IMPROVED CONFINEMENT MODEL FOR BEHAVIOUR OF REINFORCED CONCRETE CIRCULAR BRIDGE COLUMNS UNDER FLEXURE DOMINANT MODE

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Department of Civil Engineering

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This thesis entitled "*Improved Confinement Model for Behaviour of Reinforced Concrete Circular Bridge Columns under Flexure Dominant Mode*" by SUREPALLY NARESH is approved for the degree of Master of Technology from IIT Hyderabad.

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Dedicated to

My Family.

Abstract

This study presents an improved analytical approach for the analysis of reinforced concrete (RC) circular columns by proposing a new confinement model. Three widely used confinement models (Sheikh and Uzumeri 1982, Saatcioglu and Razvi 1992 and Mander and Priestly 1988) are examined and improvements are suggested. It is well established that stress-strain behaviour of confined concrete is completely different from that of plain concrete. The level of confinement depends on the (i) amount of transverse reinforcement, (ii) amount of longitudinal reinforcement and spacing of rebar and (iii) level of axial load. The influence of these parameters on the section and member level behaviours are analysed and an improved model is proposed for analysis of RC circular columns under flexure. Predictions of the analytical models considered in this study were compared with the experimental data of two columns tested by the authors and others from PEER DATABASE. Experimental data of eight full scale columns tested under flexure-shear is used for the evaluation of existing and the proposed models over a range of parameters. Evaluation of test results indicate a very good agreement of the prediction of the proposed model with the test data.

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CHAPTER 1

INTRODUCTION

1.1 BACKGROUND

It is uneconomical to design buildings and bridges to withstand lateral forces for severe and infrequent earthquakes using elastic analysis. An alternative and widely accepted approach is to design structures to have sufficient energy dissipation capacity by the formation of plastic hinges so that collapse is prevented. Experience and research on reinforced concrete bridges indicate that relatively stable response to strong ground motions can be obtained if the system is proportioned and detailed so that the predominant inelastic response is restricted to flexure in the column. In this case, it is important to know the strength of the column so that strengths of adjacent components can be set high enough to avoid inelastic action in those components. Details also are required that will enable the column to sustain the necessary inelastic deformations without disabling loss of resistance. Of interest are the configuration and amount of the transverse reinforcement required to sustain expected earthquake demands. The formation of hinges in columns is undesirable, as this may result in the formation of a weak story mechanism in buildings. For this reason, most seismic design codes attempt to ensure having hinges in the beams rather than the columns. It is more desirable to have plastic hinges in beams than in columns to ensure large energy dissipation in buildings in a safe manner. However, from a practical standpoint, it is not possible to prevent the formation of plastic hinges in the first-story columns of a multi-storey buildings during a strong earthquake (Figure.1.1). Moreover, bridges are designed as weak column and strong floor mechanism and it is expected to have the plastic hinges in the columns. In order to have sufficient energy dissipation in columns, the concrete stress strain behaviour should be very ductile with high strength. Therefore, columns have to be detailed appropriately to have enough confinement as this increases ductility and strength of concrete. But, the confinement effects are not same throughout the length of columns because of strain gradients under combined axial-flexure-shear loading. Hence, the overall aim of this project is:

(i) To evaluate the effect of different confined concrete models at the sectional level and its effect on global behaviour of bridge columns failing under flexure-shear mode.

(ii) To improve the existing models for predicting the behaviour of RC columns under flexure-shear mode to accurately calculate the ultimate strength, stiffness and displacement capacity due to increased shear effects

Figure 1.1 Plastic hinge in column (Source: earthquake.usgs.gov)

1.2 PROBLEM STATEMENT

Reinforced concrete columns experiences large displacements under cyclic lateral loads while supporting gravity load. Hence, severe damage occurs at regions where large inelastic curvatures are observed. The severely damaged region is called the plastic hinge region. Hence, the column needs to have more energy dissipation capacity through high strength and ductile stress strain curves. To achieve the aim of this project, the behaviour of concrete columns is evaluated at two different levels: (1) sectional-level behaviour, which is a local response at the plastic hinge region where the inelastic deformations are concentrated, and (2) member-level behaviour, which is an overall column response. Two different stress strain curves for concrete will be considered at the sectional level as shown in Figure 1.2. One is for the core region which is confined and other one is unconfined stress strain curve for the outer core. For steel reinforcement, elastic–plastic–strain hardening behaviour will be considered. At the member level, to trace the actual behaviour of the column the slip of the longitudinal bar will be considered along with higher axial load effects and different length to depth ratios. The flow of the Analysis is shown in Figure.1.2.

