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What Do We Know about Confinement in Reinforced Concrete Columns? (A Critical Review of Previous Work and Code Provisions)

by Koji Sakai and Shamim A. Sheikh

Based on an extensive review of the literature, a state-of-the-art report on concrete confinement is presented. It is aimed at defining the status of the problem and the future direction of work including revision of the design codes' provisions. Topics discussed include properties of confined concrete, behavior of confined sections and columns including plastic hinge regions, and a critical evaluation of the design codes' provisions. With the reinterpretation of previous data in light of the results from recent tests at the University of Houston, apparent contradictions on the effects of several variables have been explained. Several areas in which the design codes' provisions need revisions have been identified. An extensive list of references on related topics is also included.

Keywords: axial loads; **building codes; columns (supports); confined concrete;** ductility; earthquake-resistant structures; moment-curvature relationship; **reinforced concrete;** reinforcing steels; **reviews.**

It is uneconomical to design a structure to respond in the elastic range to the greatest likely earthquake-induced inertia forces because the maximum response acceleration may be several times the maximum ground acceleration, depending on the stiffness of the structure and the magnitude of damping.[1] This suggests the necessity to design structures so that the energy can be dissipated by postelastic deformations of members, which requires certain elements to be designed for ductility as well as strength. It is well known that the ductile behavior of concrete sections can be attained by carefully detailed transverse reinforcement, which improves the properties of concrete by confining it.

To discourage plastic hinging in columns, most building codes[2-7] have adopted the design concept of "strong column-weak beam," which is stated in the form of restricting the ratio of the sum of flexural strengths of the columns ΣM_c to that of the beams ΣM_g at a beam-column joint, or amplifying the column bending moments found from elastic frame analysis. Appendix A of the ACI Building Code[2] requires that $\Sigma M_c \geqslant (6/5)\, \Sigma M_g$. The magnitude of the amplification factor to minimize the possibility of column hinging during inelastic displacements of a frame has been a debatable issue.[8,9] Especially, Paulay[9] suggests, if all uncertain features are taken into consideration, the ratio of nominal flexural strengths of columns to those of beams meeting at a joint may have to be in the range of 2 to 2.5 to prevent the plastic hinges from forming in columns.

From the observation of several damaged structures, it can be seen that in several cases failure of an entire structure was triggered by the failure of columns[10-12] by chain action. Since effectiveness of the design approach involving strong column-weak beam concept is still a controversial matter, it will be dangerous to design the structures without considering the likelihood of the formation of plastic hinges in columns. Furthermore, taking into consideration the failure of structures due to unexpected actions and consequently the loss of lives, the design on the premise that plastic hinges may occur in columns may be eventually more economical, even though the initial cost of detailing will be higher.

RESEARCH SIGNIFICANCE

The preparedness for the formation of plastic hinges in columns requires confinement of concrete by transverse reinforcement. There has been extensive research on concrete confinement recently. However, it cannot be said that the results of this research have been effectively reflected in codes, as most of the information obtained from the research was fundamental and fragmented and consequently did not significantly influence the established provisions of codes. A systematic evaluation of the previous research on confinement and ductility of reinforced concrete columns and of the

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codes' provisions is reported elsewhere[13] and a summary is presented here with a view that it will help define the status of the problem and indicate the direction of the future work.

HISTORICAL BACKGROUND ON CONCRETE CONFINEMENT

Spiral reinforcements in concrete columns were originally introduced by Considere.[14] Based on the results of an extensive experimental program, Richart, Brandtzaeg, and Brown[15-16] and Richart and Brown[17] proposed the following relationship for strength applied to both spirally reinforced and hydraulically confined columns

$$f_{cc} = f_{cp} + 4.1 f_l \quad (1)$$

The study on the effects of rectangular transverse reinforcement in reinforced concrete columns traces back to the work by King.[18-20] The main purpose of the study was to establish a formula for ultimate strength of reinforced concrete columns with single square hoops. No attention was paid to column ductility. Chan[21] published his work that aimed at the verification of the validity of plastic hinge theory in reinforced concrete frameworks. In this study, the failure mechanism of core concrete under rectilinear confinement was described. In addition to the beneficial effects obtained from the rotation capacity of the confined plastic hinges in the design of statically indeterminate structures, Blume, Newmark, and Corning[22] pointed out the advantages of using confined concrete in earthquake-resistant design.

SCOPE OF PREVIOUS RESEARCH AND RELATIONSHIP TO CODES

Fig. 1 outlines the scope of research and the relationship to codes. The objectives of the research can be divided fundamentally into four categories: 1) characteristics of materials; 2) characteristics of cross section; 3) behavior of reinforced concrete columns; and 4) other mechanical characteristics and design constraints, such as structural detailing.

It is well known that the confinement by circular steel spirals is generally more effective than that by rectilinear hoops. In this paper, mainly the topics on rectangular or square columns will be discussed.

Characteristics of material

To understand the behavior of reinforced concrete columns, a knowledge of the fundamental properties of concrete and steel is required. The concrete in columns with transverse reinforcement consists of cover (unconfined) concrete and core (confined) concrete. The load-carrying behavior of cover concrete is generally different from that of plain concrete cylinders or prisms because the behavior will be affected by the thickness of cover and the spacing of transverse reinforcement. With transverse reinforcement, strength and ductility of concrete are generally improved depending on the degree of confinement. The stress-strain relationship of confined concrete is a function of many variables. Therefore, the main interest of most researchers was to examine the effects of an array of variables and to propose analytical models for the stress-strain curve of confined concrete.

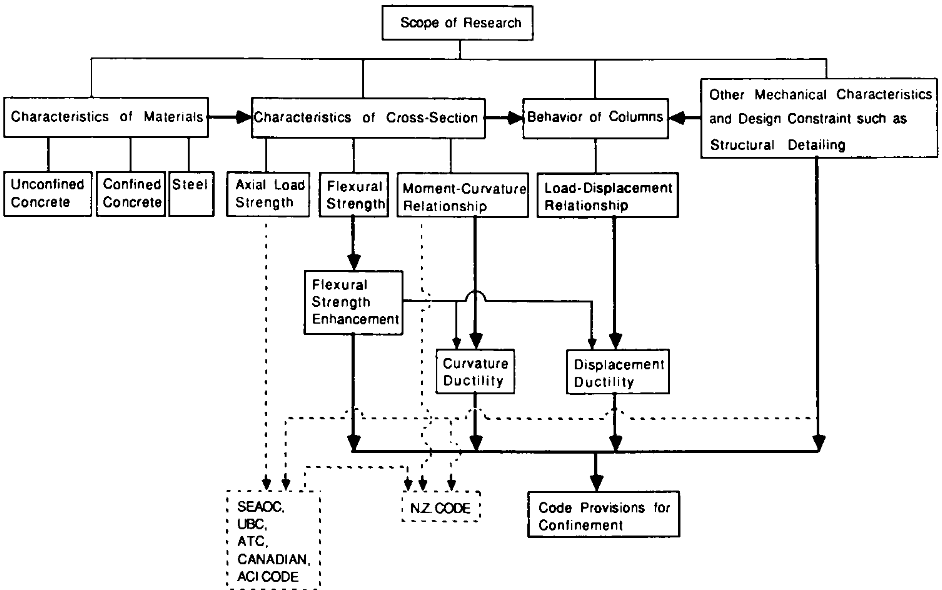

Fig. 1—Scope of research and relation with codes

Characteristics of cross section

The flexural strength of a confined section calculated according to the ACI Building Code (ACI 318-83)[2] procedure based on unconfined concrete properties will usually be a conservative estimate of the actual strength. This conservative prospect is not necessarily on the safe side for shear design, which is in general based on the flexural strength. In addition, flexural failure may occur outside the confined region. The ultimate curvature of the section according to the ACI procedure, which is based on the maximum concrete compressive strain of .003, will only provide a lower bound for the confined concrete section. The ductility of the section, which can be expressed by the ratio of ultimate curvature ϕ_2 to the curvature at first yield ϕ_1, would significantly improve by concrete confinement. The level of axial load on the section would also affect curvature and ductility significantly.

