Curvature ductility of reinforced and prestressed concrete columns
by Bruce Alan Suprenant

A thesis submitted in partial fulfillment of the requirements for the degree of Doctor of Philosophy in Civil Engineering
Montana State University

Abstract:
Engineers are concerned with the survival of reinforced and prestressed concrete columns during earthquakes. The prediction of column survival can be deduced from moment-curvature curves of the column section. An analytical approach is incorporated into a computer model. The computer program is based on assumed stress-strain relations for confined and unconfined concrete, nonprestressed and prestressing steel. The results of studies on reinforced and prestressed concrete columns indicate that reinforced concrete columns may be designed to resist earthquakes, while prestressed concrete columns may not.

The initial reduction in moment capacity, after concrete cover spalling, of a prestressed concrete column could be as much as 50%. Analyses indicate that the bond between concrete and prestressing strand after concrete cover spalling is not critical.

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APPROVAL

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Bruce Alan Suprenant

This thesis has been read by each member of the thesis committee and has been found to be satisfactory regarding content, English usage, format, citation, bibliographic style, and consistency, and is ready for submission to the College of Graduate Studies.

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Any thesis would be difficult to finish without the support of many people. I would like to thank my parents, Jack and Betty, for their encouragement and support over many years and my wife, Susan, for her patience. Also, my little one, Ashley, for sleeping through the nights.

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ABSTRACT

Engineers are concerned with the survival of reinforced and prestressed concrete columns during earthquakes. The prediction of column survival can be deduced from moment-curvature curves of the column section. An analytical approach is incorporated into a computer model. The computer program is based on assumed stress-strain relations for confined and unconfined concrete, nonprestressed and prestressing steel. The results of studies on reinforced and prestressed concrete columns indicate that reinforced concrete columns may be designed to resist earthquakes, while prestressed concrete columns may not.

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CHAPTER I

BACKGROUND

Introduction

The ACI Code [10,11] requires the consideration of strength and serviceability in reinforced concrete design. Ductility is considered to be of secondary importance and is detailed into a structure, rather than designed. Because the ACI Code [10,11] regulates or delegates ductility to a secondary level, many engineers have been confused by the importance of ductility for a structure or member and its implication in the selection of lateral load levels for earthquake design.

Dynamic analyses of structures responding to ground motions recorded during severe earthquakes have been performed [35,58]. These show that the theoretical elastic response inertia loads are much greater than the lateral loads recommended by design codes [10,11, 46,59] for earthquake loading. It is well documented [9,24,30,37] that structures designed to the lateral load levels of design codes have survived severe earthquakes. This post earthquake survival has been attributed mainly to the ability of ductile structures to dissipate energy by post-elastic deformations. Although other energy dissipaters such as damping and soil-structure interaction help provide a reduced response, ductility of reinforced concrete members is considered to be the most important energy dissipater [35].

This thesis presents a theoretical study of the ductility of square confined reinforced and prestressed concrete column sections under monotonic loading. The parameters which were investigated to determine their effect on the ductility of reinforced and prestressed concrete columns are: axial load, longitudinal steel content, confinement, bond compatibility factor, concrete cover, and non-prestressed steel.