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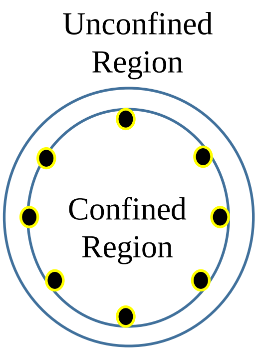
<td>
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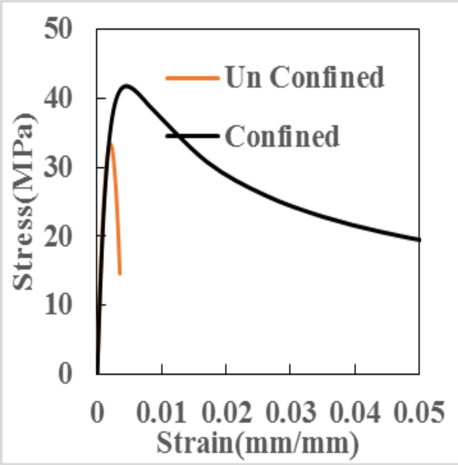
<td>
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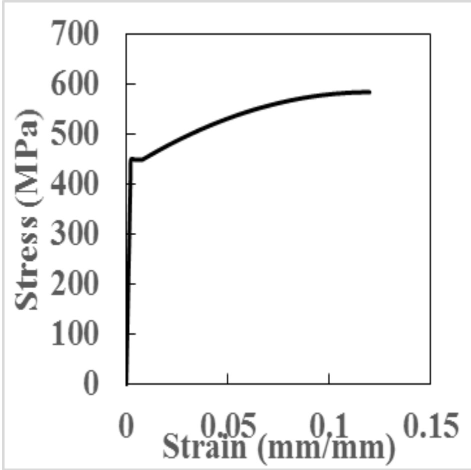
<td>
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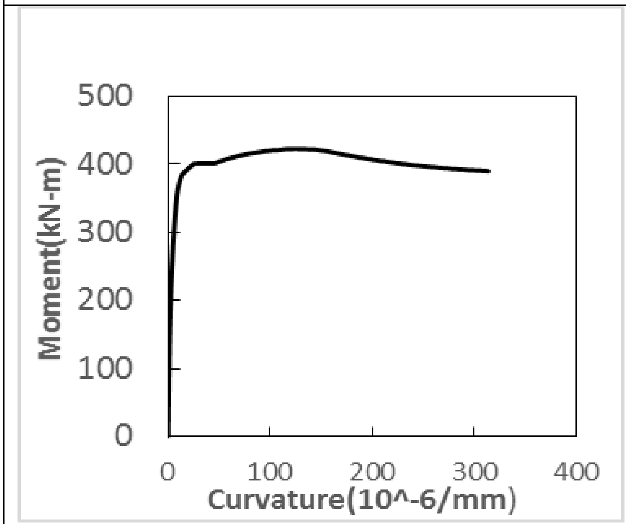
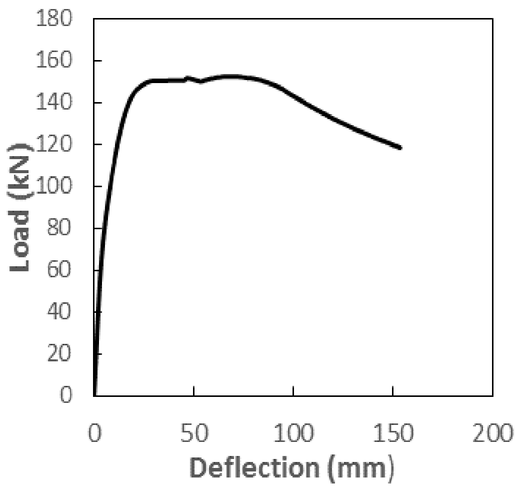
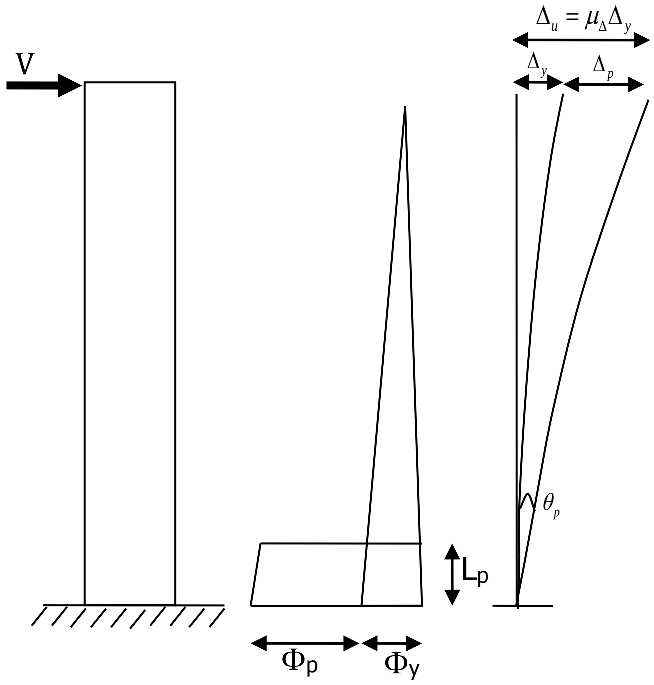
(a) Input Details

M-ϕ Curve (Sectional Analysis)	Load-Δ_{tot} Curve (Member Analysis)
	

