Confined concrete columns with varied concrete strength

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ABSTRACT

Results from a select group of column specimens which are part of an extensive research program in the area of concrete confinement, are presented in this paper. The prismatic specimens were 305 mm square and 2.44 m or 2.74 m long. The non-prismatic specimens were 305 mm square and 1.45 m long with a 510 x 760 x 810 mm stub. The target values for concrete strength were 30 MPa and 60 MPa. A total of 24 specimens were tested under monotonic or cyclic flexure and shear while simultaneously subjected to large constant axial load. The objectives of this research are to evaluate the performance of confined concrete members as influenced by concrete strength, presence of stub, type of loading, amount of lateral steel and distribution of steel. Confinement provisions from the design codes are also critically evaluated in the light of the test results.

INTRODUCTION

In a framed structure during a severe earthquake, it is preferable to restrict the inelastic deformations to the beams in general to ensure structural stability and to maintain its vertical load carrying capacity. Accordingly, the concept of "strong column - weak beam" is suggested in most design codes to force the plastic hinges in beams rather than in columns. However, with uncertain earthquake demands, the occurrence of plastic hinges in columns cannot be avoided completely (Paulay 1986). Therefore, the potential plastic hinge regions of columns must be detailed for ductile behavior which can be achieved by confining the concrete effectively. Although the primary purpose of confining concrete is the improvement in ductility, confinement also enhances concrete strength significantly. The overall behavior of a member may not improve due to confinement if the length of the plastic hinge is underestimated or shear reinforcement is designed based on an underestimated flexural capacity. An accurate evaluation of the section behavior is therefore necessary for safe design.

CURRENT RESEARCH

The work reported here is part of an extensive research that started with a study of confinement mechanism under concentric compression (Sheikh and Uzumeri 1980). In the current phase of research, the details of which are listed in Table 1. The first experimental work involved large-size specimens, the details of which are listed in Table 1. The first

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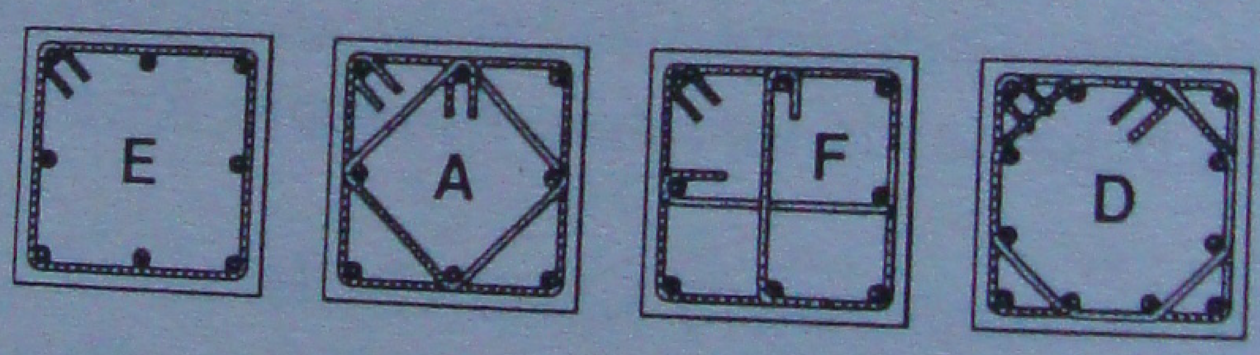
Table 1 - Details of the Test Specimens

Spec.	Concrete strength (MPa)	Longitudinal steel: No. & Size	Longitudinal steel: ρ (%)	Transverse steel: Size & Spacing (mm)	Transverse steel: ρ_s (%)	Transverse steel: f_{yh} (MPa)	$\frac{P_a}{f'_c A_g}$	$\frac{M_{max}}{M_{ACI}}$
Prismatic Specimens								
E-2	31.4	8-19 mm	2.44	12.7 @ 114	1.69	483	0.61	1.08
A-3	31.8	8-19 mm	2.44	9.5 @ 108	1.68	490	0.61	1.23
F-4	32.2	8-19 mm	2.44	9.5 @ 95	1.68	490	0.60	1.22
D-5	31.2	12-16 mm	2.58	9.5 @ 114	1.68	490	0.46	1.26
F-6	27.2	8-19 mm	2.44	12.7 @ 173	1.68	483	0.75	1.15
D-7	26.2	12-16 mm	2.58	6 @ 54	1.62	469	0.78	1.22
E-8	25.9	8-19 mm	2.44	9.5 @ 127	0.84	483	0.78	0.96
F-9	26.5	8-19 mm	2.44	9.5 @ 95	1.68	490	0.77	1.25
E-10	26.3	8-19 mm	2.44	9.5 @ 64	1.68	490	0.77	1.10
A-11	27.9	8-19 mm	2.44	6 @ 108	0.77	469	0.74	0.97
F-12	33.4	8-19 mm	2.44	6 @ 89	0.82	462	0.60	0.98
E-13	27.2	8-19 mm	2.44	12.7 @ 114	1.69	483	0.74	1.01
D-14	26.9	12-16 mm	2.58	6 @ 108	0.81	469	0.75	1.01
D-15	26.2	12-16 mm	2.58	9.5 @ 114	1.68	490	0.75	1.17
A-16	33.9	8-19 mm	2.44	6 @ 108	0.77	558	0.60	0.95
E-13H	57.6	8-19 mm	2.44	12.7 @ 114	1.69	464	0.63	1.43
F-9H	58.3	8-19 mm	2.44	9.5 @ 95	1.68	507	0.64	1.30
A-17H	59.1	8-19 mm	2.44	9.5 @ 108	1.68	507	0.65	1.30
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 6 @ 108	1.30	507 469	0.47	1.32