Fig. 2 shows an example comparing the maximum design axial loads according to the ACI and New Zealand (NZS 3101:1982) codes.[2,7] The maximum design axial load in the ACI code comes from the consideration of accidental eccentricities not considered in the analysis. In this code, there is no additional provision on the maximum allowable axial load for seismic design. On the other hand, the New Zealand (NZS 3101:1982) code limitation on the design axial load is based on the adverse effects of high axial load on the available curvature ductility. It should be noted that the axial load limits for nonseismic design in the New Zealand code are the same as in the ACI code and are lower than that for seismic design. As shown in Fig. 2, the provisions of both codes allow considerably high levels of axial load. Furthermore, actual axial load on columns may be higher than the code-specified loads due to unexpected actions during an earthquake. Although it is well known that the level of axial load has a significant effect on the flexural behavior of a reinforced concrete section, most of the experimental studies have been done under comparatively low levels of axial load.

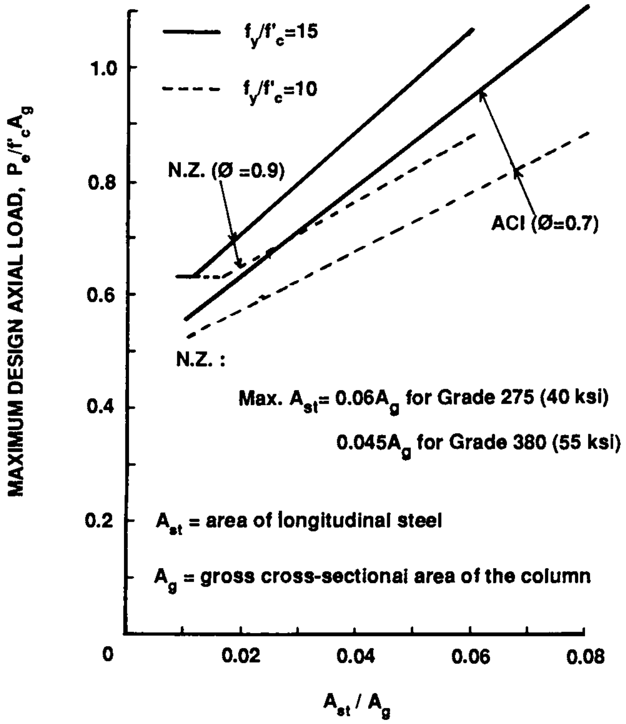

Fig. 2—Maximum design axial loads in the ACI and New Zealand Codes

Code requirements for confining steel

The current ACI code requirements for transverse reinforcement were derived on the basis of strength enhancement of concrete due to confinement as observed by Richart et al,[15-17] with the concept that axial load-carrying capacity of a column should be maintained after spalling of cover concrete. The code equations for the total volumetric ratio of spiral or circular hoop reinforcement ρ_s and for the total area of rectilinear transverse reinforcement are as follows

$$\rho_s = 0.45\left(\frac{A_g}{A_c} - 1\right)\frac{f'_c}{f_y} \tag{2}$$

$$\geqslant 0.12\frac{f'_c}{f_y} \tag{3}$$

$$A_{sh} = 0.3\, sh_c \frac{f'_c}{f_y}\left[\left(\frac{A_g}{A_{ch}}\right) - 1\right] \tag{4}$$

$$\geqslant 0.12\, sh_c \frac{f'_c}{f_y} \tag{5}$$

From Eq. (2) and (4), it is found that the efficiency of rectangular transverse reinforcement corresponds to 75 percent of that of the same volume of circular spirals. Similarly, the efficiency of rectangular transverse reinforcement in Eq. (5) corresponds to 50 percent of that of Eq. (3). Thus, there is a clear inconsistency. Furthermore, it is obvious that the philosophy of maintaining the axial load strength of the section after spalling of the cover concrete does not directly relate to the ductility of reinforced concrete column sections subjected to combined flexural and axial loads. Ideally, codes should provide the required amount of transverse reinforcement needed for a certain value of curvature ductility. The New Zealand code[7] attempts to achieve this by including the level of axial load in the confinement equations that are given below for rectilinear ties

$$A_{sh} = 0.3\, sh_c\left(\frac{A_g}{A_{ch}} - 1\right)\frac{f'_c}{f_{yh}}\left(0.5 + 1.25\,\frac{P_e}{\phi f'_c A_g}\right) \tag{6}$$

$$\geqslant 0.12\, sh_c\frac{f'_c}{f_{yh}}\left(0.5 + 1.25\,\frac{P_e}{\phi f'_c A_g}\right) \tag{7}$$

It should be noted that Eq. (6) and (7) are similar to ACI Eq. (4) and (5) except for the term $0.5 + 1.25\, P_e/\phi f'_c A_g$ which accounts for the effect of axial load.

Behavior of columns

The displacement ductility factor, commonly used to assess the behavior of members, can be generally expressed by the ratio of ultimate displacement Δ_2 to the displacement at first yield Δ_1 in lateral load-displacement relationships. The displacement ductility in columns is closely related to the curvature ductility in column sections. Fig. 3 shows relationships between curvature ductility factors and displacement ductility factors in which the effect due to additional deformations such as slippage of longitudinal bars and shear cracking is neglected. For a given displacement ductility, the required curvature ductility is influenced strongly by the geometry of the structure and length of the plastic hinge. The displacement ductility factor is fundamentally an indication to assess the plastic displacement in a column in which static load less than the elastic response inertia load is used for design. On the basis of the assumption of equal maximum deflections, it can be shown that, for elastoplastic systems, if the ratio of design load to elastic response load is x, the required displacement ductility factor is $1/x$. For severe earthquakes, New Zealand (NZS 4203:1984) code[23] requires that the building as a whole should be capable of deflecting laterally through at least eight load reversals so that the total horizontal deflection at the top of the main portion of the building under the given loadings (according to given equations), calculated on the assumption of appropriate plastic hinges, is at least four times that at first yield, without the horizontal load-carrying capacity of the building being reduced by more than 20 percent.

An assessment of the length of plastic hinge region for certain curvature and displacement ductility factors is important for confinement. In the ACI code, the plastic hinge region is taken as not less than: (a) the depth of the member at the joint face or at the section where flexural yielding may occur; (b) one-sixth of the clear span of the member; and (c) 18 in. (457 mm). The New Zealand code[7] requires that for $P_e \leqslant 0.3\ \phi f'_c A_g$, the plastic hinge region shall not be less than the larger of the longer cross-sectional dimensions, or the length where the moment exceeds 0.8 of the maximum moment at that end of the member, and for $P_e > 0.3\ \phi f'_c A_g$, not less than the larger of 1.5 times the longer member cross-sectional dimension, or the length where the moment exceeds 0.7 of the maximum moment at that end of the member.

Another important function of transverse reinforcement in reinforced concrete columns is to prevent buckling of the longitudinal bars. The ACI code does not address this directly and requires that the maximum spacing be the smaller of: (a) ¼ of the minimum dimension of the cross section; or (b) 4 in. (102 mm). The corresponding limits in the New Zealand code are: (a) ⅕ of the minimum dimension of the cross section; (b) six times the longitudinal bar diameter; or (c) 200 mm (7.87 in.). The second restriction is specifically to prevent the buckling of longitudinal bars when undergoing yield reversals in tension and compression. A review of the previous research indicates that the codes' provisions for maximum tie spacing are not based on any particular experimental or analytical findings. Although lapped splices in the longitudinal bars immediately above floor levels have a great advantage from the viewpoint of construction, most codes do not permit lapped splices in the potential plastic hinge region.

STRESS-STRAIN RELATIONSHIPS FOR CONFINED CONCRETE

Numerous studies have been done on the behavior of concrete confined by transverse reinforcement.[15-22, 24-74] The main factors considered in these studies are: 1) type and strength of concrete; 2) amount and distributions of longitudinal reinforcement; 3) amount, spacing, and configurations of transverse reinforcement; 4) size and shape of confined concrete; 5) ratio of confined area to gross area; 6) strain rate; 7) strain gradient; 8) supplementary crossties; 9) cyclic loading; 10) characteristics of lateral steel; and 11) level of axial load in the case of flexural behavior.