Ductility for Seismic Loading

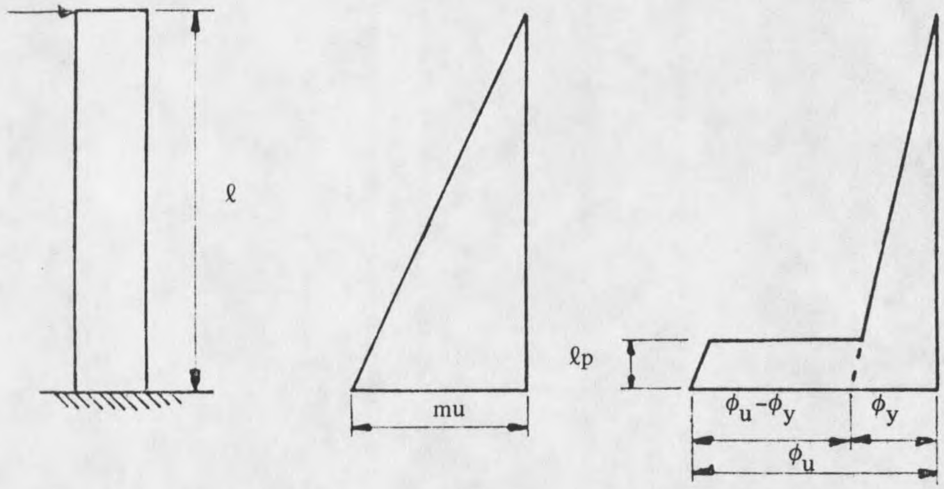
A measure of the ductility of structures with regard to seismic loading is the displacement ductility factor defined as Δ_u/Δ_y, where Δ_u is the lateral deflection at the end of the post-elastic range and Δ_y is the lateral deflection at first yield [37]. The displacement ductility factor required in design may be estimated on the basis of the ratio of elastic response load to code load and typical values for displacement ductility factors usually range from three to five [9,56].

A rotational ductility factor for members has been calculated by some dynamic analyses as Θ_u/Θ_y, where Θ_u = maximum rotation of end of member and Θ_y = rotation at end of member at first yield. The design engineer, however, would benefit by information concerning the member section behavior, expressed by the curvature ductility factor ϕ_u/ϕ_y. Where ϕ_u = maximum curvature at the section and ϕ_y = curvature of the section at first yield. For the designer, required ϕ_u/ϕ_y values are a more important index for ductility demand than the displacement or rotational ductility factors. This is because once yielding has commenced in a structure the deformations concentrate at plastic hinge positions and further displacement occurs mainly by curvature in the plastic hinge region [37]. Thus the required ϕ_u/ϕ_y ratio will be larger than the Θ_u/Θ_y ratio, and the Θ_u/Θ_y ratio will be greater than the Δ_u/Δ_y ratio.

The relationship between curvature ductility and displacement ductility has been illustrated with reference to a cantilever column with a lateral load at the end by Park and Paulay [37]. By using well known curvature-area theorems, the lateral deflection at the column top for first yield and ultimate deformation can be determined. Therefore, the displacement ductility factor may be related to the curvature displacement factor as shown in Figure 1.

For multistory frames, the ϕ_u/ϕ_y ratio required of members, designed according to present code seismic loading, has not yet been clearly established. The relationship between displacement and curvature ductilities can be complex [37]. Plastic hinges forming in beams and columns do so at different levels of axial and lateral load. Park and Paulay [37] have attempted to deduce curvature ductility demand of multistory frames using static collapse mechanisms.

The sequence of plastic hinge development in structures will influence the curvature ductility demand. Nonlinear dynamic analyses [37] have indicated that ductility demand concentrates in the weak parts of structures. The weak portion may act as a plastic hinge, enabling the remaining structure to respond elastically, or requiring the first hinge, i.e., weak portion, to achieve high curvature ductilities. Park and Paulay [37] have illustrated this by examining static collapse mechanism. Frame and shear walls which can be used for seismic resistance are shown in Figure 2. Possible mechanisms which could form due to flexural yielding and formation of plastic hinges are also shown in Figure 2. A column sidesway mechanism can form, if yielding commences in the columns at only one level, this would indicate that the columns of all other levels are stronger or carry less load. Such a mechanism requires very larger curvature ductility demands on the plastic hinges of the critical level [37], particularly for tall buildings. If, however, the yielding commences in the beams before the columns, a beam sidesway mechanism will develop [37]. This mechanism makes more moderate demands on the curvature ductility at the plastic hinges in the beams and column bases. This is basically due to the greater number of hinges required to



Member with ultimate curvature reached: (a) member, (b) bending moment diagram, (c) curvature diagram.