letter in specimen designations refers to the steel configuration shown in the sketches included in Table 1. All the prismatic specimens were 305 mm square. The normal strength concrete specimens were 2.74 m long and the length of high-strength concrete specimens was 2.44 m. The prismatic specimens were tested under constant axial load and two lateral point loads such that the middle 0.91 m length of the specimens was free from shear due to lateral loads (Figure 1). The prismatic normal strength concrete specimens were tested under monotonically increasing flexural deformations until the lateral loads dropped to zero. The high-strength concrete specimens were subjected to cyclic lateral loads until the specimens could not maintain the axial load.

The non-prismatic specimens were 305 x 305 x 1450 mm columns with 510 x 760 x 810 mm stubs and were tested under constant axial load and cyclic point lateral load applied on the stub near the column-stub junction. The critical section adjacent to the stub was thus subjected to a constant axial load and cyclic shear and flexure. Standard cyclic loading consisted of one cycle of deflection to $0.75\Delta_o$ followed by two cycles each to a displacement of Δ_o, $2\Delta_o$, $3\Delta_o$, --- until the specimen could not maintain the axial load. The Δ_o is the estimated deflection at the critical section that causes curvature ϕ_o at that section. The ϕ_o is the curvature corresponding to the maximum unconfined section moment on a straight line joining the origin and a point corresponding to about 65% of the maximum moment. In the case of Specimens E-13H and F-9H, the upward displacement could not be effectively controlled due to malfunctioning of the loading system. A symmetrical cyclic loading was therefore not achieved. It is, however, believed that the envelope curves are relatively independent of the load excursions, and can be compared with the results from other specimens to study different variables.

For normal strength concrete ($f_c' \simeq 30$ MPa) specimens, the lateral reinforcement ratio (ρ_s) required according to the design codes' seismic provisions ("Building" 1983; "Code" 1984; "Recommended" 1980) is approximately 1.4% and for high-strength concrete ($f_c' \simeq 58$ MPa) specimens, this ratio is approximately 2.6%. Nominal yield strength of 400 MPa for steel was used to calculate ρ_s. With the actual yield strength of steel, the corresponding ρ_s values are approximately 1.2% and 2.1%, respectively, for normal and high strength concrete specimens.

RESULTS

The section moment capacities, non dimensionalized with respect to M_i, are listed in Table 1 for all the specimens. The M_i is the theoretical moment capacity of a section based on the concrete stress block suggested in the ACI and CSA design codes. Several points should be considered when the moment capacities of different sections are compared. The moment capacities reported are the ones experienced by the critical sections in the specimens. In the case of prismatic specimens failure occurred at the critical sections. In the case of non-prismatic specimens, the failure in the column occurred approximately 150 mm to 300 mm away from the critical section that was adjacent to the stub. Since the critical sections in the non-prismatic specimens did not fail, their capacities would be higher than the moment values shown in Table 1. In several specimens, the entire cover concrete was effective in compression when the section carried the maximum moment. This was especially the case for specimens which were not well-confined. The three high-strength concrete specimens fall in this category.

In several prismatic, normal-strength concrete specimens, the moment capacity was less than the theoretical capacity, M_i. All these specimens contained approximately 0.8% lateral reinforcement ratio and were subjected to large axial loads. It appears that the strength of concrete in these specimens that can be used to calculate section strength under axial load and flexure, is less than the concrete strength in a standard cylinder.

Results from a select group of specimens are shown in Figures 2 to 7 in the form of lateral load vs. deflection and moment vs. curvature curves. For ease of comparison between specimens made of normal strength and high-strength concretes, the lateral load is non-dimensionalized with respect to P_i, the load required to produce M_i at the critical section without considering P-Δ effect and the moment axis is non-dimensionalized with respect to M_i. From a comparison of specimens in Figures 2 to 5 it appears that the cover concrete in the case of high-strength concrete specimens is more effective initially and spalls off more rapidly than in normal-strength concrete specimens. Crushing strain in high strength concrete is larger than that in normal strength concrete and at that strain confinement of core becomes somewhat effective particularly because of internal cracking due to cyclic loading, possibly resulting in higher section capacity. Due to relatively larger lateral strain in lower strength unconfined concrete, the separation between the restrained core and the cover may result in weaker than normal strength of cover concrete. Higher section strength before cover spalling in the case of Specimen E-13H can perhaps be attributed to the fact that the separation of cover concrete from the core is less disruptive in Configuration 'E' than in other configurations.