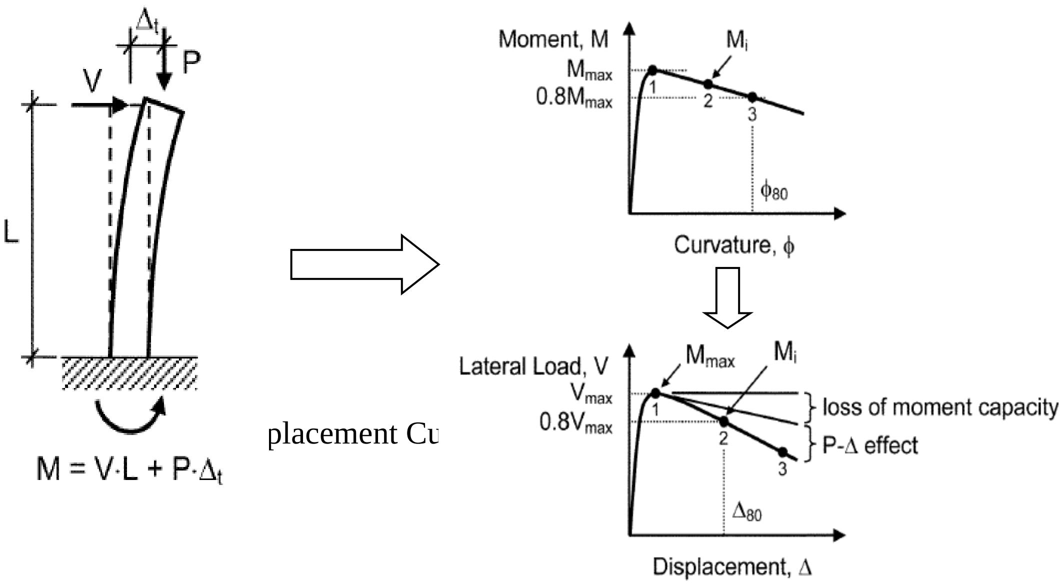
(b) Sectional and Member-level Analysis

Figure 1.2. Section to Member Level behaviour

Seismic evaluation of RC members under flexural loads requires a detailed representation of the complete hysteretic load-displacement relationship. Available models for hysteresis analysis include fiber, lumped-plasticity, and multilinear force-displacement. Simplified model to approximate the displacement capacity of RC member includes the plastic-hinge method. Moment-curvature analyses commonly form the basis for assessing the nonlinear force displacement response of a particular RC cross section. Plastic-hinge analyses require assumptions about the plastic zone in a structural member to calculate plastic rotations and displacements based on plastic curvatures. They can be enhanced by accounting for shear displacements and end rotations resulting from strain penetration into the footing or bentcap. The equivalent plastic-hinge analysis (Park and Paulay, 1975; Priestely et al., 1996) is a popular method for assessing plastic rotation which strongly influences ductile seismic design. This method assumes a given plastic curvature lumped at the center of an equivalent plastic-hinge. The plastic-hinge length is the length over which this plastic curvature is integrated to solve the total plastic rotation which is assumed to be constant . Under flexure, the displacement ductility can be derived using the moment curvature relationship and the assumed length (Priestley et al., 1996). Using a plastic-hinge concept and the second moment area theorem, Park and Paulay proposed an expression for the tip displacement of a column, which is expressed in Equation 1.1. From this equation, they further derived a relationship between curvature ductility and displacement ductility, as shown in Equation 1.2. The latter equation indicates a linear relationship between the curvature and displacement ductilities of the columns. The plastic-hinge length l_p and the column height L are two important factors influencing this relationship. The flexural displacement distribution is essentially linear until the yielding of the longitudinal bars on the tension side; thereafter, it becomes nonlinear. The yielding of longitudinal reinforcement and the subsequent crushing of cover concrete result in the formation of a flexural plastic-hinge. Well confined columns tested under flexure (single curvature) typically form a plastic-hinge in the bottom portion where the bending moment is greatest. The total flexural displacement of the column under flexure can be expressed as the sum of yield displacement and plastic displacement:



(a) Loading and Curvature Distribution



(b) Moment Curvature and Load-Displacement

Figure 1.3. Process of arriving the load deflection behaviour

$$\Delta_t = \Delta_y + \Delta_p = (\phi_u - \phi_y) l_p (L - 0.5 l_p) \quad \text{...} \quad (1.1)$$

where Δ_t is the total displacement, Δ_y is the yielding displacement, Δ_p is the plastic displacement, l_p is the length of the plastic-hinge, L is the length of the column, Φ_u is the curvature at ultimate moment, and Φ_y is the curvature at yield moment.

As demonstrated by Priestly et al. (1996), the displacement ductility can be expressed in terms of curvature ductility:

$$\mu_\Delta = 1 + 3(\mu_\phi - 1)\frac{l_p}{L}(1 - 0.5\frac{l_p}{L}) \quad \ldots\ldots (1.2)$$

where μ_Δ is the displacement ductility and μ_Φ is the curvature ductility. Further, the estimation of flexural displacement using the above equations depends on the accuracy of the plastic-hinge length calculations. The M-ϕ Curve will be obtained through the constitutive relationships as shown in Figure.1.3. After getting the Sectional behavior this data is used to predict the Load Deflection curve (Figure.1.3). In member level analysis deflection contribution due to flexure ($\Delta_{flexure}$), shear (Δ_{shear}), slip (Δ_{slip}) and P-Delta ($\Delta_{p\text{-}delta}$) effect need to be considered.

$$\Delta_{tot} = \Delta_{flexure} + \Delta_{slip} + \Delta_{shear} + \Delta_{p\text{-}delta} \quad \ldots\ldots (1.3)$$

1.3 OBJECTIVES AND SCOPE

Considerable amount of experimental research has been performed regarding the behavior of reinforced concrete columns. However, very little research has been performed on evaluation of existing confinement models of different aspect ratios and their applicability to predict the overall force-displacement response. Of particular interest are the extent of inelasticity along the column height (and, hence, the length along which confinement is required), the achievable ductility under reversed cyclic loading, and the applicability of current design tools (which are largely based on the experimental results of short columns) toward predicting the load-displacement response of columns with higher aspect ratio. The sectional behavior of RC column depends on a) arrangement of longitudinal reinforcement, b) The amount of transverse reinforcement and c) axial load levels. Hence, the above factors need to be studied thoroughly. When it comes to Member level, the factors such as Slip, H\D ratio, Plastic hinge length and P-Delta effects should be included in the analysis(Table1.1). The existing models are applicable only for flexure dominated behavior. Their validity for columns failing in flexure-shear mode is not explored so far. Hence, the objectives of the work are:

- To improve the understanding in sectional behavior (Part-1) by considering different confinement models for columns failing in flexure and flexure-shear mode
- To develop an improved model by considering the effect of factors such as Slip, H\D ratio, Plastic hinge length and P-Delta effect at the member level for columns failing in flexure and flexure-shear mode (Part-2)

The specific objectives include:

1. To develop a program to capture the Sectional behavior M-ϕ Curve including different constitutive behavior of concrete and steel.

2. To develop a program to capture Load v/s Deflection curve, the deflection due to following effects namely, P-delta, slip and H/D ratio effects.

Table 1.1 Influencing Factors at the Sectional and Member Level

	Sectional Level	Member Level
1.	Longitudinal Reinforcement Effect	P-delta Effect
2.	Transverse Reinforcement Effect	Slip Effect
3.	Axial Load Effect	H/D Ratio Effect

A program is coded in MATLAB for capturing sectional behavior which is the moment curvature (M-ϕ Curve). It is worth mentioning that only circular columns are considered in this analysis.

CHAPTER 2

LITERATURE REVIEW

2.1 INTRODUCTION

In seismic design, reinforced concrete columns are detailed to behave in a ductile manner in order to absorb and dissipate the energy transmitted from strong ground motions. Confining concrete is an effective method to provide adequate ductility for reinforced concrete columns. In the following section, background to the existing confining models are presented.