$$\Delta_y = \frac{\phi_y \ell}{2} \; \frac{2\ell}{3} \qquad\qquad \Delta_u = \left(\frac{\phi_y \ell}{2} \; \frac{2\ell}{3}\right) + (\phi_u - \phi_y)\, \ell p\, (\ell - 0.5\ell p)$$

$$u = \frac{\Delta_u}{\Delta_y} = 1 + \left(\frac{\phi_u - \phi_y}{\phi_y}\right) 3\ell p \left(\frac{\ell - 0.5\,\ell p}{\ell^2}\right)$$

$$\frac{\phi_u}{\phi_y} = \frac{\ell^2\ (u - 1)}{3\ \ell p\ (\ell - 0.5\ell p)} + 1$$

Curvature Ductility versus Displacement Ductility

	ℓp/ℓ	0.02	0.05	0.07	0.10	0.15	0.20	0.25
u =	5	68	28	21	15	11	8	7
	4	51	22	16	12	8	7	6
	3	35	15	11	8	6	5	4
	2	18	8	6	5	3	3	3

Figure 1. Relation between displacement and curvature ductility.

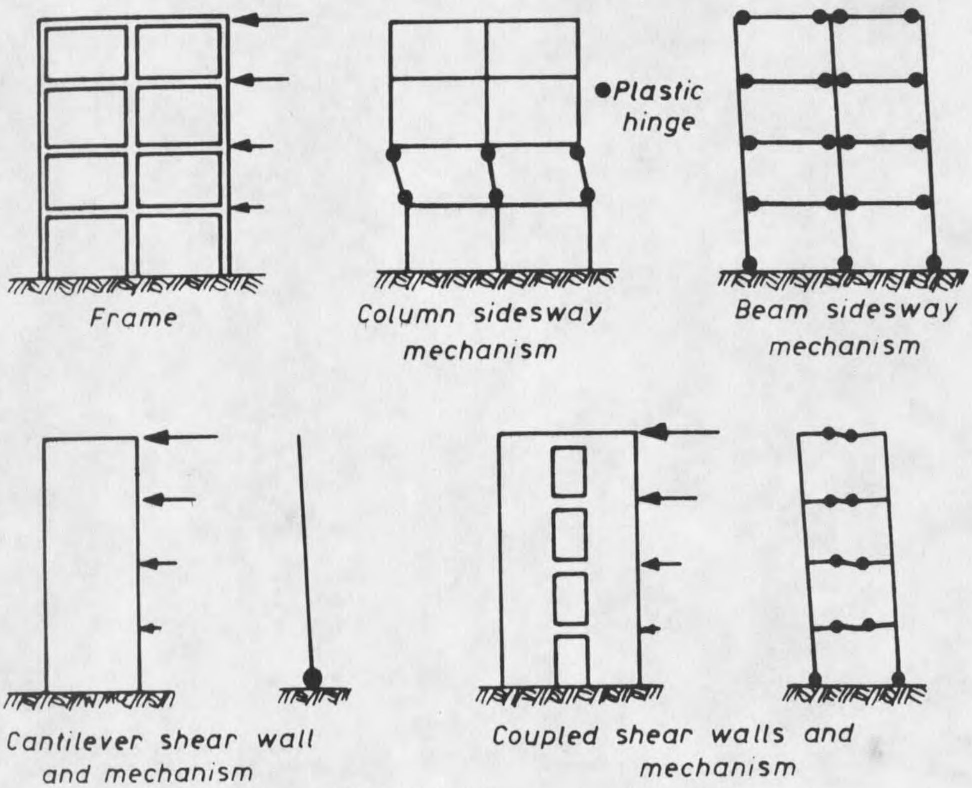


Figure 2. Building structures under seismic loading and possible mechanisms [37].

form in the beam mechanism. A beam sidesway mechanism is the preferred mode of inelastic deformation. Hence most codes [10,11,30,59] require a strong column-weak beam approach when designing moment resistant frames.