A comparison of specimens of Configurations E and F in Figures 2 to 5 shows that a more efficient confinement is achieved if all the longitudinal bars are supported by tie bends. A rapid drop in the section capacity of Specimens F-9H and F-12 during the later part of the tests was caused by the opening of the 90° hooks (Sheikh and Yeh 1990). The combination of high axial load and small lateral reinforcement ratio results in this type of failure which can be prevented by the use of internal ties such as those used in Configuration A (Figure 6). The adverse effects of high axial load on ductility can also be observed by comparing the behavior of Specimens F-4 and F-9. The beneficial effects of increased amount of lateral reinforcement are obvious from a comparison of Specimens F-4 and F-12.

In the case of Specimens E-2, F-4 and F-12 the axial load was about 10-12% below the limit allowed by the ACI codes. For high-strength concrete specimens (E-13H and F-9H), however, applied axial load exceeded the code limits by about 10% although for all five specimens the index $P/f_c'A_g$ was approximately 0.62 and the ratio between the concrete stress caused by the axial load and f_c' was also constant at about 0.53. A limit on axial load that causes lower concrete stresses in high strength concrete columns is quite logical in view of the brittle nature of this material and the adverse effects of high axial load on ductility. For the same absolute amount of lateral steel, ductility in higher strength concrete specimens is lower indicating that the required amount of confining steel should be dependent on concrete strength; however, not for the purpose of compensating for the loss of strength due to cover spalling but to maintain the integrity of the core to provide ductile behavior. It appears that the required amount of steel is less than proportional to concrete strength.

For Specimens FS-9 and F-9, the moment-curvature relationships of the sections where failure occurred, are provided in Figure 7. The failure section was approximately 200 mm away from the critical section in Specimen FS-9. Moment at the critical section which did not fail was approximately 10% larger than the moment at the failed section indicating that the stub provided significant restraint to the adjacent critical section. The similarity in the moment-curvature behavior between the two specimens indicates that the effect of stub restraint on the failed section was minimal. In both the specimens, ultimate failure was caused by the opening of the 90° hooks. Behavior of Specimen FS-9 appears to provide healthy energy absorption properties and it is plausible to assume that Specimen F-9 has similar characteristics.

Amount of lateral reinforcement in Specimens F-4 and F-9 meets the code requirements for seismic design. Specimens F-12 and A-16 contain the amount of lateral reinforcement which is about 60% of that required for seismic design. A comparison of the behavior of these specimens in Figure 6 shows that column design according to code requirements may either be too conservative (Specimen F-4) or unsafe (Specimen F-9). Behavior of Specimens F-12 and A-16 that violate the code requirements should also be acceptable if the behavior of Specimens F-9 and FS-9 is acceptable. The expected performance of a section and a member, distribution of longitudinal and lateral steel and the level of axial load must therefore be considered, in addition to other parameters, in the design of confining steel.

CONCLUSIONS

The concept of providing confining steel to compensate for the loss of load carrying capacity of cover concrete does not provide a sound basis for design. Columns designed according to the code provisions display behavior which may range from unacceptably brittle to very ductile. The required structural performance, steel detailing and the load combinations must be considered, among other parameters, in the design of confining steel. Presence of heavy elements such as stub adjacent to the critical section must also be considered since their restraining effect enhances section strength significantly. For the same amount of confining steel, normal strength concrete specimens behave in a more ductile manner than the high strength concrete specimens. Currently the required amount of confining steel is directly proportional to the strength of concrete which may not remain practical for high strength concrete. However, it appears that in order to maintain ductile behavior of the core concrete, the required amount of lateral steel may be less than proportional to concrete strength.