On the basis of the experimental data, various stress-strain curves for confined concrete have been proposed.[21-22, 24, 27, 31, 32, 34, 37, 39, 43, 44, 46, 48-51, 54, 55, 57] A comparative study[45] shows that most of these analytical models

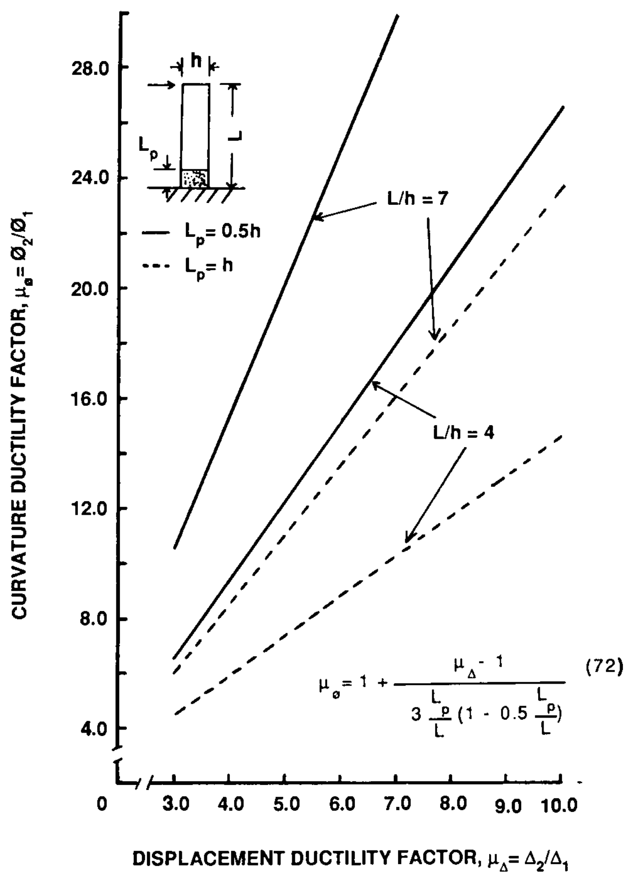

Fig. 3—Relationships between curvature ductility factor and displacement ductility factor

are effective only to interpret their own test results or data used. This may be attributed to the differences in the details of test specimens used and the variables considered in the development of the analytical models. The two models, Sheikh and Uzumeri[44] and modified Kent and Park,[42] which are based on the test results using large-size specimens with practical detailing of transverse and longitudinal reinforcements, appeared most promising. The model by the Sheikh and Uzumeri[44,55] considers the distribution of both longitudinal and lateral steel in a column and predicted results of a large variety of tests better than other models. The modified Kent and Park model[42] overestimated strength and ductility of columns in which longitudinal steel bars were not extensively distributed and supported by tie bends. Mander[49] recently developed a model in which ultimate strength surface for concrete under a multiaxial state of stress and ultimate concrete compression strain at which first hoop fracture occurs have been theoretically introduced. Fig. 4 through 6 show comparison of the experimental results with the predictions of various models for the tests on 12 x 12 x 108-in. (305 x 305 x 2743-mm) columns tested recently at the University of Houston.[57] Predictions from various models differ significantly because different sets of variables are considered in different models. It was observed from the 16-column test program that the predictions from the Sheikh and Uzumeri model changed from conservative at low axial load levels to unsafe at high axial loads. Following an approach similar to that used by Vecchio and Collins,[75] the model was modified to account for a reduced strength of concrete when it is subjected to tensile strains in the lateral direction caused by high axial loads.[57] Fig. 4 through 6 also show the results from the modified Sheikh and Uzumeri model.

Most of the studies on confinement have treated normal weight concrete. In the special provisions for seismic design in the ACI code, maximum compressive strength of lightweight-aggregate concrete used in design is restricted to 4000 psi (27.6 MPa). Although several studies have been done on spirally reinforced lightweight-aggregate concrete,[59-62] only limited data is available for rectangular lightweight concrete columns with transverse reinforcement.[63-64] Rabbat et al.,[63] based on their test data, concluded that the requirements for column confinement in the ACI code can be extended to lightweight concrete columns with concrete strengths up to 6000 psi (41.4 MPa). However, it is to be noted that the flexural tests were carried out under axial loads up to 30 percent of the column design strength. Basset and Uzumeri,[64] in their experimental investigation, found that the existing models for concrete confinement by rectilinear ties gave unconservative results for high-strength lightweight aggregate concrete and overestimated the ductility.

The actual rate of application of forces during an earthquake is considerably higher than that of quasi-static loading generally used in the tests. With respect to the effect of strain rates on confinement, a few studies[41,48,49,54] have been carried out. Scott, Park, and Priestley,[43] based on the results in which, for high strain rate (0.0167/sec), the peak stress and the slope of the falling branch in the stress-strain curve were increased by about 25 percent compared with those for the low strain rate (0.0000033/sec), further modified the modified Kent and Park model. In deriving the stress-strain

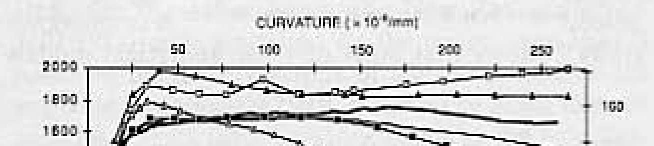

Fig. 4—Comparison of experimental results with the predictions from analytical models (Column A3MH-3)

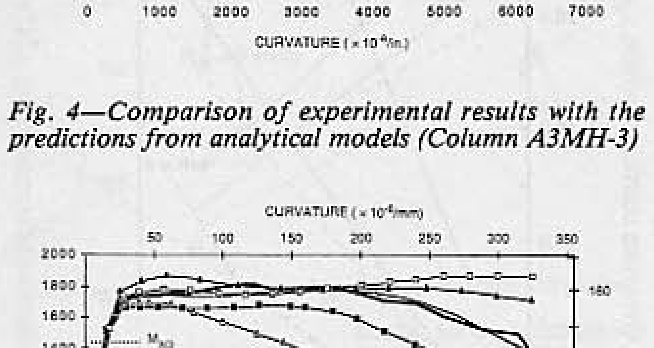

Fig. 5—Comparison of experimental results with the predictions from analytical models (Column D3MM-5)

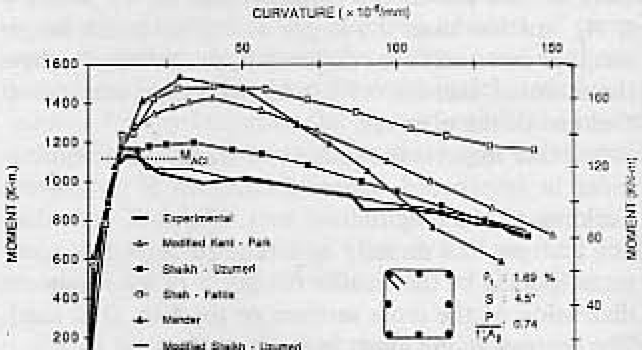

Fig. 6—Comparison of experimental results with the predictions from analytical models (Column E3MH-13)

curve of confined concrete from the test data, the load-carrying capacity of the longitudinal steel was based on the stress-strain curve of steel at low loading rate. More recently, the model by Mander[49] considered the effect of strain rate on both steel and concrete and showed good agreement with his experimental stress-strain curves of confined concrete at high strain rate (0.0167/sec). Fig. 7 shows that the Scott et al. model provides a very conservative prediction of actual behavior. Dilger, Koch, and Lowakzyk[48] have also proposed a stress-strain curve that includes the effect of strain rate. The model was derived from the test results in which the 6 x 6 x 24-in. (152 x 152 x 610-mm) specimens had only single hoops and no longitudinal reinforcement, and the behavior of concrete core and cover was not separated.