In general, parameters such as member sectional details, material properties, and loading conditions characterize the behaviour of RC columns under flexure, as shown in Figure2.1. Several experimental studies have examined the response of concrete elements under flexure and axial compression. A number of tests have been carried out to determine the cyclic behaviour of RC columns under flexure, with or without axial compression. The earliest tests on bridge columns under seismic loading were carried out in New Zealand and Japan. Several studies have provided valuable information on the behaviour of RC columns under cyclic uniaxial flexural loads (Kent, 1969; Mander et al., 1988; Ang et al., 1989; Wong et al., 1990 and 1993; Kawashima et al., 1994; Priestley et al., 1996; Kowalsky and Priestley 2000). The following review of these studies classifies the behaviour of RC columns according to the effect of aspect ratio, confinement, axial load, and other parameters.

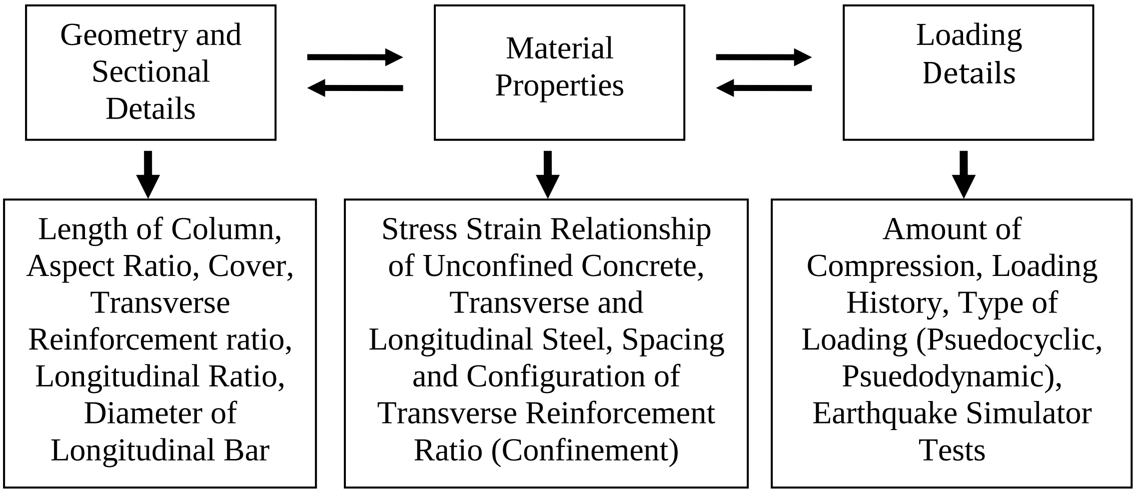


Figure 2.1. Factors Affecting the Behavior of RC Columns under Combined Flexure and Axial Compression

2.1.1 Effect of Aspect Ratio and Spiral Ratio

Several researchers have studied the effect of confinement by testing columns under monotonic and cyclic axial loads (Mander et al., 1988; Sheikh and Uzumeri, 1982; Calderone et al., 2001). Wong et al. (1990) tested columns with a smaller aspect ratio and found that the columns with a smaller transverse reinforcement ratio have a smaller curvature demand. Prakash et al. (2009) tested columns under different aspect ratio (6 and 3) and found that the shear capacity of the columns under bending and shear increased marginally with reduction in aspect ratio. They also found that there was no appreciable reduction in bending and torsional strength with reduction in aspect ratio under combined loadings. The damage zone was found to increase with increase in aspect ratio. (Figure.2.2). Similarly, several researchers have examined the effect of spiral reinforcement ratio on the behavior of circular columns (Potangaroa, 1979; Ang, 1981; Zhan, 1986). Increase in transverse reinforcement confines the concrete core more effectively and improves shear resistance. However, the effect of transverse reinforcement on shear dominated behavior and its influence on flexural ductility is relatively not very well understood.

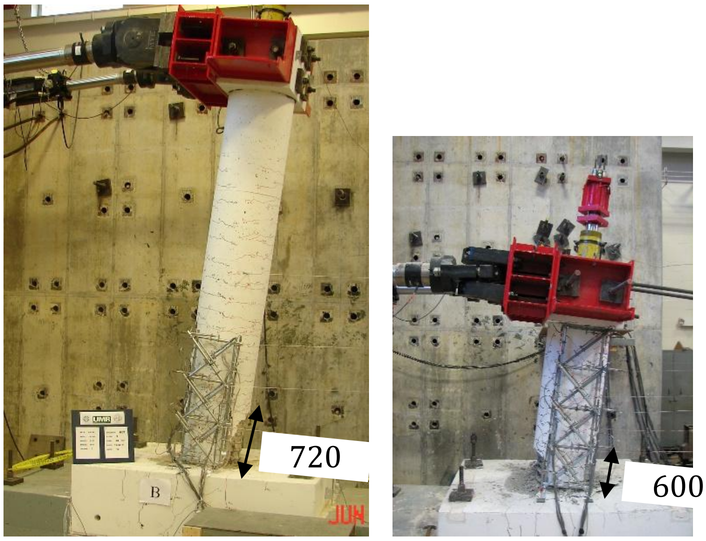


Figure.2.2 Effect of Aspect Ratio on Damage Distribution

2.1.2 Effect of Axial Loads

Previous studies have shown that an increase in axial compression reduces displacement capacity (Saatcioglu, 1991; and Sheikh and Khoury, 1994). The increase in axial compression increases the shear strength by enhancing the aggregate interlock and increasing shear transfer across the compression zone. On the other hand, when the axial load is tensile in nature, there is a decrease in shear strength, which most of the prevailing codes take into account. However, with increasing displacement ductility demand, shear strength decays significantly within the plastic end regions of columns and this effect has not yet been studied in depth. Axial load may vary during an earthquake due to vertical ground motions. Previous studies have reported failures due to significant vertical motions (Hachem et al., 2003); therefore, the effect of axial-flexure-shear interaction in the presence of very high vertical motions must be investigated. However, test data on RC columns under various vertical ground motions have been limited. Other important parameters that influence the behavior of RC members (in particular columns) are concrete cover thickness, longitudinal reinforcement ratio, bar diameter, and loading patterns. Dynamic and pseudo-dynamic test data on RC circular columns are also very limited for clarifying its dynamic behavior.