ACKNOWLEDGEMENTS

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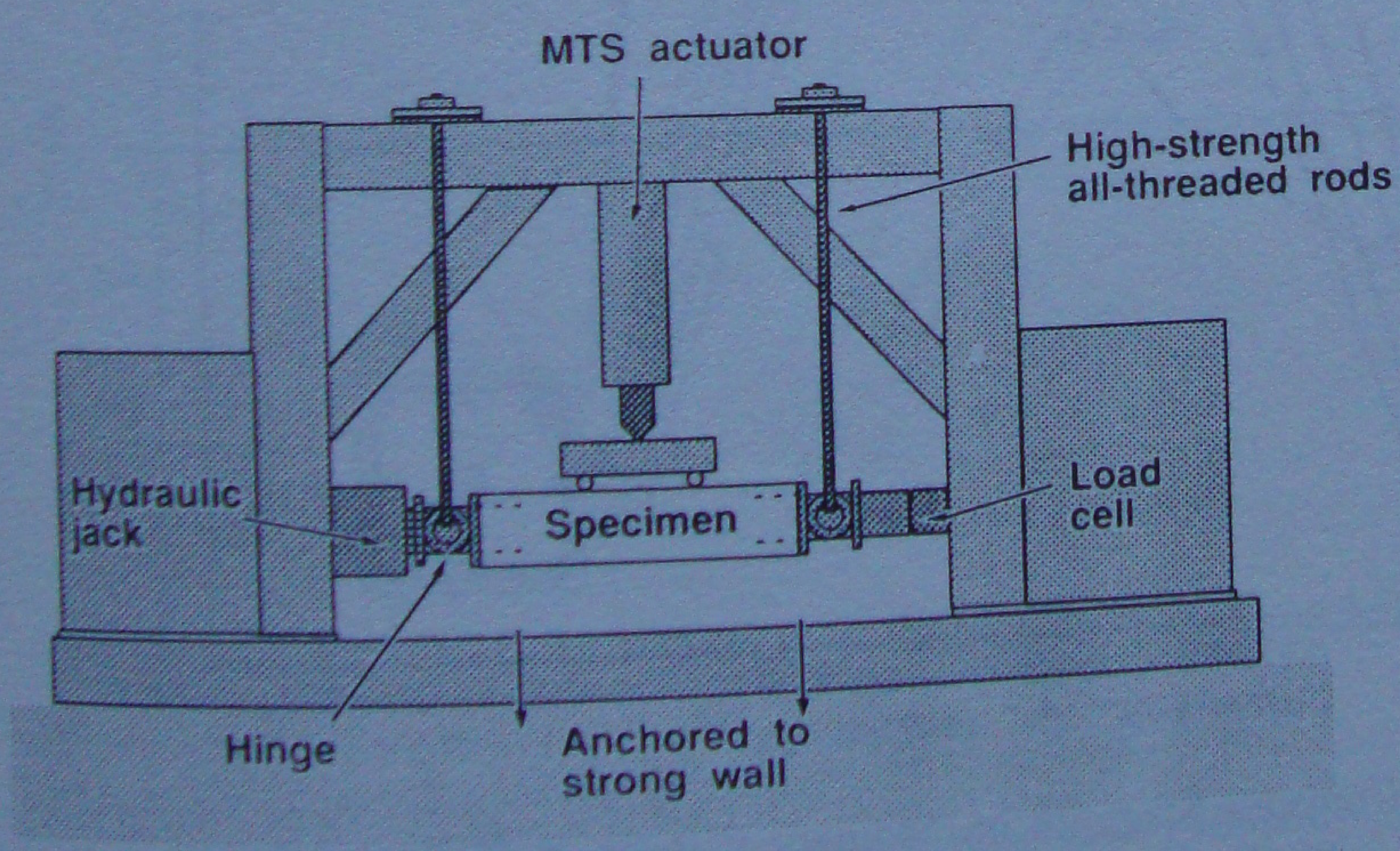


Figure 1 - Test Setup

