4"	0.77	81	0.60

ρ_1, ρ_s = volumetric ratio of longitudinal and lateral steel, respectively

f_{yt} = yield stress of lateral steel

f'_c = concrete strength from a standard cylinder

A_g = gross area of section

P = axial load

under monotonically increasing flexural loads while simultaneously being subjected to constant axial loads. The details of the specimens are given in Table 1 and Figure 1. All the columns were 12 inches (305 mm) square and 9 feet (2.74 m) long except Specimen E4MM-1, which was 13 inches (330 mm) square. After this first test, it was decided to use smaller section size so that, with the limited capacity of the test frame, the specimens could be tested under higher levels of axial load when the role of confinement became more pronounced in achieving ductile column behavior. Concrete cover to the center of the perimeter tie bar was 0.75 inch (19 mm) in all the columns. The two-point lateral load was applied to create a uniform-moment, shear-free test zone in the middle 3 feet (0.91 m) of the column length. Transverse reinforcement heavier than that specified in Table 1 for the test zones was provided outside the test region to avoid failure there.

The variables examined in the study include distribution of laterally supported and unsupported longitudinal bars, overlapping hoops and crossties, amount of lateral reinforcement, spacing of ties and axial load level. The axial load was applied to the column through the two hinges which were attached to the ends of the column. Each hinge consisted of five roller bearings, which allowed the rotation of the column with minimal restraint. The column was supported from these hinges as shown in Figure 2. The lateral load was applied using 150-kip (670 kN) MTS actuator. With the help of a function generator, the displacement rate was controlled at about .04 - .05 inch (1.0 - 1.3 mm) per minute up to about 80% of the maximum load on the descending part