2.2 RICHART et.al (1928)

The authors found that strength and ductility of concrete significantly increased under triaxial compressive stresses. They reported that the lateral confining pressure reduced the tendency of internal cracking and volume increase prior to failure. Richart et al. (1929) further demonstrated that the enhancement of strength and ductility of concrete confined by fluid pressure was similar to that observed for concrete confined by transverse reinforcement.

Based on the work of Richart et al., ACI Committee 105 (1933) reported that the ultimate strength of concentrically loaded reinforced concrete columns confined by spirals could be expressed as follows:

$$\frac{P}{A_c} = C.f_c^{'}.(1-\rho) + f_y.\rho + k.f_{yh}.\rho_s \qquad (2.1)$$

Where

P = axial load capacity of column

A_c= cross sectional area of core concrete

C = constant, found to be 0.85

f'_c = compressive strength of concrete cylinders

ρ = ratio of cross sectional area of longitudinal reinforcement to core concrete area

f_y and f_{yh} = yield strengths of longitudinal reinforcement and spirals, respectively

k = constant, ranging between 1.5 to 2.5 with an average of 2.0

ρ_s = volumetric ratio of spirals to core concrete

With the assumption that spalling of cover concrete should not result in a loss of axial load capacity of a column to ensure a sufficient deformation capacity. Equation. 2.1 was further simplified to give the limitation for transverse reinforcement and Equation .2.2 was obtained.

$$\rho_s = 0.43\left(\frac{A_g}{A_{ch}} - 1\right)\frac{f_c^{'}}{f_{yh}} \quad (2.2)$$

Where

A_g= Gross area of column section

A_{ch} = area of core concrete measured out-to-out of transverse reinforcement

f'_c= compressive strength of concrete

f_{yh} = yield strength of transverse reinforcement.

2.3 CODE PROVISIONS FOR CONFINING REINFORCEMENT DESIGN

Various design codes have developed different recommendations for the quantity of confining reinforcement to be used in the potential plastic hinge regions in terms of sectional dimensions, strength of concrete and transverse reinforcement, and axial load level. In this section, various code provisions for confining reinforcement design are reviewed. The early research that led to the code development efforts is briefly discussed prior to the review of code provisions for confinement. Following this background research, the ACI 318-05 and NZS 3101:1995 provisions for confining reinforcement are discussed.

2.3.1 ACI 318

Equation.2.2 of the ACI Committee 105 (1933) has been used as the basis of the ACI 318 code for confining reinforcement requirements for seismic design since 1971. The current building code requirements (ACI 318-05) for the amount of spiral reinforcement in potential plastic hinge regions of columns are as follows:

For columns with $P_u > f'_c A_g/10$, where P_u is a factored axial compressive force, the volumetric ratio of spiral reinforcement (ρ_s) shall not be less than the values given by:

$$\rho_s = 0.45\left(\frac{A_g}{A_{ch}} - 1\right)\frac{f_c'}{f_{yh}} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (2.3)$$

The total cross sectional area of rectangular hoop reinforcement for confinement (A_{sh}) shall not be less than that given by the following two equations:

$$A_{sh} = 0.3 s b_c\left(\frac{A_g}{A_{ch}} - 1\right)\frac{f_c'}{f_{yt}} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (2.4)$$

where

A_g = gross area of column section

A_{ch} = area of core concrete measured out-to-out of transverse reinforcement

f'_c= compressive strength of concrete

f_{yt}= yield strength of transverse reinforcement

s = spacing of transverse reinforcement

b_c = cross sectional dimension of column core, measured center-to-center of transverse reinforcement when $P_u \leq f'_c A_g/10$, columns are designed as flexural members.

The length of the potential plastic hinge regions is specified as the greatest of the overall depth (*h*) of a column at the joint face, where *h* is the larger sectional dimension for a rectangular column or the diameter of a circular column, one-sixth of the clear height of a column, 457 mm. The spacing of transverse reinforcement is required to be less than *h*/4, $6 \times d_b$, and s_o, where *h* is the minimum member dimension, d_b is the diameter of longitudinal reinforcement, and s_o is defined as $4 + (14 - h_x) / 3$. Here, h_x is defined as the maximum value of spacing of crossties or legs of overlapping hoops and it has be kept less than 356 mm. The value of s_o to be less than 153 mm. and need not be taken less than 102 mm.

2.3.2 NZS 3101:

Park and Sampson (1972) and Park and Leslie (1977) conducted analytical research on the moment-curvature response of concrete columns and concluded that the curvature ductility capacities were significantly influenced by axial loads. This conclusion was experimentally examined by Ang et al. (1981) and Park et al. (1982). Based on these investigations, the New Zealand design code adopted modified versions of the ACI code requirements for confining reinforcement to account for the effect of axial loads in 1982 (NZS 3101:1982). For circular columns, the NZS 3101:1995 code requires that the volumetric ratio of spiral reinforcement (ρ_s) shall not be less than the values given by Equations. 2.5 and 2.6.

$$\rho_s = \frac{(1.3 - m\rho_t)}{2.4} \frac{A_g}{A_{ch}} \frac{P}{\phi A_g f_c'} \frac{f_c'}{f_{yt}} - 0.0084 \qquad (2.5)$$

For columns with rectangular hoops, NZS 3101:1995 requires that the total cross sectional area of rectangular hoop reinforcement for confinement shall not be less than that given by the following equation:

$$\rho_s = \frac{(1.3 - m\rho_t)s_h h''}{2.4} \frac{A_g}{A_{ch}} \frac{P}{\phi A_g f_c'} \frac{f_c'}{f_{yt}} - 0.006 s_h h'' \quad \text{...(2.6)}$$

Where

$\frac{A_g}{A_{ch}} \geq 1.2$ and $m\rho_t \leq 0.4$

A_g = Gross area of column section

A_{ch}= area of core concrete measured out-to-out of transverse reinforcement

ρ_t= longitudinal reinforcement ratio

m = f_y / (0.85f'_c)

f_c= compressive strength of concrete

f_{yt}= yield strength of transverse reinforcement

d'' = diameter of core concrete of circular column measured out-to-out of spiral

d_b= diameter of longitudinal bar

s_h= spacing of transverse reinforcement (= s)

h'' = concrete core dimension measured outer-to-outer peripheral hoop

P = design axial load

The spacing of spirals or hoops along the member shall not exceed the smaller of 1/4 of the least lateral dimension of the cross section or 6 times the diameter of the longitudinal bar to be restrained. Potential plastic hinge regions in columns are considered to be the end regions adjacent to moment resisting connections over a length from the face of the connection as follows:

(a) Where $P \leq 0.25\varphi f'_c A_g$ the greater of the longer member cross section dimension in the case of a rectangular cross section or the diameter in the case of a circular cross section, or where the moment exceeds 0.8 of the maximum moment, taking into account dynamic magnification and over strength actions, at that end of the member.