When reinforced concrete columns are subjected to combined axial and flexural loads, strain gradient exists in the sections. Chan,[21] Soliman and Yu,[27] and Sargin, Ghosh, and Handa[32] attempted to include the effect of strain gradient in the stress-strain models. There is little data that quantify the effect of strain gradient in comparison with the stress-strain relationships obtained from concentric compression tests. This may be attributed to the difficulties in tests and the problems concerning the method of treatment of test data. From the comparisons of the measured loads and moments and those calculated from the concentric stress-strain curve, Scott et al.[43] concluded that the presence of the strain gradient significantly improved the stress-strain curve for the concrete by reducing the slope of the falling branch of the curve. A comparative study of confinement by Sheikh[45] indicated that the application of the Sheikh and Uzumeri model to the specimens subjected to combined axial and flexural load provided conservative results at large curvature values. Sheikh[46] and Sheikh and Yeh[55] attributed the result to the existence of strain gradient and modified the Sheikh and Uzumeri model using previous test data.[21,31,32]

Supplementary crossties will be effective in restraining, directly or indirectly, longitudinal bars that are not supported by a corner of hoop reinforcement. Some of the crossties that are encountered in codes are shown in Fig. 8. It is obvious that the installation of Types A and B crossties is usually difficult. To facilitate the fabrication of reinforcing cages, the ACI code permits the use of Type D crossties, the effectiveness of which has been a controversial issue because the 90 deg hook is not anchored in the confined core. Supplementary crossties with a 135 deg hook at one end and a 90 deg hook at the other end were originally presented in the recommendation by ACI-ASCE Committee 352 for the design of shear reinforcement in beam-column joints.[76] Although Sakai, Kakuto, and Nomachi[53] did not treat Type D crossties, it was demonstrated that the crossties in general confine concrete under concentric compression as effectively as the closed hoops. Furthermore, Moehle and Gavanagh[52] have observed that crossties having 180 deg hooks are effective in confining concrete as intermediate hoops, and crossties having 135 deg and 90 deg hooks are nearly as effective. The effectiveness of crossties with 135 and 90 deg hooks may show a different trend under other circumstances, e.g., cyclic loading, combined flexure and high axial loads.

FLEXURAL STRENGTH

Recently, Priestley and Park[72] observed from the test results that the moment capacity of sections, confined according to the New Zealand code[7] provisions, was considerably larger than the capacity predicted by the ACI method, particularly for columns subjected to large axial loads. It was suggested that the increased capacity was caused by the enhancement in concrete strength due to confinement and the increased strength of steel in the strain hardening region, which is ignored in the ACI method. Fig. 9 shows the suggested curves along with the test data by Priestley and Park,[72] Soesianawati[71] and Sheikh, Yeh, and Menzies.[56] The amount of transverse reinforcement in the test specimens used by Soesianawati ranged between 16.7 and 45.8 percent of that required in the current New Zealand code.[7] The flexural strength enhancement in the test results by Sheikh et al. is much lower than that suggested by Priestley and Park. In fact, four columns did not even reach the ACI moment capacities. The amount of transverse reinforcement in these four columns was about half that required in Appendix A of the ACI code. Only one data point, in which the axial load was $0.46 f'_c A_g$ and steel configuration involved 12 laterally supported longitudinal bars, was within the 15 percent of the suggested value. It appears that at high axial load levels, when flexural capacity is strongly in-

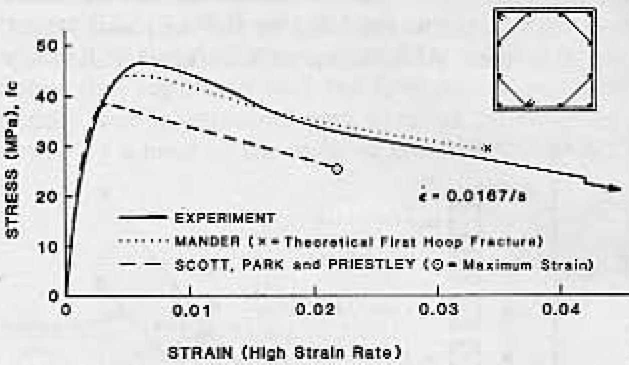

Fig. 7—Comparison of experimental and analytical stress-strain curves[49]

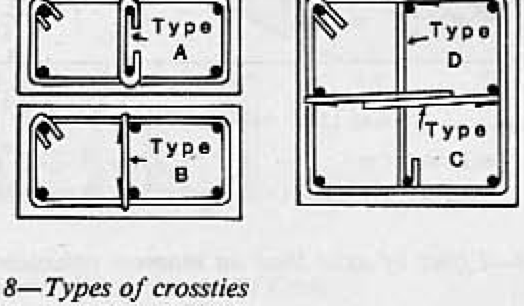

Fig. 8—Types of crossties

fluenced by the strength of concrete, the enhancement in flexural capacity can be expected only if a large amount of lateral reinforcement is used in a well-distributed configuration. The amount of reinforcement required in the ACI code is not enough to produce the strength enhancement of the magnitude as suggested in Fig. 9. In addition, the presence of heavy stubs near the test regions of the columns tested by Soesianawati[71] and Priestley and Park[72] are believed to have contributed significantly toward the strength enhancement of the column sections. Early start of strain-hardening of longitudinal steel in the case of cycling loading[71,72] may also be partially responsible for additional flexural capacity. It should be strongly emphasized that an accurate prediction of flexural strength is important to assess reasonably the curvature or displacement ductility and to design the member safely for shear.

MOMENT-CURVATURE RELATIONSHIPS

The object of early research on moment-curvature relationships concerning the members with confined concrete was mainly beams.[22,24,34,65-67] In 1972, Park and Sampson[68] provided comprehensive descriptions on the displacement ductility and curvature ductility of reinforced concrete columns for seismic design. It was suggested that the columns capable of reaching a curvature ductility factor ϕ_u/ϕ_y of at least 15, with ϕ_u defined as the curvature when the moment has reduced to 80 to 90 percent of the maximum moment, would appear to have adequate seismic resistance. From the moment-curvature analyses of confined concrete sections based on the Kent and Park[34] model in which no concrete strength enhancement due to confinement was considered, it was concluded that the amount of transverse reinforcement specified by the ACI and Structural Engineers Association of California (SEAOC) equations was conservative for low axial load levels and unconservative for high axial load levels. It should be emphasized that the theoretical moment-curvature curves were not compared with tests because of lack of such data. Based on this work, the SEAOC equations were modified to the present New Zealand code[7] provisions in which the adverse effects of axial load are taken into account. The present New Zealand code[7] provisions, although more comprehensive than those of other codes,[2,4,6] do not take all the important variables into account. In addition to the work already mentioned, a number of studies[42,44,45,49,50,55,56,58,70,71] on moment-curvature relationships have been carried out.

To examine the validity of the provisions for transverse reinforcement in the draft of the present New Zealand code,[7] Park, Priestley, and Gill[42] conducted four tests under combined axial load and flexure. The main variable in the tests was the level of axial load applied ($0.214\ A_g f_c' \sim 0.6\ A_g f_c'$) that caused the volumetric ratio of the transverse reinforcement to vary between 1.5 and 3.5 percent. The ACI Building code requires about 1.5 percent lateral reinforcement in these columns. Lower ductility was observed under high axial loads even when a larger amount of transverse reinforcement was used.