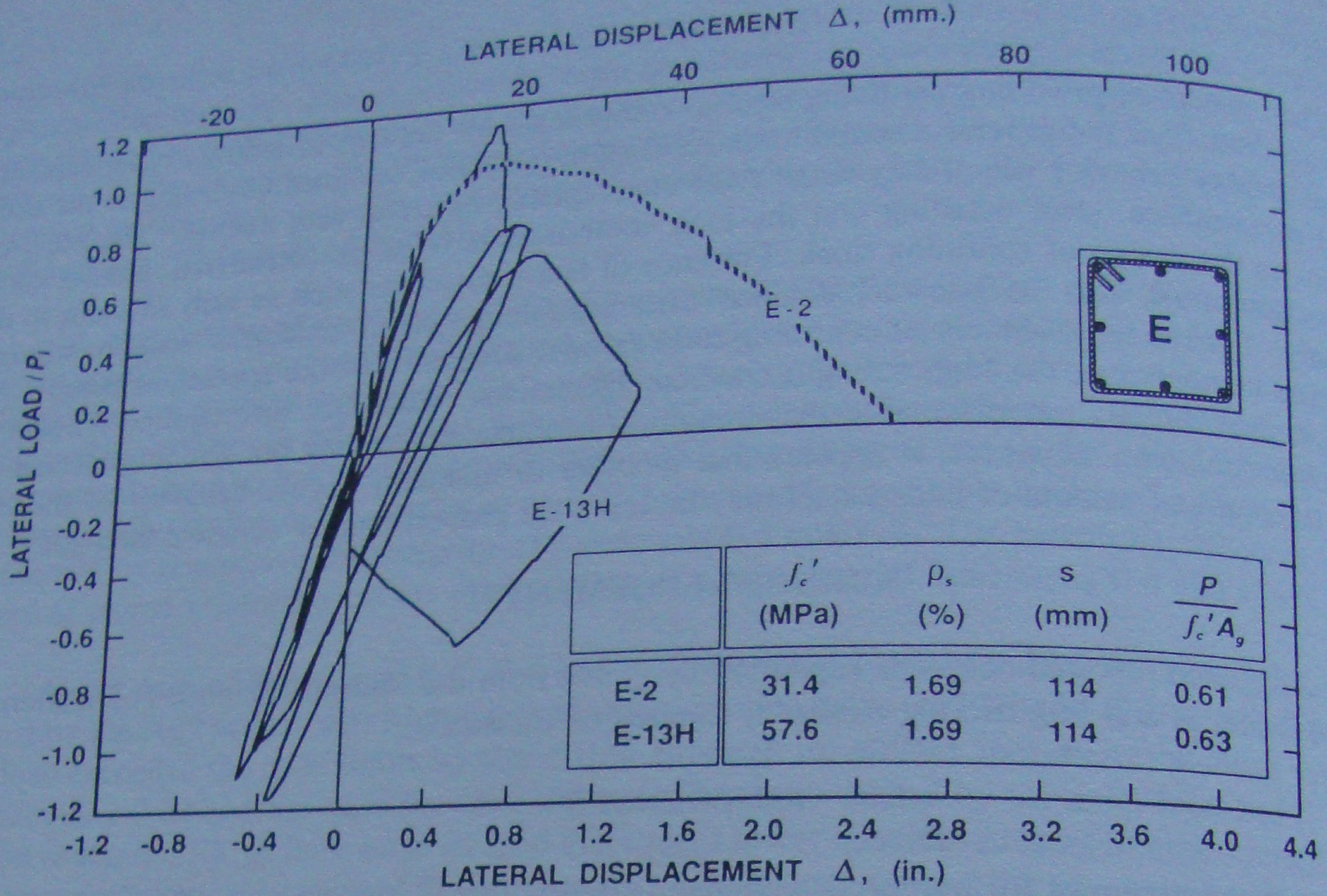


Figure 2 - Effect of Concrete Strength on Load-Deflection Behaviour of Configuration "E" Specimens