(b) Where $0.25\varphi f'_c A_g < P \leq 0.5\varphi f'_c A_g$ the greater of 2.0 times the longer member cross section dimension in the case of a rectangular cross section or 2.0 times the diameter in the case of a circular cross section, or where the moment exceeds 0.7 of the maximum moment, taking into account dynamic magnification and over strength actions, at that end of the member.

(c) Where $0.5\varphi f'_c A_g < P \leq 0.7\varphi f'_c A_g$ the greater of 3.0 times larger member cross section dimension in the case of a rectangular cross section or 3.0 times the diameter in the case of a circular cross section, or where the moment exceeds 0.6 of the maximum moment, taking into account dynamic magnification and over strength actions, at that end of the member. It is interesting to note that the effect of axial load is considered in determining both the amount of confining reinforcement and the length of the potential plastic hinge region.

CHAPTER 3

CONFINEMENT MODELS

3.1 INTRODUCTION

Existing well known models such as (i) Sheikh and Uzumeri; (ii) Mander and Priestly and (iii) Saatcioglu and Razvi are evaluated in this study. All the models considered for evaluation includes the effects of strain gradient, axial load ratio and member level parameters. A brief background on the development of these models are explained in the following sections.

3.2 SHEIKH AND UZUMERI'S MODEL

Sheikh and Uzumeri (1992 proposed an improved confinement model based on their test results on circular columns. The main variables included were (i) the distribution of longitudinal and lateral steel, (ii) amount of lateral steel, (iii) tie spacing, and (iv) axial load level. The authors proposed a new confinement model by modifying the model developed for concentric compression by including the effects of strain gradient and the axial load level. As a result of strain gradient, the concrete is able to sustain additional deformation at and beyond the peak stress. The effect of increased axial load is incorporated with reduced concrete strength. The model proposed by Sheikh and Uzumeri (1992) is shown in Figure 3.1.

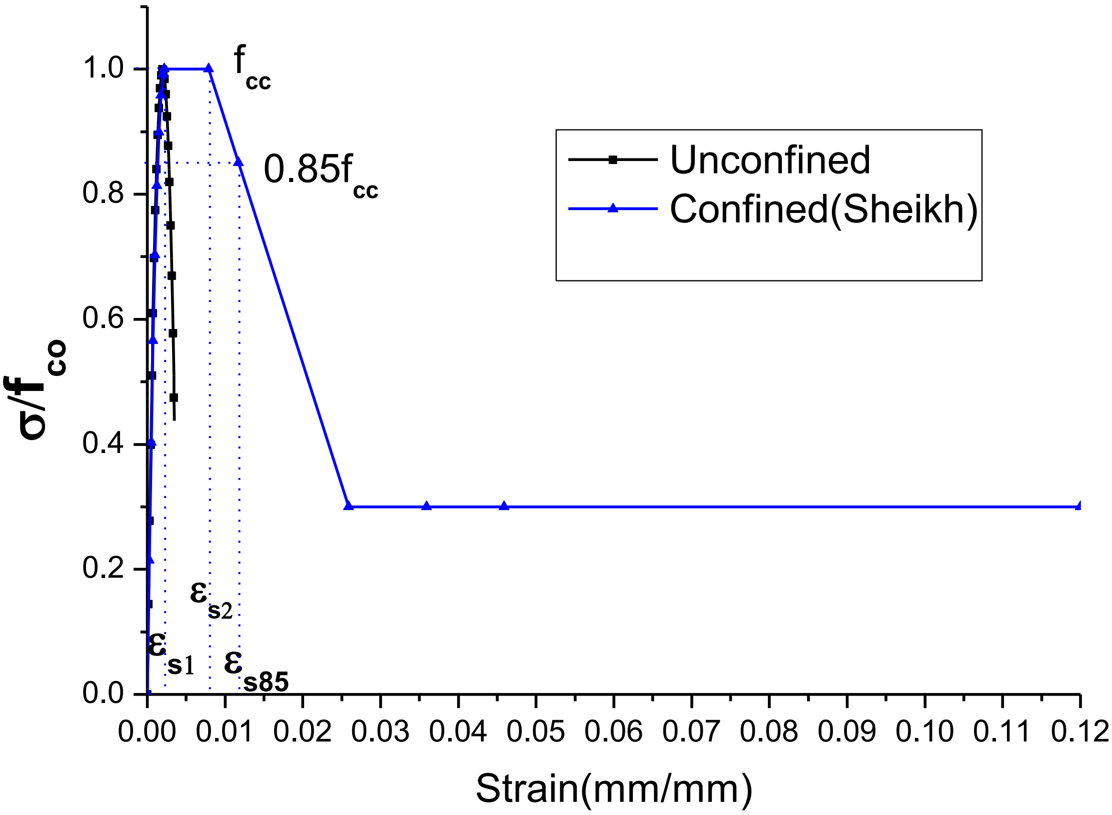


Figure 3.1. Normalized Confined and Unconfined stress strain curves of Sheikh and Uzumeri's Model