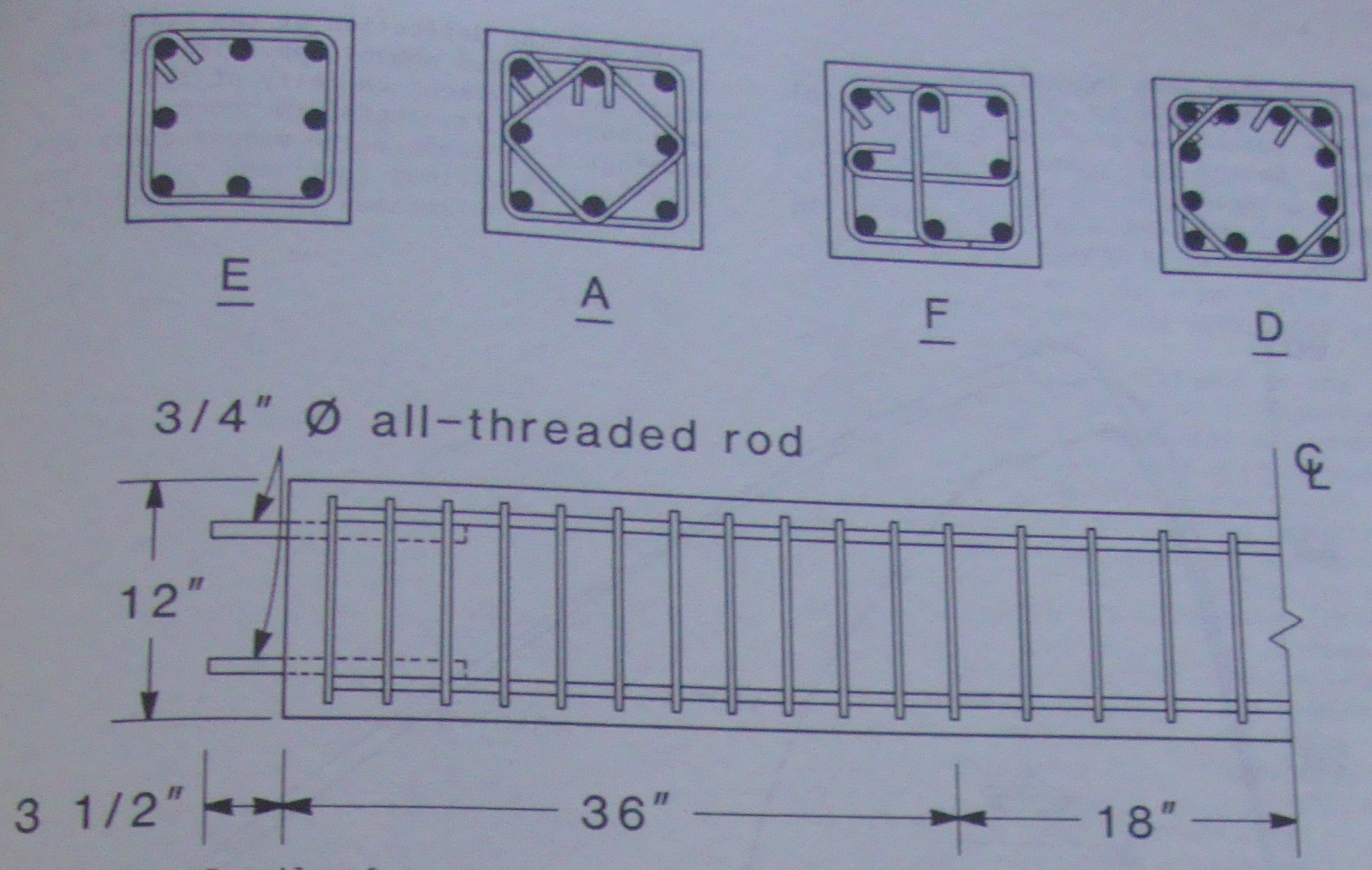

Figure 1. Details of specimens

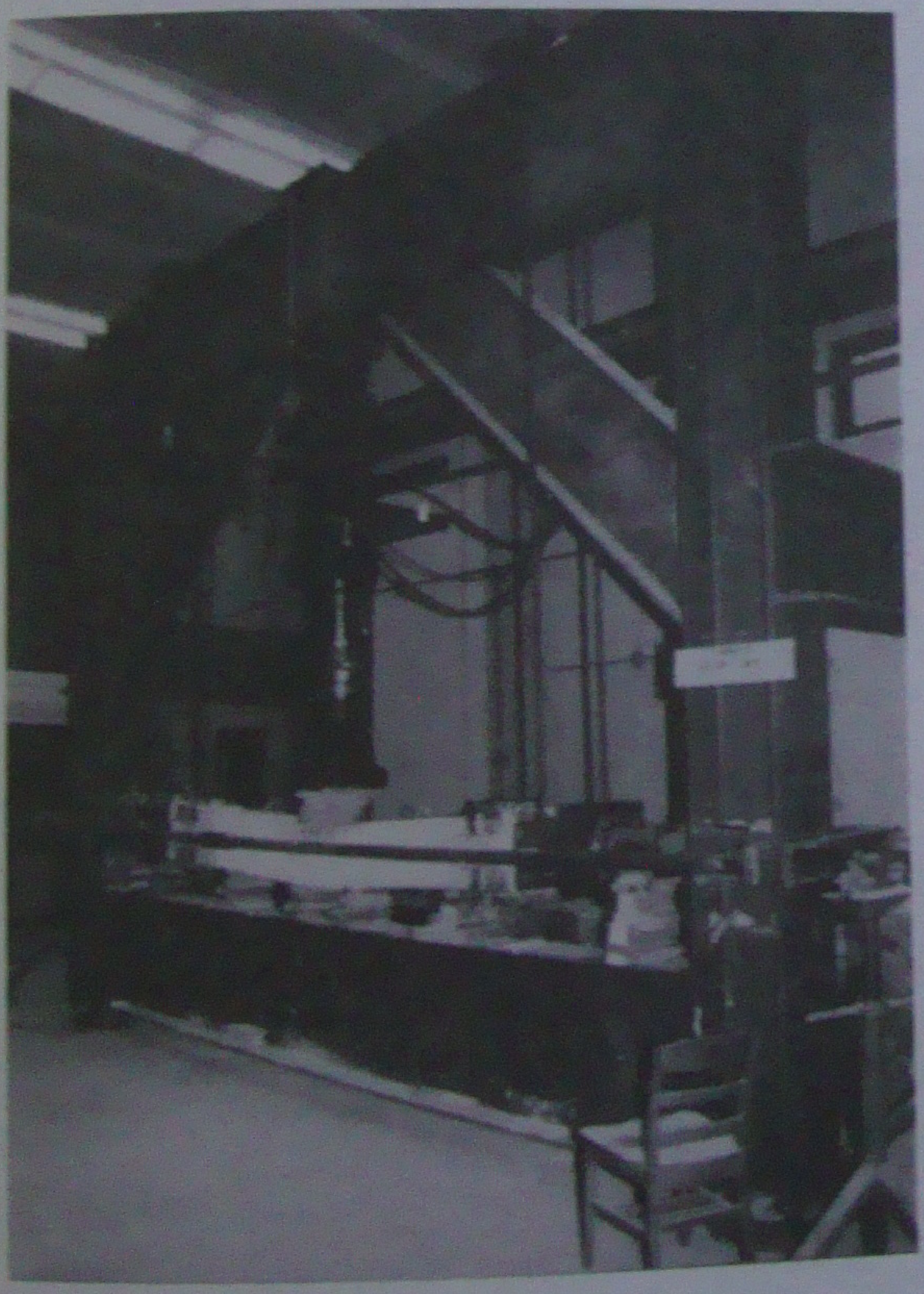
Figure 2. Test set-up

of the curve. The rate was increased by a factor of 2 to 3 in the latter part of the test. Most of the tests were terminated when the lateral load dropped to zero or a small fraction of the maximum. It should be noted that the axial load remained constant throughout this period. In the case of a few columns, the tests were continued beyond the point when lateral load dropped to zero and the axial load dropped below the specified level.