From the comparisons of the experimental data due to Park et al.[42] with the analytical results from various models, Sheikh[45] concluded that for low to moderate levels of axial loads, the envelope moment-curvature curve for reinforced concrete section under cyclic bending can be determined with reasonable accuracy by using the Sheikh-Uzumeri model.[44] Recently, Mander[49] used his model to predict the results of column tests by Park et al.[42] and found reasonably good agreement for specimens tested under combined axial load and cyclic flexure. Fafitis and Shah[50] also made a comparison of the test results due to Park et al.[42] and the analytical results using their own model for confined concrete. Soesianawati[71] carried out tests to investigate the applicability of a modified New Zealand code equation[7] in

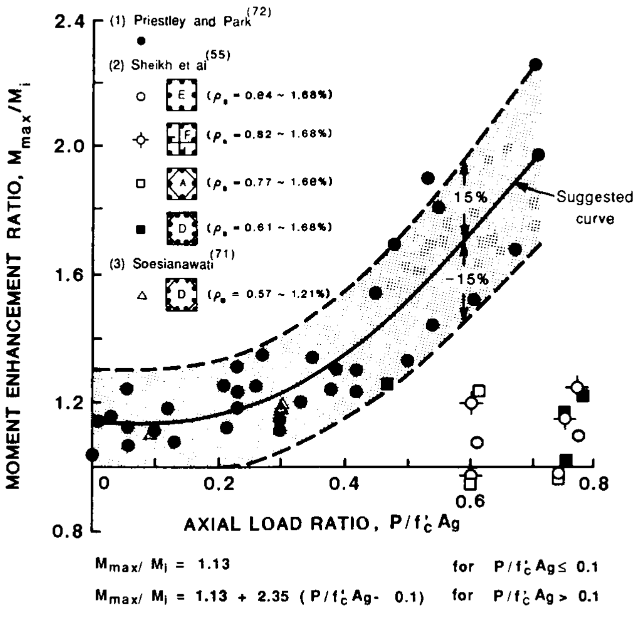

Fig. 9—Effect of axial load on moment enhancement ratio

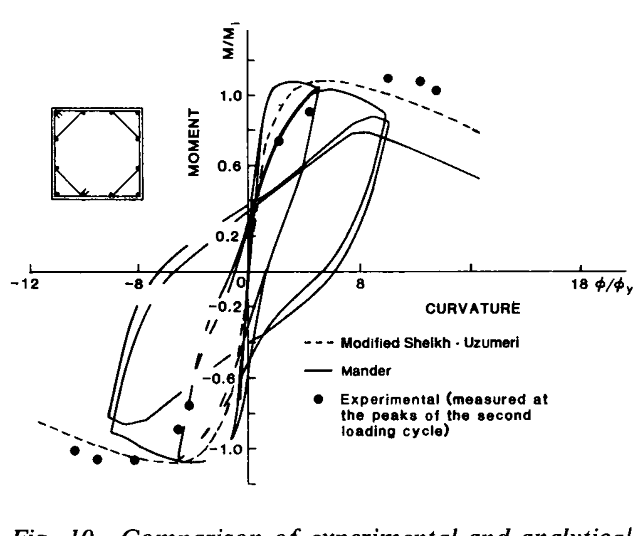

Fig. 10—Comparison of experimental and analytical moment-curvature relationship[72]

which the numerical coefficient of 0.12 in Eq. (7) is replaced by 0.08 to eliminate the inconsistency in the confinement effectiveness between rectangular and circular transverse reinforcement required in the code. Fig. 10 shows a comparison of experimental and analytical moment curvature relationships for a specimen tested under an axial load of 0.3 $f'_c A_g$. The amount of transverse reinforcement in this specimen was only 16.7 percent of that required in the current New Zealand code.[7] The comparison indicates that Mander's model under certain circumstances may not predict the results well. The applicability of the Mander model in another specimen, in which the amount of transverse reinforcement is 43 percent of that required in the current New Zealand code[7] and the level of axial load is 0.1 $f'_c A_g$, is good. The M–ϕ curve from the modified Sheikh-Uzumeri model[57] approximates the envelope curves quite well.

Fafitis and Shah[50] conducted a parametric study to assess the influence of concrete strength, degree of confinement, level of axial load, and the shape of the section on the capacity of columns subjected to large deformations. It was concluded that the ACI method of accounting for the influence of compressive strength or the amount of confinement seemed adequate for high-strength concrete columns. Sheikh and Yeh[55] also carried out moment-curvature analyses using the Sheikh and Uzumeri model which includes the effect of strain gradient. Based on their analyses, it was concluded that the ACI requirements may be either too conservative for columns with well-distributed steel or unsafe for columns with only four corner bars fully supported by a tie or both. It was suggested that confinement requirements should be a function of axial load level on the column. It has also been pointed out that the maximum tie spacing limit of 4 in. (102 mm) in the ACI code may be relaxed, and it should be related to the size of the confined core and the diameter of the longitudinal steel bar. Zahn[70] conducted cyclic moment-curvature analyses based on the model by Mander[49] and prepared curvature ductility charts. However, it should be emphasized that experimental verifications for a wide range of variables have not been adequate on the applicability of the model used.

Experimental and analytical studies have shown that moment-curvature behavior of columns depends on the amount of transverse reinforcement and the level of axial load. However, most of the experimental studies on moment-curvature relationships have been carried out at relatively low levels of axial load. The $P_c/f'_c A_g$ values were 0.214, 0.26, 0.42, and 0.6 in the tests by Park et al.[42] and were 0.1 and 0.3 in the tests by Soesianawati.[71] As shown in Fig. 2, codes allow considerably higher than tested levels of axial load. Sheikh et al.[56-58] conducted an experimental study involving high axial loads. Fig. 11 shows the relationship between the test parameters and the ACI code, Appendix A, requirements on the level of axial load and the amount of transverse reinforcement. The test specimens which fall in the shaded portion satisfy the ACI code requirement. The details of the test specimens, moment capacities, and the curvature ductility factors ($\mu_\phi = \phi_2/\phi_1$) obtained are given in Table 1. The value ϕ_1 is the curvature corresponding to the maximum moment on a straight line joining origin and a point corresponding to about 65 percent of the maximum moment on the ascending part of the M–ϕ curve. The curvature ϕ_2 corresponds to about 90 percent of the maximum moment on the descending part of the curve. The required transverse reinforcement ratio for these specimens according to the ACI code is approximately 1.5 percent.

The test specimens strictly under the ACI code requirements have shown satisfactory curvature ductility factors (22.0 ~ 40.0) except for Specimen E4MH-2 in which μ_ϕ was 10. The specimen E4MH-2 had single hoops that supported only the four corner bars. The middle four longitudinal bars were not laterally supported by a bend of the hoop or crossties. In the ACI

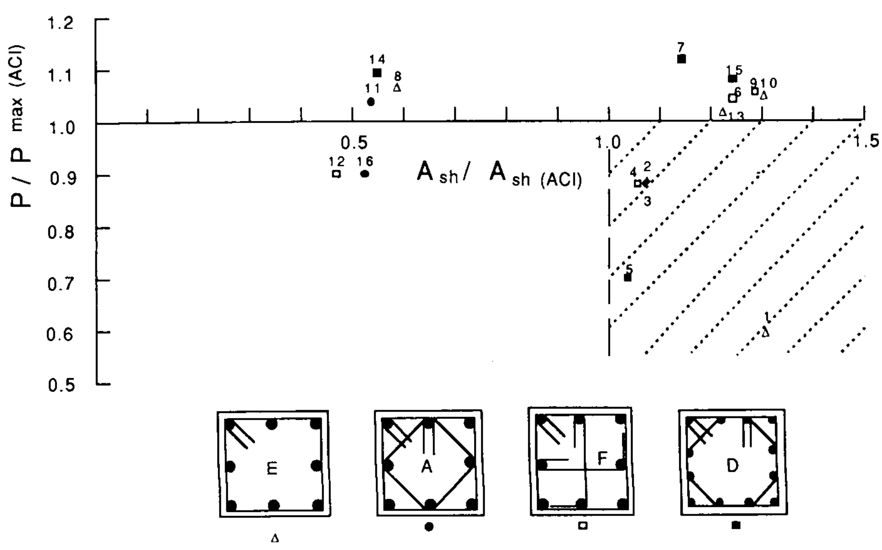

Fig. 11—Relationships between the test parameters[56,58] and the ACI Code

code the use of such a single hoop has been allowed. The amount of transverse reinforcement in Specimen E4MH-2 is more than that required in the ACI code, and the axial load is approximately 88 percent of the maximum design axial load. The results suggest that the column sections with single hoop and laterally unsupported longitudinal bars may not have adequate ductility even if the code requirements are satisfied. On the other hand, Columns A3MH-3 and F4MH-4 with the same amount of reinforcement as Column E4MH-2 and tested under similar conditions showed better ductility. The column with Configuration A performed in a more ductile manner than Column F in which 90 deg hooks were used. With respect to the ductility under reversed cyclic loading, however, further tests are required.