Formulations:

$$f_{cc} = f_{co}\eta K_s \tag{3.1}$$

$$\eta = 1 - 0.575\frac{P - P_o}{f_{co}A_c} \tag{3.2}$$

$$K_s = 1 + \frac{B^2}{10.58P_{occ}}[(1 - \frac{nC^2}{5.5B^2})(1 - \frac{s}{2B})^2]\sqrt{f_{yh}\rho_s} \tag{3.3}$$

$$\varepsilon_{s1} = 0.0022K_s \tag{3.4}$$

$$P_{occ} = (A_c - A_l)\eta f_{co} \tag{3.5}$$

$$\frac{\varepsilon_{s2}}{\varepsilon_o} = 1 + \{\frac{0.81}{C}[1 - 5(\frac{s}{B})^2] + 0.25\sqrt{\frac{B}{c}}\}\frac{\rho_s f_{yh}}{\sqrt{f_{co}}} \tag{3.6}$$

$$\varepsilon_{s85} = 0.225\rho_s\sqrt{\frac{B}{s}} + \varepsilon_{s2} \tag{3.7}$$

where σ = confined stress; Po= Axial load capacity with zero eccentricity; A_c= area of core measured from centre to centre of the perimeter tie; A_s = area of longitudinal steel; *B* = core size measured from centre to centre or perimeter tie in in; C = distance between laterally supported longitudinal bars of *4B/n*; f_c= cylinder strength of concrete; f_{cp} = strength of unconfined concrete in the column =k_p f_c; f_s = stress in the lateral steel; k_p = ratio of unconfined concrete strength in the column *to* f_c; *n* = number of arcs containing concrete that is not effectively confined, also equal to the number of laterally supported longitudinal bars; P_{occ} = unconfined strength of concrete core; *s* = tie spacing.; ρ_s= ratio of the volume of tie steel to the volume of core; ε_0 = strain corresponding to the maximum stress in unconfined concrete; ε_{s1}=strain corresponding to the first peak strength of confined concrete; ε_{s2}= strain corresponding to the second peak strength of confined concrete; ε_{s85}= strain corresponding to 85% of peak strength of confined concrete;

3.3 MANDER AND PREISTLY'S MODEL

The authors developed a stress-strain model for concrete subjected to uniaxial compressive loading and confined by transverse reinforcement. The developed model was able to accommodate concrete section with any general type of confining steel: either spiral or circular hoops; or rectangular hoops with or without supplementary cross ties. These cross ties can have either equal or unequal confining stresses along each of the transverse axes. The model also allowed to consider the effect of cyclic loading and included the strain rate effects. The influence of various types of confinement was taken into account by defining an effective lateral confining stress, which is dependent on the configuration of the transverse and longitudinal reinforcement. An energy balance approach was used to predict the longitudinal compressive strain in the concrete corresponding to first fracture of the transverse reinforcement by equating the strain energy capacity of the transverse reinforcement to the strain energy stored in the concrete as a result of the confinement. This model was derived based on pure axial compression tests and has not been validated adequately under the combined loadings of axial compression, flexure and shear as shown in the Figure 3.2. Previous work (Lehman 1998, Calderone et al. 1996) have modified the peak confined strength in the Mander's model for the flexural analysis and predicted the behavior of columns under flexure reasonably well.

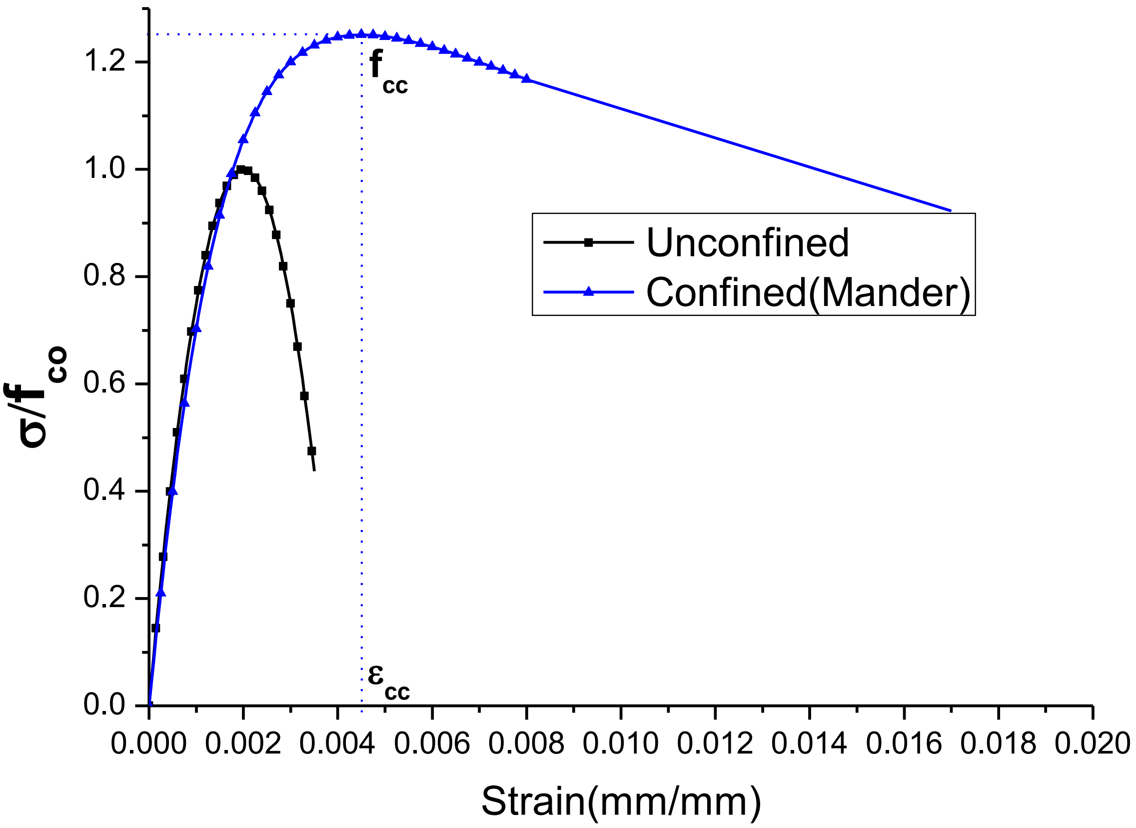


Figure 3.2. Normalized Confined and Unconfined Stress Strain curves by Mander and Preistly's Model