Higher modes of vibration in the actual dynamic situation influence the moment pattern and it has been found [37] that plastic hinges in beams move up the frame in waves involving a few levels at a time. For static collapse mechanism involving a plastic hinge at the base, the curvature ductility demand for a given displacement ductility factor depends very much on the plastic hinge length. Recent experimental information is available [17, 38,50] on plastic hinge lengths for reinforced concrete columns confined by hoop and spiral reinforcement.

The static collapse mechanisms of Figure 2 are idealized in that they involve behavior under code type static loading. Due mainly to the effects of higher modes of vibrations, the actual dynamic situation is different, but a review of Figure 2 will give the designer a reasonable perception of the collapse mechanism.

The strong column-weak beam design concept attempts to have plastic hinges form in the beams rather than the columns. The seismic provisions of ACI 318-77 [10] and ACI 318-83 [11] require that at beam-column connections the sum of the moment strengths of the column should exceed the sum of the moment strengths of the beams along each principal plane at the connections. Park and Paulay [37] have shown that this code requirement will not prevent plastic hinges formation in the columns for three reasons:

(a) The beam input moment may be considerably higher than calculated because of statistical variations in steel yield strength and the onset of steel strain-hardening.

(b) Points of inflection may occur well away from the mid-height of columns at various stages during an earthquake [19], thus requiring a column strength to prevent hinge formation which is much greater than required by ACI 318-77 and ACI 318-83.

(c) Biaxial bending resulting from an arbitrary ground motion will generally reduce the flexural strength of a column [34].

It is evident that column flexural strengths greater than the ACI requirements would be needed if plastic hinges in columns are to be avoided. However, column hinging may occur.

A column sidesway mechanism, besides requiring large curvature demands on the section, will also require an engineer with considerable ingenuity, since the straightening and repair of the columns will prove to be difficult. A recent example of damage concentrating mainly in one story of a structure is the Olive View Hospital which suffered considerable damage because of the 1971 San Fernando earthquake [24]. The permanent lateral displacement of the structure after the earthquake, which was about 2 ft, resulted almost entirely from the deformations in the first story columns. Because of the extensive damage, the Olive View Hospital was demolished.

As an example demonstrating the necessity of analyzing column curvature ductility, consider the shock-absorbing soft story concept for multistory earthquake structures proposed by Fintel and Khan [19]. This concept is based on controlling the lateral forces that occur in the structure during an earthquake by forcing all the inelastic deformation to a soft story. This concept requires extensive yielding in the columns and therefore requires the columns to sustain large post-elastic deformations. It appears that the "soft story" concept should be implemented in conjunction with appropriate consideration to future repair.

The "Guide Specifications for Seismic Design of Highway Bridges, 1983" [20] indicates that bridge piers should be designed for a required curvature ductility. This is the first code to imply that curvature ductility is an important requirement in design.

In the preceding discussion, the difference between reinforced or prestressed concrete ductility was not considered. While the ductility of reinforced concrete has been the subject of numerous experimental and theoretical studies [6,13,16,17,21,22,25,30,35,36,39, 41,42,43,44,45,52], the ductility of prestressed concrete has received little attention [9, 51]. Because of the lack of experimental and theoretical studies, prestressed concrete in primary seismic resistant elements such as shear walls and frames has not met with the same code acceptance as reinforced concrete.

There is a lack of detailed building code provisions for the seismic design of prestressed concrete. For example, ACI 318-83 [11] contains special provisions for the seismic design of reinforced conrete structures but does not have corresponding provisions for prestressed concrete. This is also true of the Uniform Building Code [59], the SEAOC Code [46], and the seismic design provisions of the Applied Technology Council [44].

Lin [30] presented results of a dynamic computer analysis of a 19 story prestressed concrete apartment building. The building was first designed using the equivalent static earthquake forces specified by the 1961 Uniform Building Code. The elastic dynamic response of the structure to the North-South component of the 1940 El Centro earthquake was calculated. The El Centro earthquake produced forces and displacements about five times the Code values, This indicates the need for prestressed structures to be capable of developing large post-elastic deformations or be designed for greater equivalent static lateral loads.