Instrumentation of the specimens was aimed at obtaining the moment-curvature relationships of the critical sections and measuring strains in steel and concrete required to compare the analytical and experimental results. Six LVDT's (Linear Voltage Differential Transducers) were used to measure longitudinal deformations along the depth of the section to calculate curvature. Strain in longitudinal and lateral steel was measured with the help of electric strain gages. Dial gages and LVDT's were used to measure deflections along the length of the specimens. Readings from the strain gages and the LVDT's were obtained with the help of an HP data acquisition system.

3 RESULTS AND DISCUSSION

During the initial phase of a test, the moment experienced by the critical section primarily came from the lateral load. At the time when peak lateral load was applied, about 15% to 20% of the critical section moment was produced by axial force. Beyond this point the contribution of the lateral load towards the section moment reduced gradually to zero when, in several cases, the product of the axial force and the deflection at the critical section produced moment approximately equal to the moment capacity of the sections. It is, therefore, more meaningful to compare the moment-curvature relations of various specimens rather than lateral load-deflection curves. Ductility

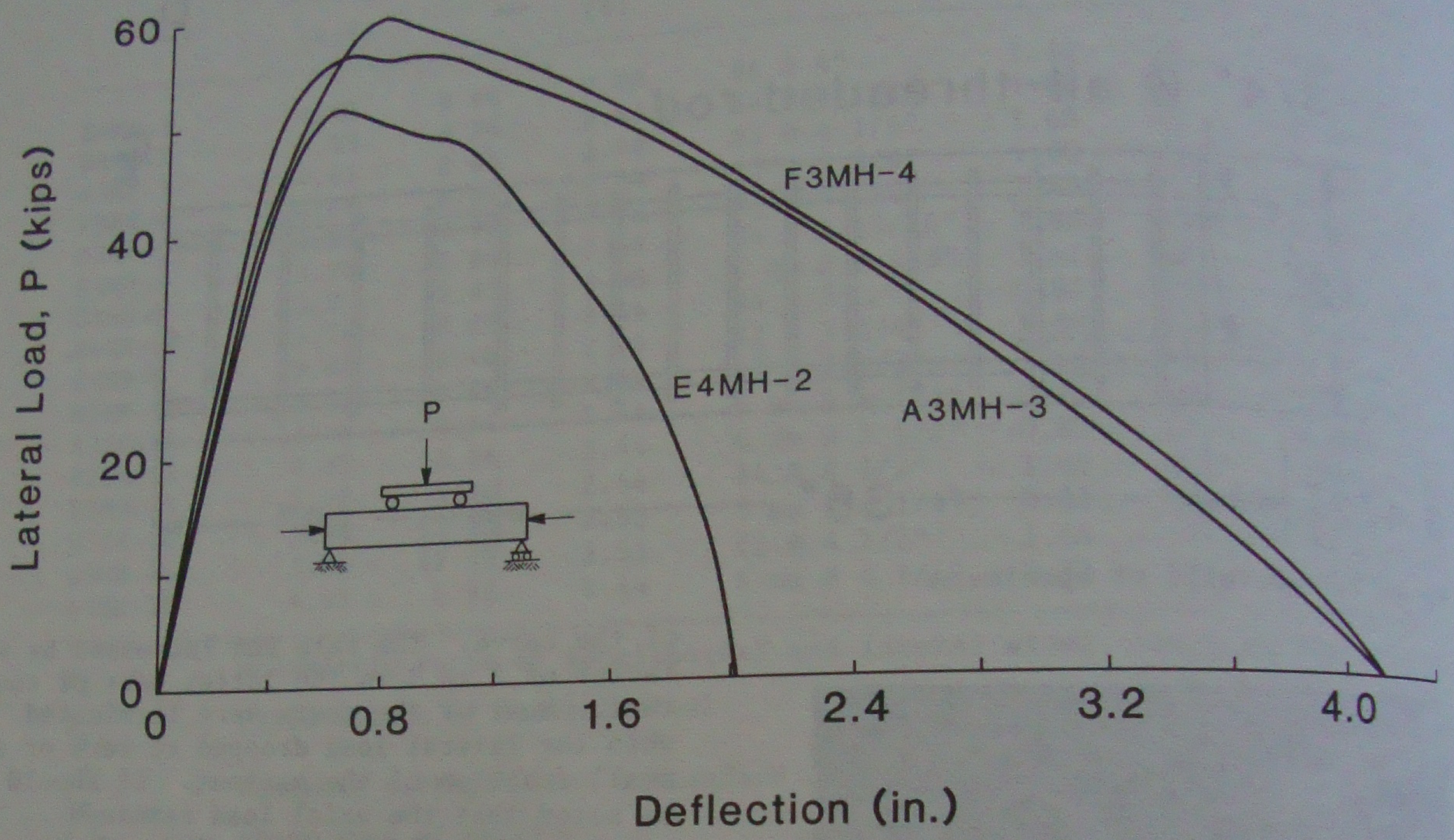

Figure 3. Effect of steel configuration on load-deflection behavior

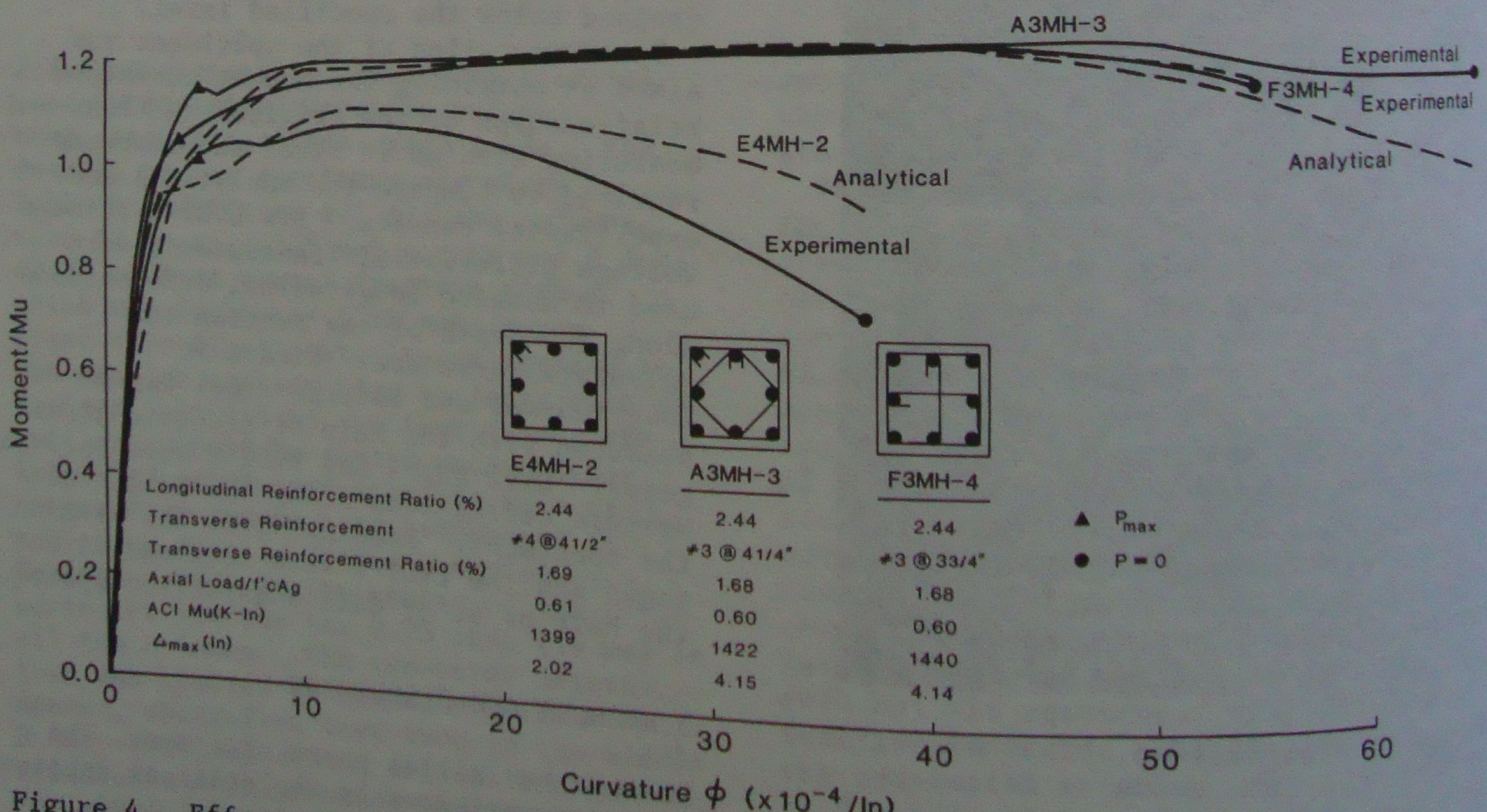

Figure 4. Effect of steel configurations

and toughness, however, can be compared using load-deflection curves.