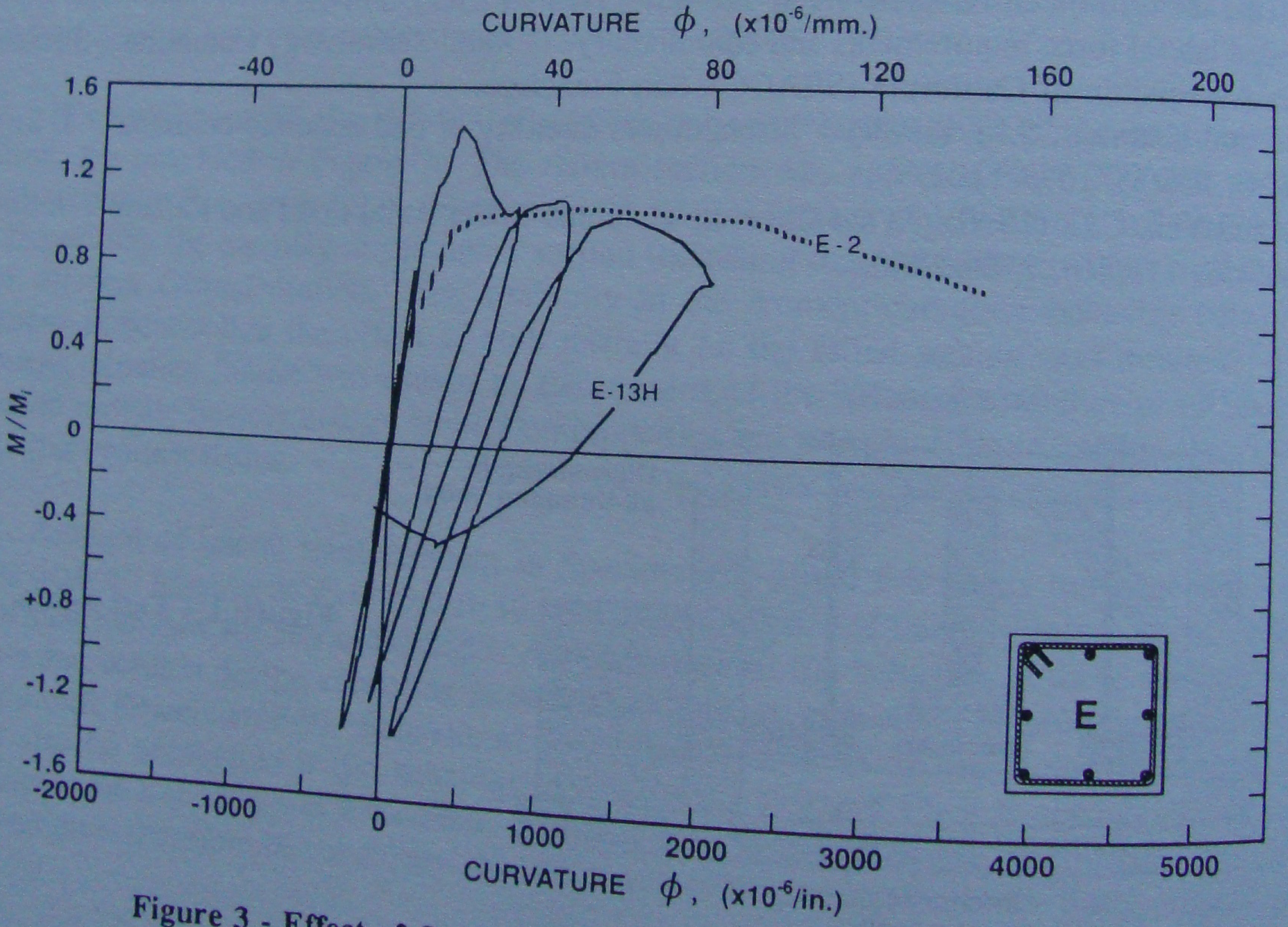
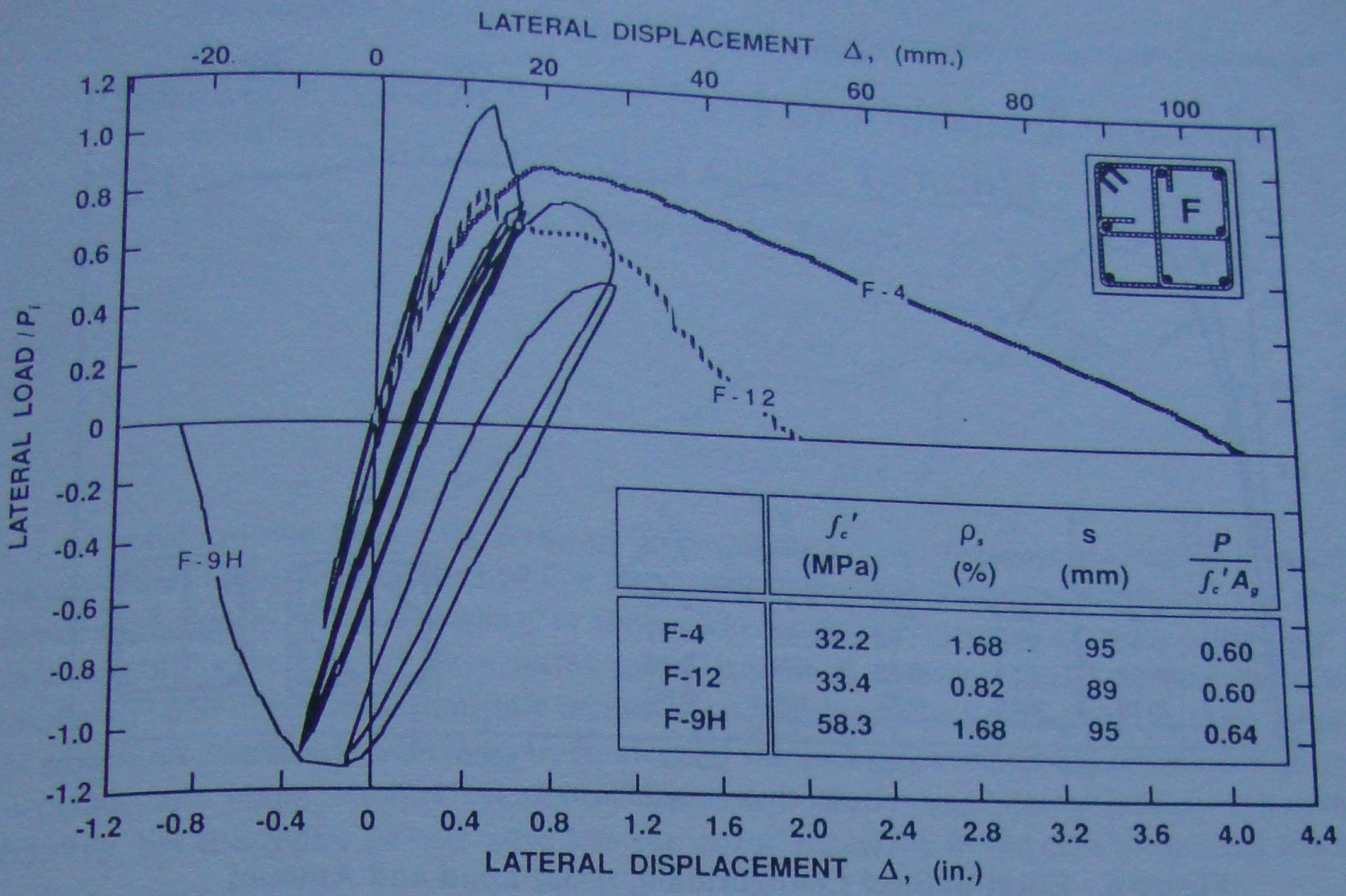