The second group of test specimens satisfied the ACI requirements on the amount of transverse reinforcement but exceeded the allowable design axial load limit by up to 11 percent. The curvature ductility factors ranged from 3.0 to 9.5 except for Specimen D1MH-7, which had the tie spacing of 2⅛ in. (54 mm) in which μ_ϕ = 14.0. On the basis of these test results, it can be concluded that columns designed according to the ACI code and tested under axial loads equal to or only slightly larger than the maximum allowable design axial load do not exhibit satisfactory ductility. It should be noted that the limit on the axial load set by the code is quite arbitrary considering the uncertainty of forces during an earthquake. The third group of the test specimens, in which the amount of transverse reinforcement is about half that required in Appendix A of the ACI code, also did not indicate adequate ductility. However, these tests results indicate that even if the amount of transverse reinforcement required in the ACI code is reduced to half, appropriate ductility may be obtained if the axial load is small and steel is detailed appropriately. Another important observation can be made for this group of specimens with respect to the moment capacity. Except for Column 14, no specimen reached the theoretical section moment capacity calculated according to the ACI procedure for unconfined concrete. It should be noted that these four columns satisfy the nonseismic design requirements of the ACI code.

BEHAVIOR OF COLUMNS

As an extension of the studies on the characteristics of cross sections, research on the general behavior as a member has begun recently. Park et al.,[42] from their tests on four full-size reinforced concrete columns under reversed cyclic lateral load and constant axial compression, concluded that the amount of transverse reinforcement according to the draft of the current New Zealand code[7] enabled columns of the type tested to reach a curvature ductility factor of approximately 20 and a displacement ductility factor approaching 10. The displacement ductility was assessed on the basis of the yield displacement that was obtained from the intersection point of the horizontal line at the theoretical ultimate load and the straight line from the origin passing through the point on the measured load-displacement at 0.6 of the theoretical ultimate load, based on the ACI method. In the Moment-Curvature Relationships section, it has been shown that the actual flexural strength is considerably larger than the theoretical strength, especially in the case of high axial load if the section is heavily confined (Fig. 9). Due assessment of the displacement ductility factor, which is based on the actual

Table 1 — Details of test specimens and some results[56-58]

Specimen	f'_c, ksi	Longitudinal steel ratio, percent	Spacing, in.	Transverse steel ratio, percent	Axial load ratio, P_e/f'_cA_g	Curvature ductility factor, μ_ϕ	$\frac{M}{M_{ACI}}$
E4MM-1	4.45	2.08	4.00	1.74	0.40	*	1.08
E4MH-2	4.55	2.44	4.50	1.69	0.61	10.0	1.23
A3MH-3	4.61	2.44	4.25	1.68	0.61	40.0	1.22
F4MH-4	4.67	2.44	3.75	1.68	0.60	30.0	1.26
D3MM-5	4.53	2.58	4.50	1.68	0.46	22.0	1.15
F4MH-6	3.95	2.44	6.81	1.68	0.75	3.5	1.15
D1MH-7	3.80	2.58	2.13	1.62	0.78	14.0	1.22
E4SH-8	3.76	2.44	5.00	0.84	0.78	3.0	0.96
F3MH-9	3.84	2.44	3.75	1.68	0.77	5.0	1.25
E1MH-10	3.81	2.44	2.50	1.68	0.77	4.5	1.10
A3SH-11	4.05	2.44	4.25	0.77	0.74	8.5	0.97
F2SM-12	4.86	2.44	3.50	0.82	0.60	8.0	0.98
E3MH-13	3.95	2.44	4.50	1.69	0.74	8.0	1.01
D3SH-14	3.90	2.58	4.25	0.81	0.75	3.0	1.01
D3MH-15	3.80	2.58	4.50	1.68	0.75	9.5	1.17
A3SH-16	4.92	2.44	4.25	0.77	0.60	10.5	0.95

*Not available due to lack of control of loading.
1 in. = 25.4 mm; 1 ksi = 6.9 MPa.

flexural strength, will reduce considerably the value, particularly when the actual strength is significantly larger than the ACI theoretical strength. Therefore, a definition of yield displacement based on the actual moment capacity rather than on the ACI moment capacity appears more appropriate and is applicable to both cases of high and low axial load levels.

Mander[49] carried out tests on four hollow columns that had an overall height of 4.1 m (161 in.) and a 750 mm (29.5 in.) square hollow cross section with a 120 mm (4.72 in.) wall thickness. The axial load applied to the specimen was $0.1 f'_c\ A_g$, $0.3 f'_c\ A_g$, and $0.5 f'_c\ A_g$. It was concluded that the provisions of the New Zealand code can be used to detail the transverse reinforcement in the flanges of hollow columns in the same manner as for solid column members. It was also suggested that the full quantity of hoop steel according to the New Zealand code may be excessive if only limited ductility is required. Because of the reduced weight, hollow columns may be advantageous for bridge piers that are usually subjected to low axial loads.

Zahn[70] conducted tests on six 16 in. (400 mm) square reinforced concrete columns to investigate the effects of the direction of flexural load and the strength of lateral steel on the column behavior. The axial load in the columns loaded along the section diagonal ranged from $0.23 f'_c\ A_g$ to $0.42 f'_c\ A_g$. It is difficult to assess the effect of the direction of applied load on ductility because of a lack of data from similar specimens loaded in the direction of a section principal axis. Results, however, indicated that smaller quantities of higher strength steel can be used without adversely affecting the column behavior.

Soesianawati[71] tested four 16 in. (400 mm) square reinforced concrete columns under cyclic lateral load with axial load ranging between $0.1 f'_c\ A_g$ and $0.3 f'_c\ A_g$. It was stated that the specimens with 43.1 and 45.8 percent of the amount of transverse reinforcement recommended by the New Zealand code[7] achieved a displacement ductility factor of at least 8 without significant strength degradation and the specimens with 30.4 and 17 percent of the code-required amount of transverse steel were capable of reaching a displacement ductility of 6 and 4, respectively. The ductility factors were based on the theoretical capacity of the column section, ignoring the effect of confinement. As stated earlier, this would overestimate the ductility factors compared with the ones based on the actual capacity. Unlike curvature ductility factors, displacement ductility factors may not provide a common indication to assess the ductility of columns in various situations. In the column subjected to high axial load, the P-Δ effect is large. With the increase of secondary moment, the external horizontal force H must be reduced because of the limited cross-sectional strength. In this case, the commonly used definition of displacement ductility factors may not have any significance.

To examine the effectiveness of supplementary crossties under cyclic lateral loading, Tanaka, Park, and McNamee[73] conducted tests on four 16 in. (400 mm) square reinforced concrete columns. The axial load was $0.2 f'_c A_g$. Lateral steel arrangements involved perimeter hoops with 135 deg hooks, crossties with 90 and/or 180 deg hooks, and crossties and perimeter hoops with tension splices. Although the effectiveness of crossties with 90 deg hooks under reversed cyclic loading was found satisfactory in this study, it should be noted that the axial load level was very low. Columns with crossties having 180 deg hooks at both ends or "J" ties also showed satisfactory behavior. However, the columns with tension splices and "J" ties exhibited inferior behavior compared with other columns.

Rabbat et al.[63] carried out tests on 16 lightweight and normal weight concrete specimens that represented a portion of the building frame at the joint between columns and beams. The columns were 15 in. (381 mm) square and 15 x 20 in. (381 x 508 mm) in section. The cyclic loading was applied to the columns by the reversal of the moments in the beams. The supplementary crossties with a 135 deg hook at one end and a 90 deg hook at the other end were used. It was suggested that current confinement requirements of ACI 318-83[2] for normal weight concrete columns can be extended to lightweight concrete columns with axial loads up to 30 percent of the column design strength. It was also concluded that supplementary crossties engaging the column steel bars performed very satisfactorily in confining the column core. Test results indicated that strength degradation became larger with the increase of column axial load suggesting that, for columns subjected to high axial loads, these conclusions may not be valid and the columns may show unacceptable behavior.