For the first comparison, both the load-deflection and moment-curvature relations are given in Figures 3 and 4 to evaluate the effects of steel configuration. All three of the specimens -- E4MH-2, A3MH-3 and F3MH-4 -- have equal amounts of longitudinal and lateral reinforcements and similar tie spacing and were tested under 400 kips (1780 kN) of axial load (which remained constant throughout the test). The obvious difference in the behavior of columns can be attributed to the better confinement provided by better distribution of laterally supported longitudinal bars. In all the specimens strain in the lateral steel in the compression zone of the critical section increased slowly with load until crushing of concrete started at the top. This indicated a lack of need for concrete confinement. The maximum lateral load approximately coincided with this occurrence. Beyond this point tie strain increased rapidly, resulting in the yielding of steel. The column deflection was the lowest in the case of Configuration A and highest for Configuration E when steel in the outer tie yielded. Lateral load dropped rapidly in the case of Specimen 2 (Configuration E) after yielding of tie steel, which accompanied crushing of core concrete. The test was terminated when the lateral load dropped to zero and the specimen was unable to sustain the axial load.

Unlike Configuration E, the inner ties in Configurations A and F provided the necessary restraint to the middle longitudinal bars and improved confinement of concrete, which resulted in a gradual drop in the lateral load beyond peak. Stress in the inner ties was very low before yielding of the outer ties and increased comparatively rapidly thereafter as the test progressed. This continued lateral confinement appears to be responsible for increased moment capacity and ductility. After yielding of the inner ties, the moment dropped gradually. The test was continued until the lateral load dropped to zero. Specimen 3 was quite stable at this point and maintained the axial load which was only slightly less than the originally applied 400 kips (1780 kN). At this stage Specimen 4 was able to carry less load compared with Specimen 3 but was in a stable condition. The 90° hooks in Specimen 4 showed signs of opening but were able to provide effective confinement. However, it is believed that under reversed cyclic loading Configuration A would provide better confinement compared with Configuration F.

An analytical model, details of which are available elsewhere (Sheikh and Uzumeri 1982, Sheikh and Yeh 1986), was used to predict the behavior of columns shown in Figure 4. The comparisons between experimental and analytical curves show good agreement.

Figure 5 shows a comparison of the moment- curvature behavior of Specimens F4MH-6 and F3MH-9. The only difference between the two specimens which influences the behavior of confined concrete is tie spacing. Smaller tie spacing results in higher moment capacity. In Specimen 6, tensile stress in the perimeter tie in the compression zone was only about 35% of the yield stress just before the maximum lateral load was applied. The stress in the vertical crosstie at that moment was only 20% of the yield stress. The corresponding stresses in both ties in Specimen 9 were equal to about 65% of the yield stress. All the tie steel subsequently yielded in both the specimens. The crossties initially provided the necessary restraint to prevent premature buckling of middle longitudinal bars. A rapid drop in the moment capacity in the last part of the moment-curvature curves was caused by a loss of confinement due to the opening of the 90° hooks in both specimens. A reduction of steel strain in the crosstie was observed at this stage. This phenomenon occurred at a lower curvature value in Specimen 9 (smaller tie spacing) than in Specimen 6 due to the fact that crossties yielded earlier in Specimen 9. In both the specimens an extension of eight times the bar diameter (d_b) was used beyond the hooks compared with the minimum 6 d_b recommended by the ACI Code (ACI 318-83).

A comparison between analytical and experimental results indicates that, for large spacing, the model predicts the behavior reasonably well; but, for smaller spacing, the analytical model overestimates strength and ductility. It appears that, under high axial load levels, the 90° hooks are not as effective as assumed in the model.

Effect of axial force on column behavior is shown in Figure 6. Two identical columns of Configuration E were tested under different levels of axial load. Although the difference in the axial load levels (0.6 $f'_c A_g$ and 0.74 $f'_c A_g$) was not so great, the effects on the column behavior are very significant. A total lack of the