Figure 3 - Effect of Concrete Strength on Moment-Curvature Behaviour of Configuration "E" Specimens



	f_c' (MPa)	ρ_s (%)	s (mm)	$\frac{P}{f_c' A_g}$
F-4	32.2	1.68	95	0.60
F-12	33.4	0.82	89	0.60
F-9H	58.3	1.68	95	0.64

Figure 4 - Effect of Concrete Strength on Deflection Behaviour of Configuration "F" Specimens

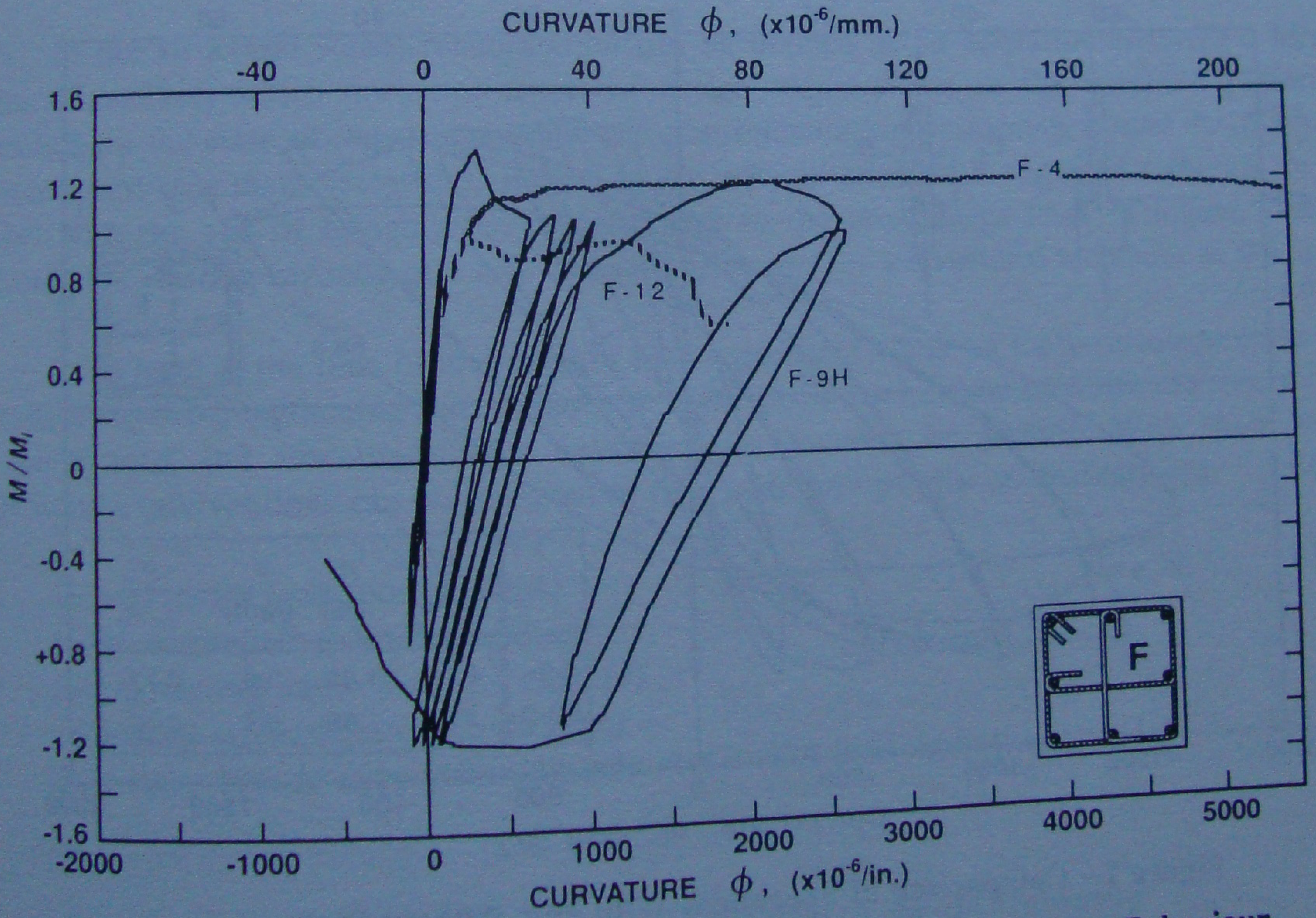


Figure 5 - Effect of Concrete Strength on Moment-Curvature Behaviour of Configuration "F" Specimens