The area under moment-curvature curves for prestressed and reinforced concrete give an indication of their energy absorption capacities. Despeyroux [18] and Candy [18] established very important differences between the behavior of reinforced and prestressed concrete. Candy illustrated the differences by using Figure 3. Cyclic loading tests have given curves for the two materials approximately as drawn. The shaded area represents the energy dissipation; the area up to the dashed line represent the energy absorbed. Thus although the energy *absorbed* by a prestressed member and a reinforced concrete member

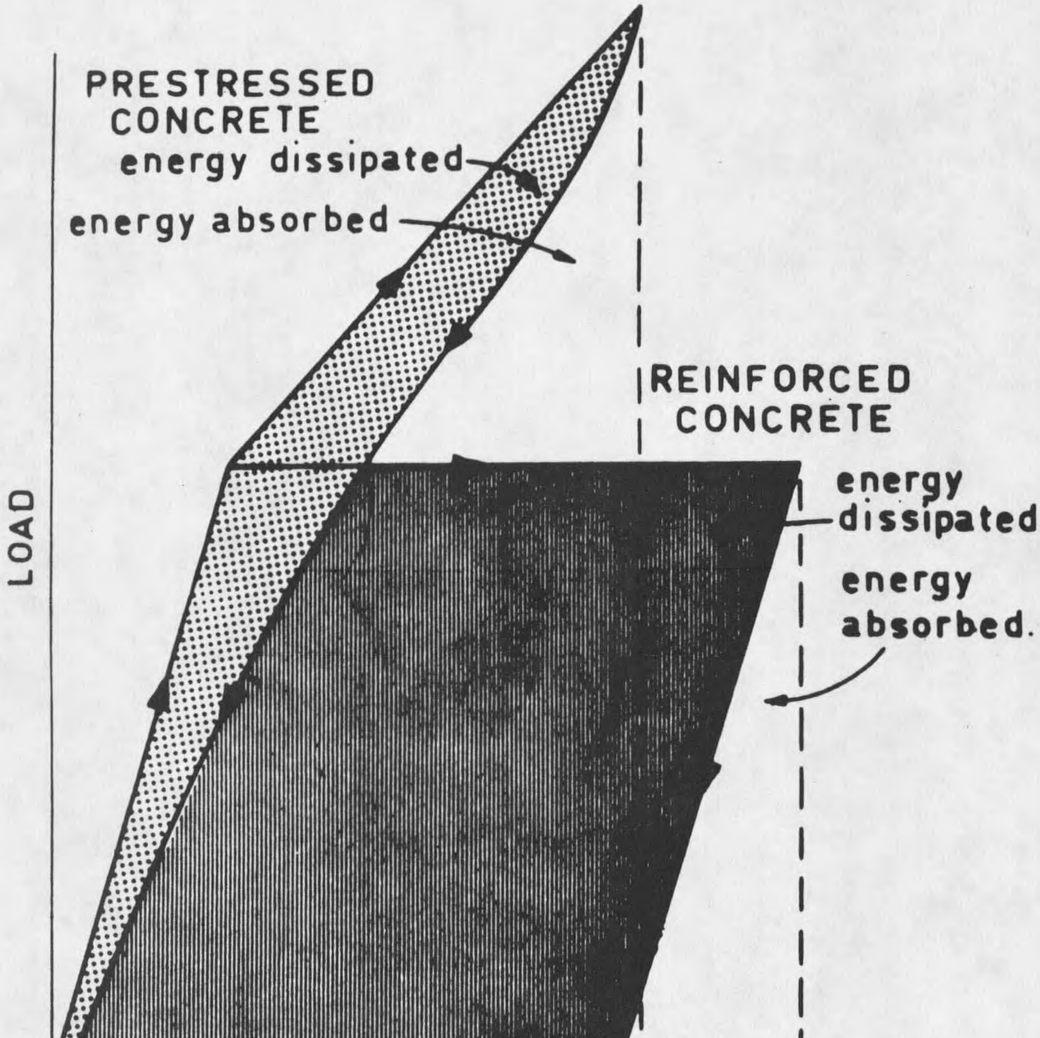


Figure 3. Dissipation of energy in prestressed and reinforced concrete members [18].