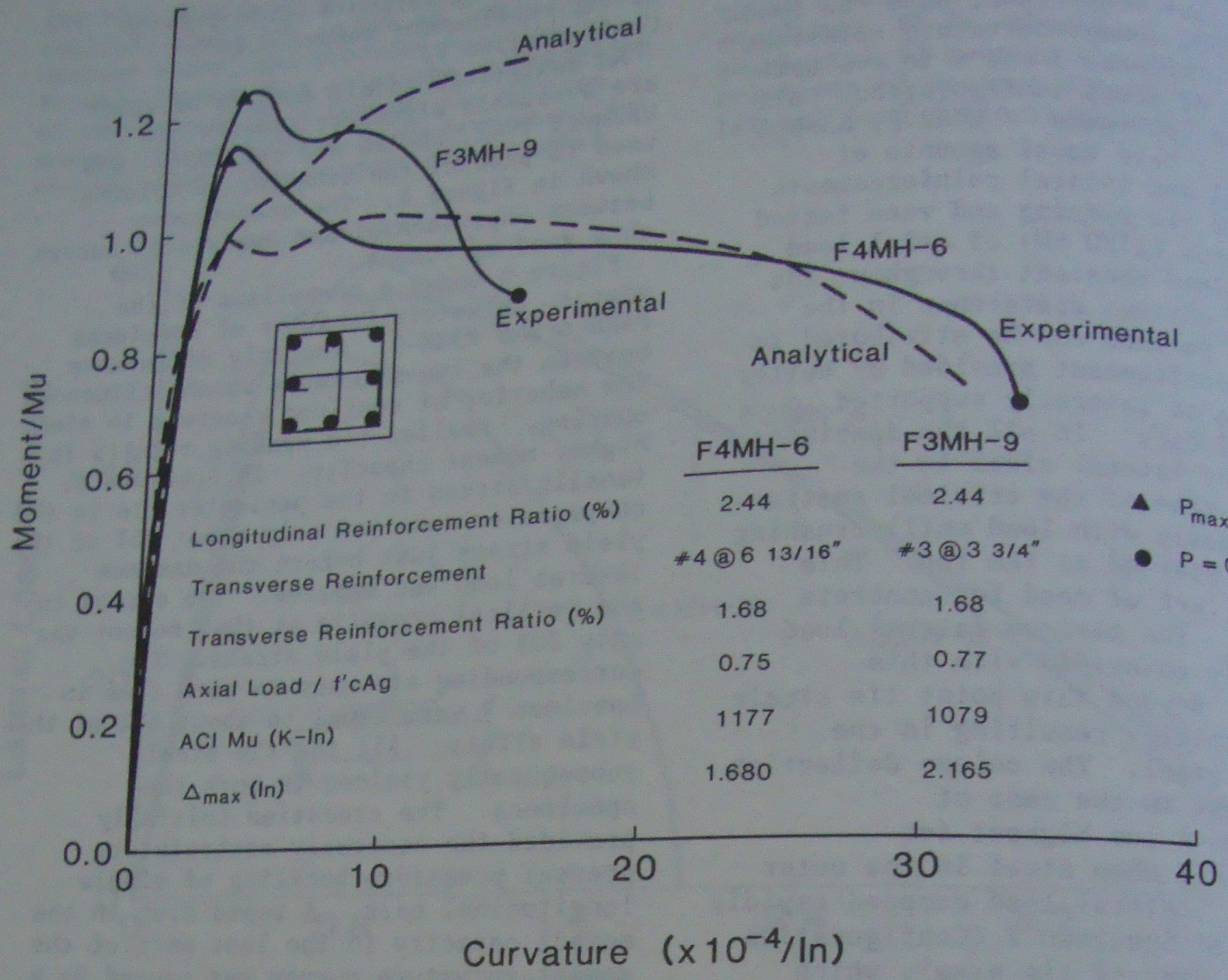

Figure 5. Effect of tie spacings

section ability to maintain the moment capacity after crushing of concrete started in Column E3MH-13 (high axial load) indicates a brittle failure. As in Specimen 2, behavior of which has been discussed earlier, tensile strain in the tie steel in the compression zone of Specimen 13 was very low before crushing of concrete started. Crushing of concrete cover resulted in a reduction in the moment capacity, which could not be compensated by the confinement of concrete even though the tie strain indicated yielding of steel. Figure 6 demonstrates the detrimental effects of the high axial load on the behavior of columns which may be classified as well confined for low to moderate levels of axial loads. This effect is not addressed in many design codes' provisions for confining steel (ACI 318-83; CSA 1984; SEAOC 1974). Predictions from the analytical model, also shown in Figure 6, slightly overestimate moment capacity at large deformations. The probable explanation is given in the following. In sections with E configuration, the sides of the ties were observed to significantly bend outward. The strain measured by electric strain gages and used in the analytical model would, therefore, overestimate the contribution of tie steel in confining the concrete.

Figure 7 shows the effect of the amount of lateral reinforcement on the column behavior. The only significant difference between Specimen 3 and Specimen 16 which influences the behavior is the amount of tie steel. Specimen 16 has about one-half of the amount of lateral steel in Specimen 3. The almost identical ascending part of the curves indicates that ties do not significantly influence the section behavior prior to the crushing of concrete cover. The variation of tie strain with load was very similar in both the specimens. The first significant drop in the moment in Specimen 16 was caused by crushing of concrete cover.

Yielding of perimeter ties followed after concrete crushing in both the specimens, but this did not cause a significant drop in the moment capacity. Yielding of inner ties resulted in a more