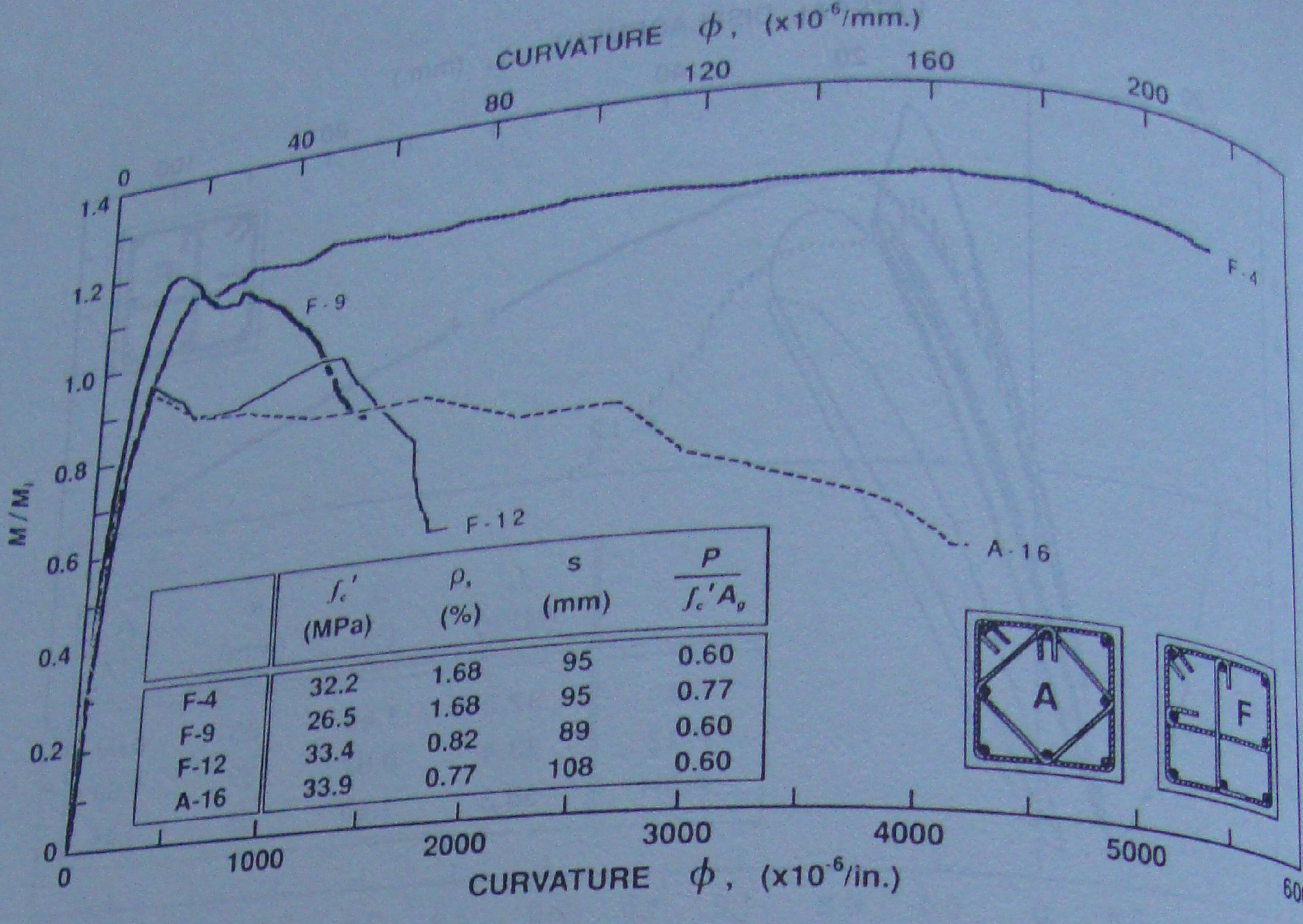


Figure 6 - Effect of Steel Configuration, Axial Load and Amount of Lateral Reinforcement

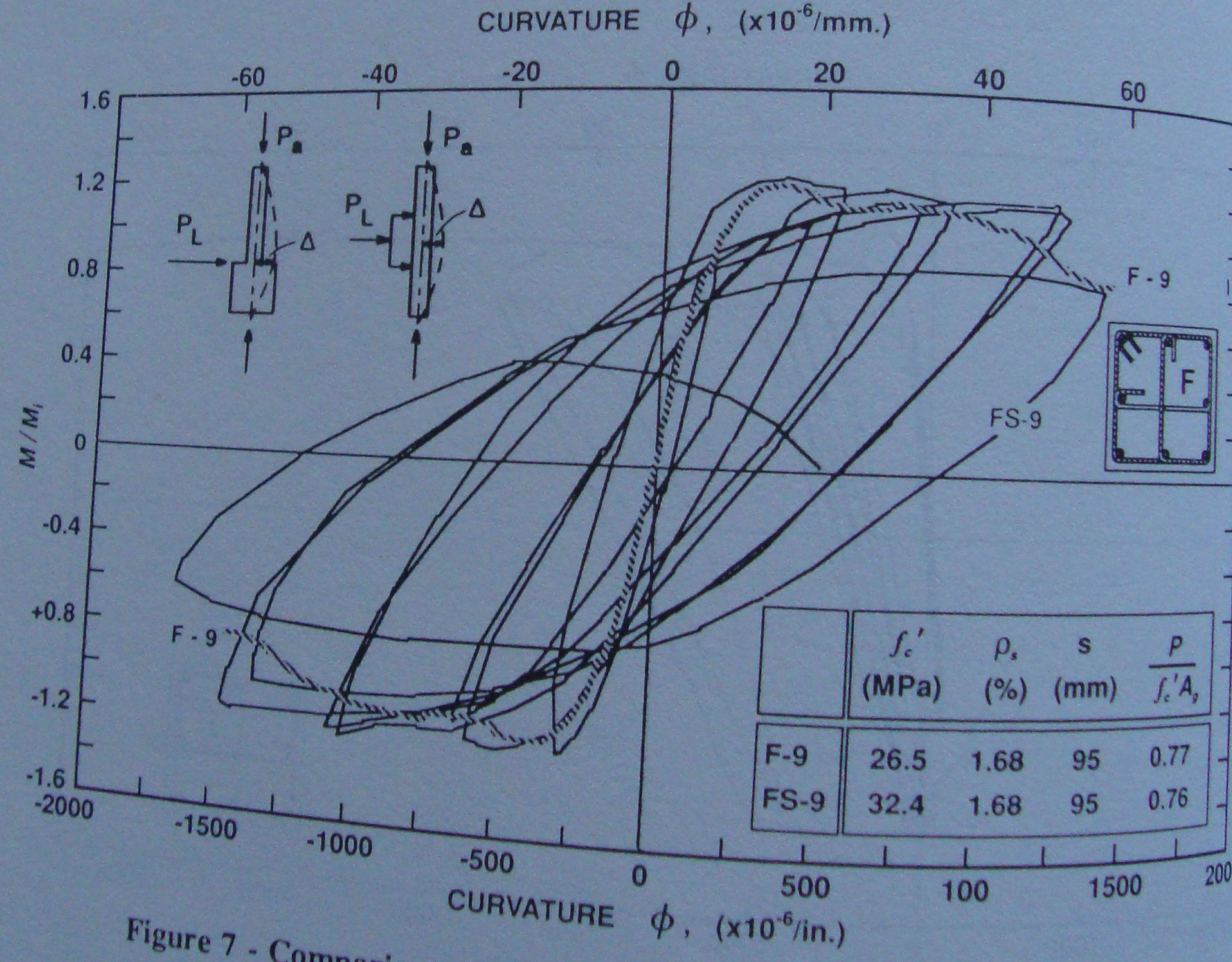


Figure 7 - Comparison of Specimens With and Without Stub