Johal, Musser, and Corley[74] summarized test results from 18 in. (457 mm) square specimens tested under cyclic flexure while simultaneously subjected to axial loads in the range of 20 to 40 percent of the cross-sectional strength. Five transverse reinforcement detailings were used: Detail A = peripheral and inner hoops with 135 deg hook bends; Detail B = peripheral hoop with 135 deg hook bend and inner hoop with 90 deg hook bend; Detail C = overlapping peripheral hoop with 135 deg hook bend and inner hoop with 90 deg hook bend; Detail D = peripheral hoop with 135 deg hook bend; Detail E = peripheral hoop formed with four identical ties with 45 deg bends at both ends. The following observations were made from the tests: flexural capacity of a column increased with axial load but ductility reduced substantially; use of almost 50 percent less transverse reinforcement resulted in slightly lower ductility; flexural capacity and ductility were not reduced by the use of overlapping peripheral hoops, 90 deg hooks on inner hoops, or special hoops (Detail E); and the use of single peripheral hoops resulted in lower flexural strength.

Ozcebe and Saatcioglu[77] recently reported test results of four 13.8 in. (350 mm) square columns that represented a portion of a first-story column between the foundation and the inflection point and were subjected to constant axial load (20 percent of column design

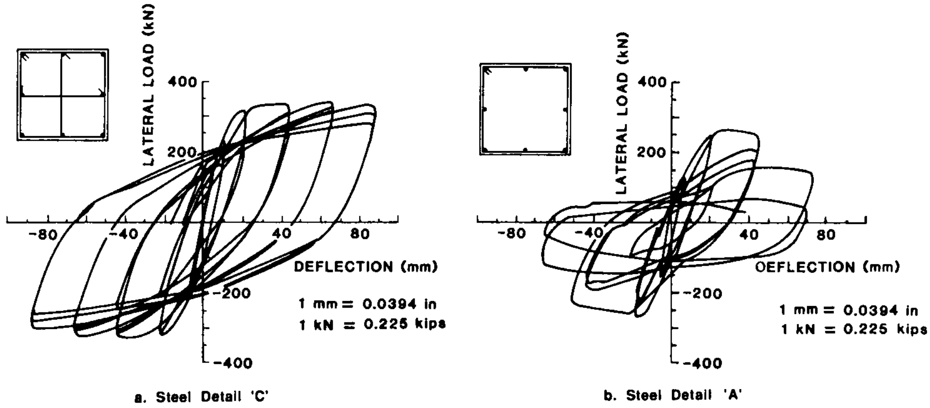

Fig. 12—Hysteresis loops for column with and without crossties

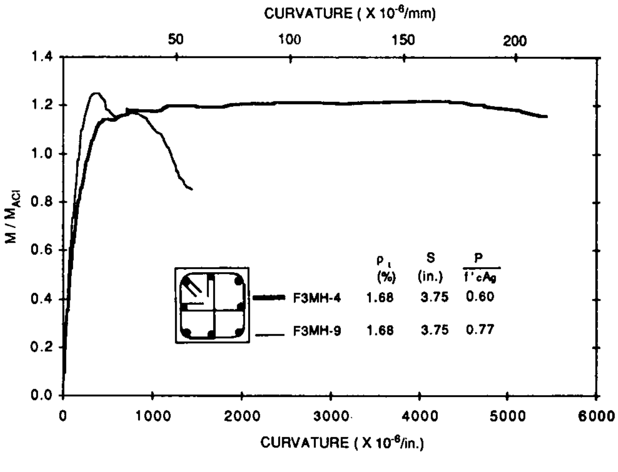

Fig. 13—Effect of axial load on columns with crossties[58]

strength) and lateral load reversals. Three tie configurations were used: (A) perimeter hoop only with 135 deg hooks; (B) perimeter hoop and crossties with 135 deg hooks at both ends; (C) perimeter hoop and crossties with a 135 deg hook at one end and a 90 deg hook at the other. From the results of the two specimens shown in Fig. 12, it is obvious that the column with perimeter hoops only behaves poorly compared with the column with the perimeter hoops and crossties. Test results indicated that the ACI detailing requirements are not adequate for columns with unsupported longitudinal bars, although the ACI requirements for the amount of confinement reinforcement appear adequate. The crossties with 90 deg hooks performed as well as those with 135 deg hooks.

A few experimental studies involving crossties with 90 deg hooks presented a positive view on their effectiveness under combined axial load and flexure. However, it should be noted that the axial load levels were relatively low in all these tests, mostly in the range below the balanced load. The experimental study by Sheikh, Yeh, and Menzies[56] and Yeh and Sheikh[57-58] exhibited the apparent inferiority of crossties with 90 deg hooks under high axial loads. Fig. 13 compares the results of two columns to highlight this point. Since cages with 90 deg hooks are easy to fabricate, their use is becoming popular. However, conditions under which 90 deg hooks can be used reliably should be clarified through further research.

In general, the loading (or displacement) histories used in the tests are different from those caused in an actual earthquake. The load-carrying capacity of a section or a column at a certain displacement decreases with an increase in the number of cycles, i.e., cyclic loading causes stiffness degradation. Therefore, in assessing the displacement ductility or the curvature ductility, the cycle number at the same displacement may have a significant effect. Nevertheless, no data is available on this aspect. The relationships between the load history due to an actual earthquake and the static simulation must be investigated.

The plastic hinge length is related directly to the extent to which a member should be confined and is needed to assess the relationships between curvature ductility and displacement ductility. Generally, in reinforced concrete columns, the equivalent plastic hinge length is calculated from the curvature distribution and deflection measured in the tests. Therefore, the effects of yield penetration along the longitudinal bars and the cracking due to the shear are included in the plastic hinge length. On the basis of their test results, Priestley and Park[72] suggested the following equation

$$L_p = 0.08\,L + 6\,d_b \qquad (8)$$

Test data from square, circular, and square hollow columns were used to derive Eq. (8). It was shown that the average plastic hinge length for all tests was close to 0.5 D (or 0.5 h). Fig. 14 plots the experimental plastic hinge length against the axial load levels for columns tested by various researchers. It is obvious that the plastic hinge length tends to increase with the increase of the axial load levels. The plastic hinge length obtained from Soesianawati's tests[71] ranged between 46 and 73 percent of that calculated using Eq. (8). Larger plastic hinge lengths were observed for larger amounts of transverse reinforcement. With the small amount of transverse reinforcement, the flexural strength enhancement is small, and hence the spread of the plastic hinge region is small. Zahn[70] suggested the following equations for plastic hinge lengths based on his own test data

For $$\frac{P_e}{f'_c A_g} < 0.3$$

$$L_p = (0.08\,L + 6\,d_b)\left(0.5 + 1.67\frac{P_e}{f'_c A_g}\right) \tag{9}$$

For $$\frac{P_e}{f'_c A_g} \geq 0.3$$

$$L_p = 0.08\,L + 6\,d_b \tag{10}$$

Fig. 15(a) and (b), drawn from the data in Reference 72, indicates that the plastic hinge length increases with an increase of column aspect ratio. However, it seems that there is a limitation on this proportionality at the column aspect ratio of approximately 4. Bilinear curves were also included in Fig. 15 to suggest such a trend. Thus, it may be concluded that the plastic hinge length is affected by the amount of transverse reinforcement, the axial load level, and the column aspect ratio. Furthermore, available test data suggests that the current code provisions for the extent to be confined are conservative.

It was a general understanding in the 1956 ACI Building Code[78] that ties should be provided to reduce the possibility of local buckling of the longitudinal bars under the load approaching the ultimate strength of the column. The 1956 ACI code required tie spacing not to exceed 16 longitudinal bar diameters, 48 tie bar diameters, or the least dimension of the column. This provision has been used in the current ACI code[2] for columns not designed for seismic loading. For seismic loading, a maximum limit of 4 in. (102 mm) or one-quarter of the minimum section dimension has been added to these restrictions. A few studies[79-81] to examine the 1956 ACI code requirements were carried out. Bresler and Gilbert[79] theoretically studied the effect of tie spacing on the buckling of longitudinal bars at a stress below yield and the effect of the ratio of tie bar diameter to longitudinal bar diameter on the buckling strength of the longitudinal bar. Pfister[80] concluded from the experimental observations that the main function of ties is to restrain the concrete laterally rather than to prevent the buckling of longitudinal bars. Hudson[81] carried out the tests to investigate the effect of tie spacing on ultimate strength. It was suggested that when designing for seismic loads, it is probable that the code requirements are too liberal.