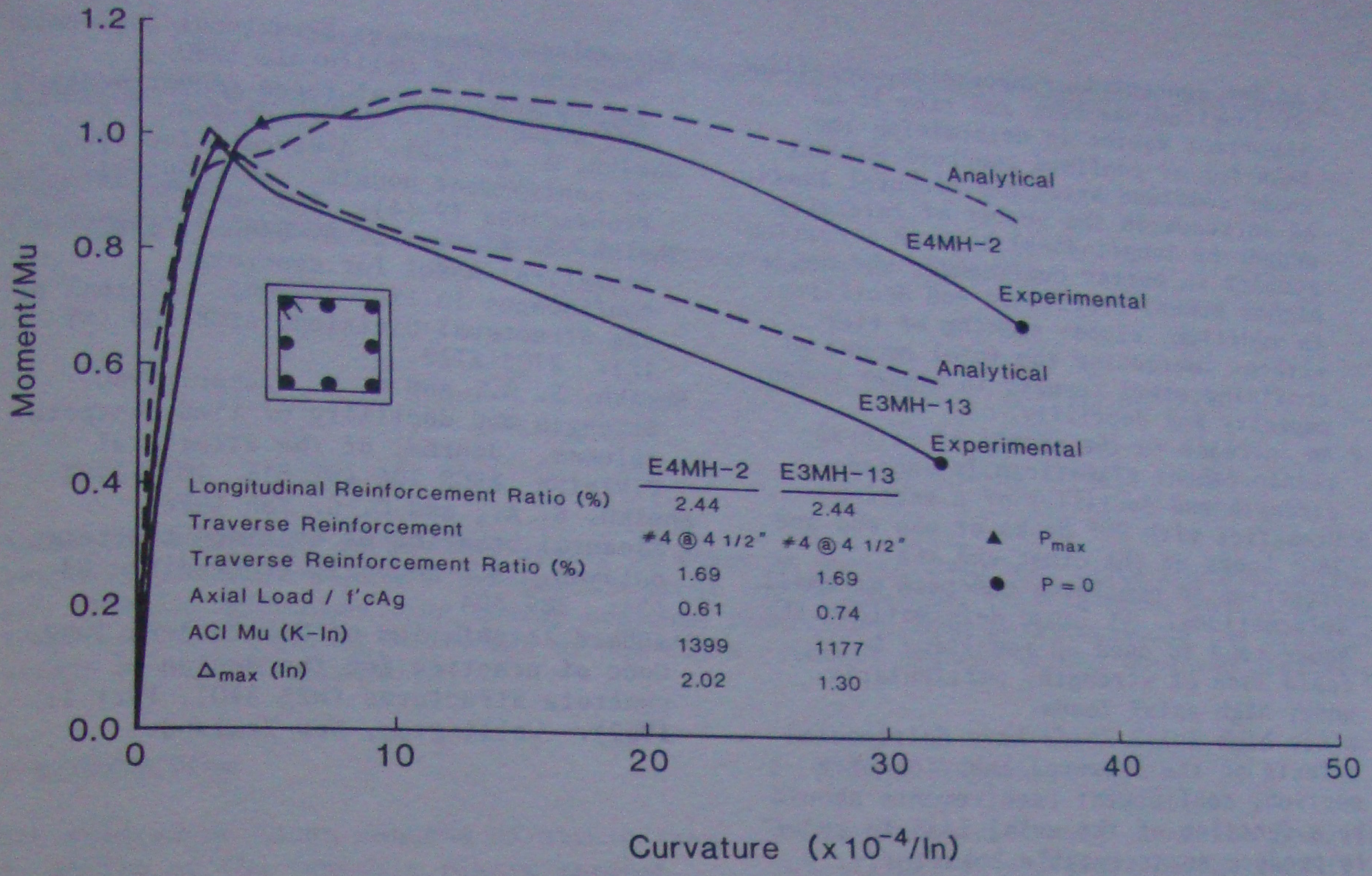

Figure 6. Effect of axial load level

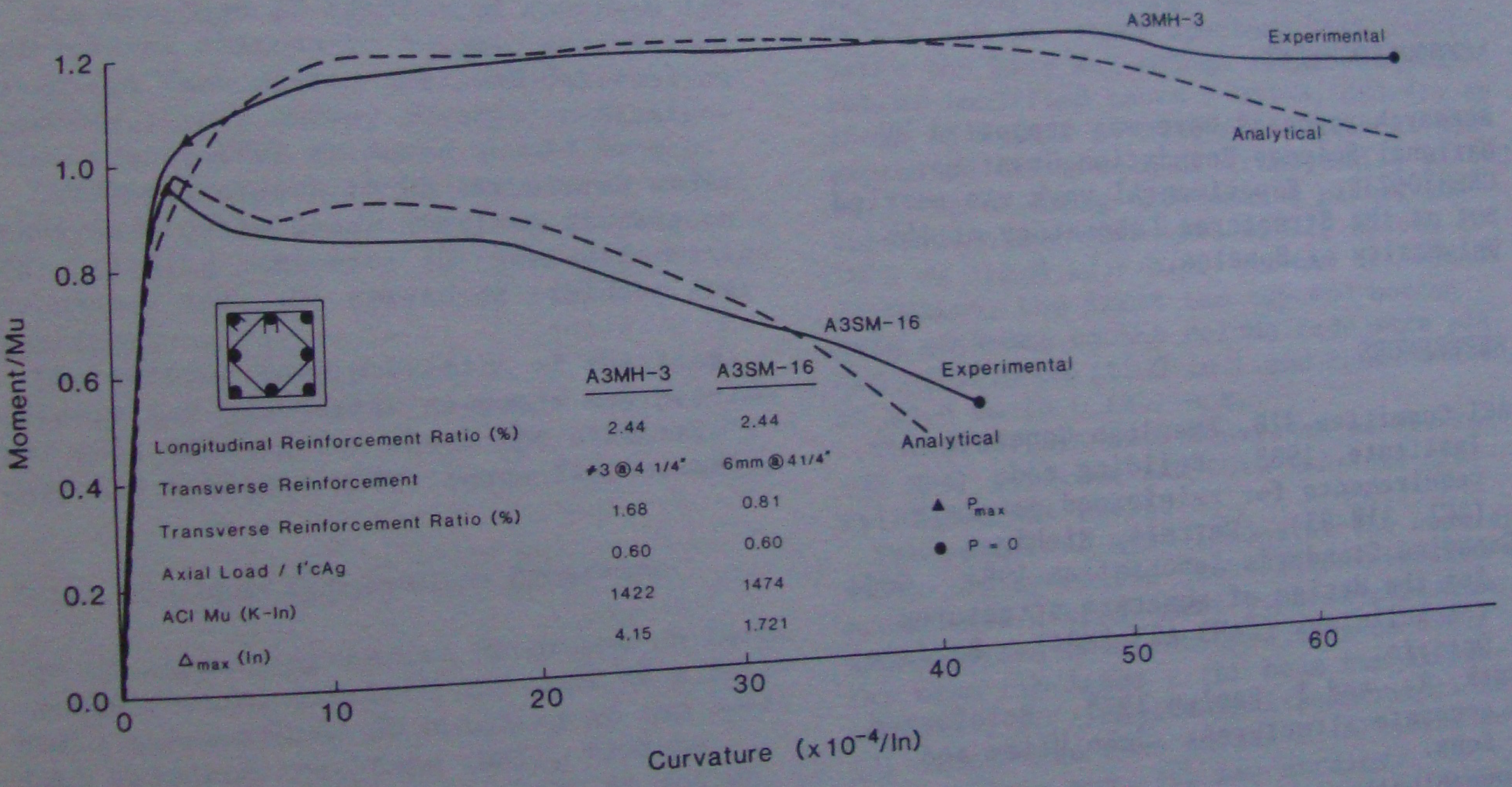

Figure 7. Effect of amount of lateral reinforcement

rapid drop in moment in the specimen with lower amount of tie steel. Figure 7 also shows the analytical curves, which compare well with the test results.

4 CONCLUSIONS

The following conclusions can be drawn from the results of the study reported in this paper:

1 As for concentric compression, detailing of longitudinal bars and ties is an important factor in determining the behavior of confined concrete columns under combined axial and flexural loads. An increase in the number of laterally supported longitudinal bars in a section results in better confinement and hence higher moment resistance and ductility. In addition, closer spacing of ties without increasing the total amount of confining steel results in higher moment capacity and ductility.

2 An increase in the amount of lateral reinforcement significantly enhances strength and ductility of a section.

3 Crossties with 90° hooks at one end and 180° hooks at the other end are effective in confining concrete at small deformations. At large deformations the hooks tend to open up resulting in a rapid loss of strength, particularly under high axial loads.

4 Since high axial loads have detrimental effects on the flexural behavior of a section, confinement requirements should be a function of the axial load in order to produce an acceptable behavior.