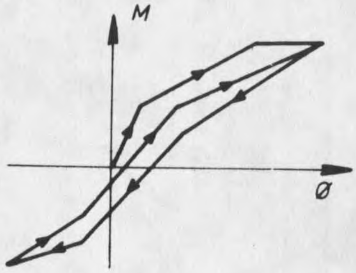
may be the same, much more energy would be *dissipated* in the reinforced concrete member [35]. Candy concluded that the reduction in response caused by plastic strain is much smaller in prestressed concrete.

Figures 4 (a), 4 (b), and 4 (c) show idealized moment-curvature hysteresis loops for cyclic loaded prestressed and reinforced concrete members [58]. Nonlinear dynamic analyses of single degree of freedom systems responding to severe earthquakes have shown that the maximum displacement of a prestressed concrete system is on the average 1.3 times that of a reinforced concrete system with the same code design strength, viscous damping ratio, and initial stiffness [37]. The moment-curvature loops of prestressed concrete members can be "fattened" to reduce the displacement response, if non-prestressed steel is added to the member to provide energy dissipation. Figure 4 (c) shows a typical moment-curvature loop for partially prestressed concrete.

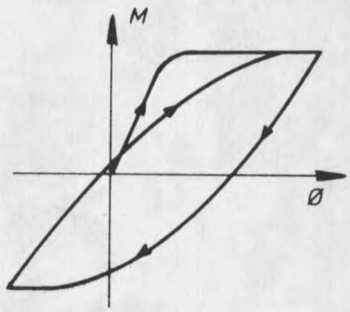
Although the displacement response of prestressed and reinforced concrete members are different, the possibility of yielding occurring at column ends makes it important to ensure that the columns are capable of behaving in a ductile manner. Hence, for reinforced and prestressed concrete columns adequate transverse steel in the form of hoops should be provided at the potential plastic hinge regions.

Existing Code Requirements for Special Transverse Steel in Columns for Seismic Loading

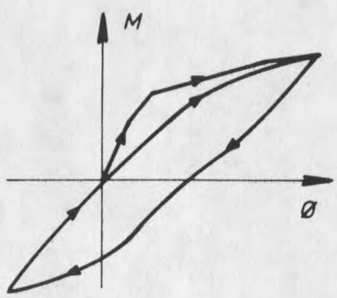
The ACI [10,11], SEAOC [46], and ATC [44] codes contain recommendations for the amount of special transverse reinforcement required in the ends of columns when ductile moment-resiting frames are to be designed for seismic loading. All columns of ductile moment-resisting frames designed according to the SEAOC code are required to have transverse reinforcement. ACI 318-77 requires special transverse reinforcement only when the column load exceeds $0.4\,\phi P_b$.



(a) Prestressed Concrete System



(b) Reinforced Concrete System



(c) Partially Prestressed Concrete System

Figure 4. Idealized moment-curvature hysteresis loops for structural concrete systems [58].

For circular spiral steel the ratio of volume of spiral reinforcement to total volume of core is given as:

$$\rho_s = 0.45 \left[\frac{A_g}{A_c} - 1\right] \frac{f'c}{fy} \qquad (1)$$

but not less than 0.12 f'c/fy.

When rectangular hoop reinforcement is used without supplementary cross ties, the required area of a bar leg is given as:

$$A_{sh} = 0.225\, \ell_h S_h \left[\frac{A_g}{A_C} - 1\right] \frac{f'c}{fy} \qquad (2)$$

Equation (1) was derived from the requirement that the strength of an axially loaded spiral column after the concrete cover has spalled should at least equal the strength before spalling. Based on test results achieved by Richart, Brandtzaez, and Brown [47], it was assumed that when the concrete cover spalls, the spiral steel yields and exerts a radial pressure on the concrete core. This pressure increases the compressive strength of the core by approximately 4.1 times the radial pressure. Assuming that rectangular hoops are only 50 percent as effective as circular spirals as confining reinforcement, Equation (2) can be derived.