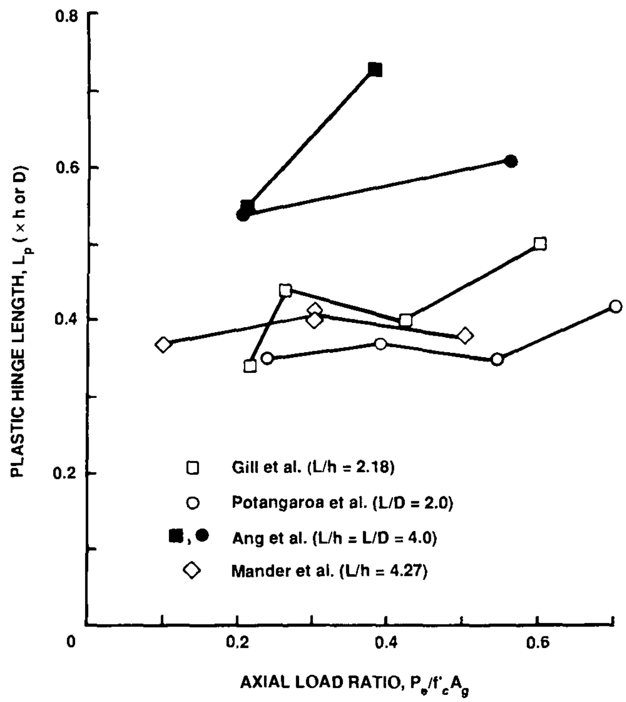

Fig. 14—Effect of axial load ratio on plastic hinge length

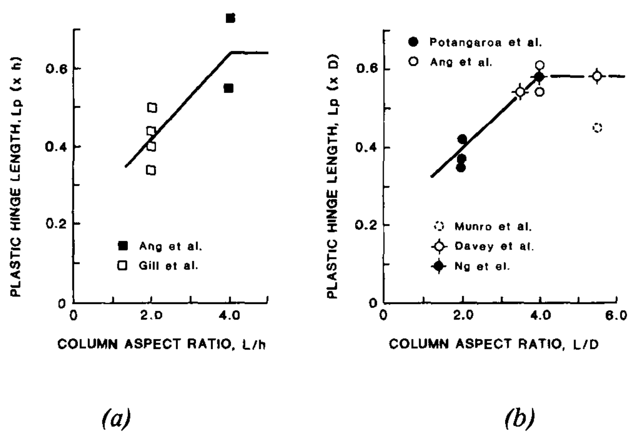

Fig. 15—Effect of column aspect ratio on plastic hinge length

Scribner[82] conducted a theoretical and experimental study to examine the relationships of the flexural stiffness of a confining tie to the inelastic buckling of longitudinal reinforcement in a beam subjected to repeated reversed inelastic flexure. The buckling of longitudinal bars not supported by the corners of transverse reinforcement was treated. From the theoretical analysis, it was shown that the buckling of longitudinal bars can be influenced by tie size and spacing. Furthermore, on the basis of the test results, it was stated that even large, closely spaced ties provide no guarantee against the buckling of unsupported bars in a beam subjected to very large flexural displacements. The tests suggest that from the viewpoint of buckling, the columns with only four corner bars fully supported by a single hoop are undesirable. On the basis of the test results in which most of the specimens had the tie spacing less than 6 d_b, Priestley and Park[72] have contended the appropriateness of 6 d_b as a maximum tie spacing.

Lukose, Gergely, and White[83] carried out tests to evaluate the effects of transverse reinforcement, splice length, bar spacing, bar size, and loading history on the performance of lapped splices in beams and column-type specimens subjected to high-level repeated or reversed cyclic flexural loads. The motivation behind this study was to investigate the code provision in which lapped splices are not allowed at locations of inelastic deformations. It was shown that to achieve satisfactory performance of the lapped splices under repeated and reversed cyclic loads amounting to at least 80 percent of the monotonic failure load, stirrup areas close to twice the maximum effective amount specified in the proposed ACI Committee 408 report[84,85] are needed. Furthermore, it was concluded that lapped splices can be designed under most conditions in regions where flexural yielding or severe stress reversals are anticipated. Sivakumar, Gergely, and White[86] presented a design recommmendation for lapped splices in which a minimum of 15 to 20 reversing load cycles beyond yield and a maximum strain of at least 2.5 times the yield strain were considered as an indication of satisfactory performance of the member. Paulay,[87] on the basis of the test results, concluded that although plastic hinges can be developed with excessive transverse reinforcement around lapped splices, splices should not be used in the potential plastic hinge region of columns, since yielding of the reinforcement would be restricted to a very small length that causes extremely large steel strains, local buckling, and in some cases fracture of the main bars.

The difference in the conclusions of the two studies[83,87] on the use of lapped splices at locations of inelastic deformations may be attributed to that of the type of test specimens and the amount of imposed deformations. In fact, the tests by Lukose et al.[83] suffered the stroke limitation on the loading actuators; while in the tests by Paulay,[87] the displacements corresponding to the displacement ductility factors of ± 2, ± 4, and ± 6 were imposed.

CONCLUSIONS

A comprehensive review on confinement of reinforced concrete columns has been presented. Several debatable issues were identified systematically, ranging from the effects of different variables on the mechanism of confinement to the behavior of a section and that of the column to the possibility of plastic hinging in columns.

The present ACI code[2] provisions for confining steel are based on the philosophy of maintaining the axial load-carrying capacity of a column section. In practice, the confinement is required to produce ductile behavior of the structural members subjected to a combination of forces. The performance in terms of strength and ductility, expected of a column during a severe earthquake, is not well defined in the literature. Lacking this information, it is difficult to propose a specific design for concrete confinement. An approach that relates the behavior of a column to the parameters comprising the lateral confinement seems more appropriate so that an individual designer can choose the extent to which the members need confinement. Based on the review of the previous research, it appears that a reexamination of the ACI code provisions for confinement will be needed at least in the following five areas: 1) distribution of longitudinal and lateral steel, particularly keeping in view the undesirable behavior of columns with single peripheral hoops; 2) amount and spacing of transverse reinforcement; 3) level of axial load; 4) crossties with 90 deg hooks; and 5) zone of inelastic deformations (plastic hinge length). The current ACI code provisions governing these areas may result in insufficient ductility in columns under certain situations, especially under the high levels of axial load that are within the permissible limits.

Experimental evidence suggests that columns with single hoops and 90 deg hooks may not provide sufficient ductility, particularly when they are subjected to high axial loads and cyclic flexure. These reinforcement details have obvious advantages for ease of construction. Therefore, the usage of such transverse reinforcement might be allowed in sections where only limited ductility is required under low-to-moderate levels of axial load.

Further research is needed to study the performance of columns with crossties with 90 deg hooks under cyclic flexure and high axial load. Several variables, such as steel configuration, amount of tie steel, spacing of ties, and level of axial loads, have been studied recently for their effects on the behavior of normal concrete. Similar investigations for high-strength and lightweight concretes are also needed. Effects of these variables on the length of plastic hinges also need investigation. Along with the fundamental study of several issues discussed in this paper, a comprehensive experimental study is needed that aims at the development of a rational procedure for the design of confinement required for a certain performance of a section and the column.

ACKNOWLEDGMENTS

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NOTATION

A_c = area of core of spirally reinforced column
A_g = gross area of section
A_{ch} = area of core bound by rectilinear ties
A_{sh} = total area of rectilinear transverse steel at a section
d_b = longitudinal bar diameter
D = diameter of circular column
f'_c = compressive strength of concrete in a standard cylinder

f_{cc} = compressive strength of confined concrete
f_{cp} = compressive strength of plain concrete
f_l = lateral pressure
f_y = yield stress of steel
h = lateral dimension of rectangular column section
h_c = cross-sectional dimension of core
L = column length between point of contraflexure and the point of maximum moment
L_p = plastic hinge length
P_e = axial force on the column
s = spacing of ties
ρ_s = volumetric ratio of lateral steel-to-concrete core
ϕ = capacity reduction factor

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