5 An analytical model presented earlier predicts the experimental results reasonably well.

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REFERENCES

ACI Committee 318, American Concrete Institute, 1983. Building code requirements for reinforced concrete (ACI 318-83). Detroit, Michigan.

Canadian Standards Association 1984. Code for the design of concrete structures for buildings (CAN3-A23.3M84). Rexdale, Ontario.

Park, R., and T. Pauley 1975. Reinforced concrete structures: John Wiley and Sons.

Rosenblueth, E., and R. Meli 1986. The 1985 earthquake: causes and effects in Mexico City. Concrete International, Design and Construction, ACI, 8 (5): 23-24.

Seismology Committee, Structural Engineers Association of California 1980. Recommended lateral force requirements and commentary. San Francisco.

Sheikh, S. A. 1982. A comparative study of confinement models. ACI Journal, Proceedings 79 (4): 296-306.

Sheikh, S. A., and S. M. Uzumeri 1982. Analytical model for concrete confinement in tied columns. Journal of the Structural Division, ASCE 108 (ST 12): 2703-2722.

Sheikh, S. A., and S. M. Uzumeri 1980. Strength and ductility of tied concrete columns. Journal of the Structural Division, ASCE 106 (ST 5): 1079-1102.

Sheikh, S. A., and C. C. Yeh 1986. Flexural behavior of confined concrete columns. ACI Journal, Proceedings 83 (3): 389-404.

Standard Association of New Zealand 1982. Code of practice for the design of concrete structures (NZS 3101, Part 1: 1982). Wellington, New Zealand.