ACI 318-83 [11] proposes two new equations for spiral and loop confinement of columns. For spiral reinforcement,

$$\rho_s = 0.12\, f'c/fyh \qquad (3)$$

and for hoop reinforcement not less than Equations 4 or 5

$$Ash = 0.3\, (sh_c f'c/fyh)[(Ag/Ach) - 1] \qquad (4)$$

$$Ash = 0.12\, sh_c f'c/fyh \qquad (5)$$

The philosophy of preserving the ultimate strength of axially loaded columns after spalling has been discussed [37]. A more appropriate criteria would emphasize the ultimate deformations and limited amount of moment reduction of concrete columns. The confinement afforded by the transverse steel of Equations (1), (2), (3) or (4) will result in improved column behavior but these equations may not accurately represent the amount of confining steel necessary to achieve the required ultimate curvatures under earthquake loading. Roy and Sozen [48] indicated that confinement by rectangular hoops may not cause any strength increase, thus the philosophy behind the derivation of Equation (2) may never be realized in actual practice.

The previous discussion has not been related to prestressed columns. While investigations on confining steel for prestressed columns is in progress, current recommendations [10,11,46,59] for prestressed columns are the same as those mentioned for reinforced concrete columns due to a lack of information.

Determining Transverse Steel Content

An assessment of the quantity of special transverse steel required in columns for seismic design should take into account:

1. Required curvature ductility factor, γ_u/γ_y
2. Column axial load level, P/P_b
3. Rate of loading, $\dot{\epsilon}$
4. Longitudinal steel content, ρ
5. Effective prestress, ϵ_{se}
6. Stress-strain curve of concrete
7. Stress-strain curve of steel

A theoretical analysis requires a stress-strain curve for concrete confined by transverse reinforcement and a stress-strain curve for steel which includes strain hardening. Ideally, the effects of cyclic loading should be considered, but the increase in expense and complexity of analysis reduces its usefulness. The complexity of the analysis would increase for prestressed concrete column because of the scarcity of experimental results.

Park and Sampson [39] have been advocating the use of monotonic moment-curvature analysis to assess the expected ductility of columns subjected to earthquake loading. In order to indicate the acceptability of this practice, the behavior of concrete and steel materials under cyclic loading must be considered.

Sinha, Gerstle, and Tulin [53] were the first investigators to indicate that the monotonic stress strain curve for unconfined concrete is an envelope curve for the cyclic stress strain characteristics of unconfined concrete. Figure 5 shows this result. The investigators [53] showed that this was also true for idealized behavior of the reinforcing (see Figure 6). Karsan and Jirsa [25] confirmed the results of Sirha, Gerstle, and Tulin for unconfined concrete.

Experimental results for an unconfined concrete section, concrete and steel acting together, indicated that the monotonic moment-curvature is an envelope curve for cyclic moment-curvature curves [35,37].

Park and Sampson [39] were the first to assume that the behavior of confined concrete would follow the same general characteristics of unconfined concrete. They assumed that the monotonic moment curvature curve was an envelope curve for cyclic moment-curvature curves for confined concrete. Just recently, investigators from the University of New Zealand have proven this behavior, for both square-confined and spirally-confined reinforced concrete columns [38,43].

Some work [50] has been performed to indicate that the monotonic stress-strain curve of confined concrete at high strain rates will also be an envelope for high strain rate cyclic loading. These results have yet to be totally confirmed.

There appears to be enough evidence [8,35] to suggest that the monotonic moment curvature curve for prestressed concrete will be an envelope curve for cycling moment-curvature curves of prestressed concrete columns. Therefore, the assessment of maximum