Formulations:

$$f_c = \frac{f_{cc} x r}{r - 1 + x^r} \tag{3.8}$$

$$x = \frac{\varepsilon_c}{\varepsilon_{cc}} \tag{3.9}$$

$$r = \frac{E_c}{E_c - E_{\sec}} \tag{3.10}$$

$$\varepsilon_{cc} = \varepsilon_{co}[1 + 5(\frac{f_{cc}}{f_{co}} - 1)] \tag{3.11}$$

$$\varepsilon_{cu} = 0.004 + \frac{1.4 \rho_s f_{yh} \varepsilon_{cu}}{f_{co}} \tag{3.12}$$

$$f_{cc} = f_{co}(-1.254 + 2.254\sqrt{1 + \frac{7.94 f_{le}}{f_{co}}} - \frac{2 f_{le}}{f_{co}}) \tag{3.13}$$

$$f_{le} = f_l k_e \tag{3.14}$$

$$k_e = \frac{A_e}{A_{cc}} \tag{3.15}$$

$$A_{cc} = A_c(1 - \rho_{cc}) \tag{3.16}$$

where σ = confined stress; f_l =Lateral Pressure= 0.5 f_{yh} ρ_s ; A_e= Area of effectively confined core; A_c=Core concrete; ρ_s =Transverse reinforcement volumetric ratio; ρ_{cc} =Longitudinal reinforcement ratio; f_{cc}= peak confined concrete strength; ε_{cu} = ultimate strain of core concrete.

3.4 SAATCIOGLU AND RAZVI'S MODEL

The model consists of a parabolic ascending branch, followed by a linear descending segment. It was derived based on the calculation of lateral confinement pressure generated by circular and rectilinear reinforcement, and the resulting improvements in strength and ductility of confined concrete. A large volume of test data, including poorly confined and well-confined concrete was evaluated to establish the parameters of this confinement model. Confined concrete strength and corresponding strain are expressed in terms of equivalent uniform confinement pressure provided by the reinforcement cage. The equivalent uniform pressure is obtained from average lateral pressure computed from sectional and material properties. Confinement by a combination of different types of lateral reinforcement is evaluated through superposition of individual confinement effects. The descending branch is constructed by defining the strain corresponding to 85% of the peak stress. This strain level is expressed in terms of confinement parameters. A constant residual strength is assumed beyond the descending branch, at 20% strength level as shown in the Figure 3.3.

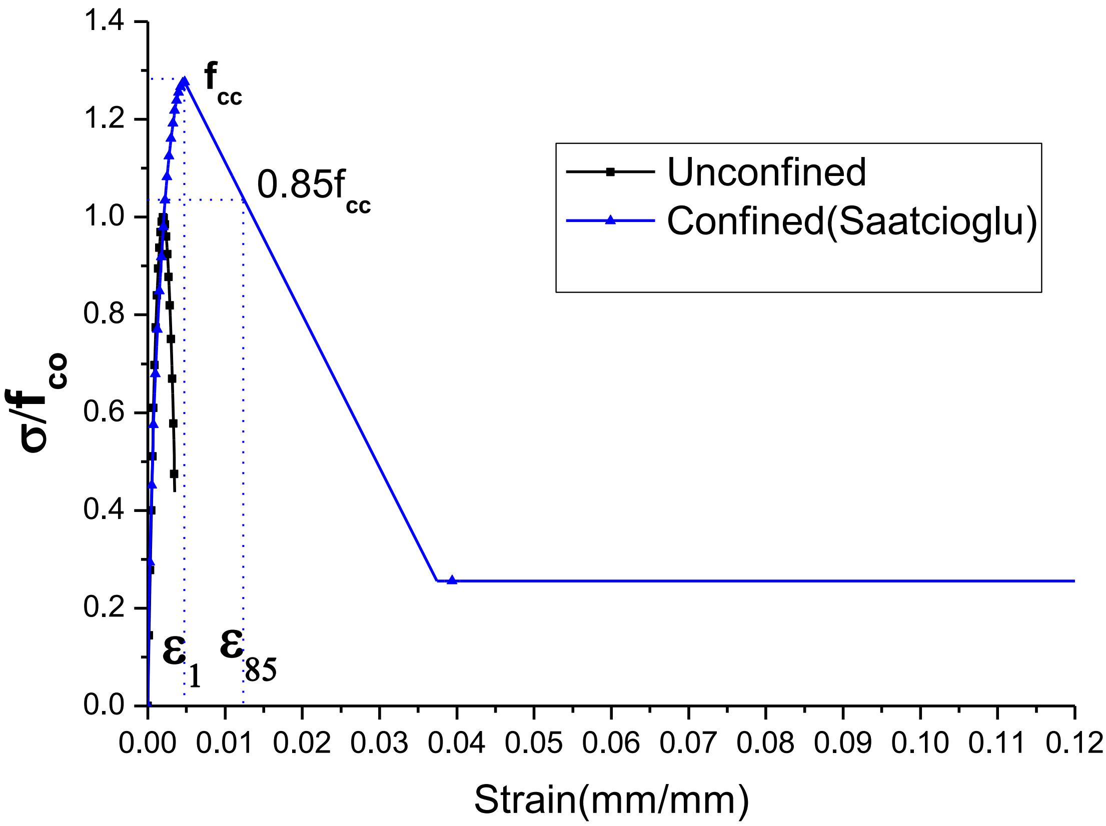


Figure 3.3. Normalized Confined and Unconfined Stress Strain Curves of Saatcioglu and Razvi's Model

Formulations:

$$f_c = f_{cc}[2(\frac{\varepsilon_c}{\varepsilon_1}) - (\frac{\varepsilon_c}{\varepsilon_1})^2]^{\frac{1}{1+2K}} \leq f_{cc} \tag{3.17}$$

$$f_{cc} = f_{co} + k_1 f_{le} \tag{3.18}$$

$$f_{le} = k_2 f_l \tag{3.19}$$

$$k_1 = 6.7(f_{le})^{-0.17} \tag{3.20}$$

$$k_2 = 0.26\sqrt{\frac{1}{f_l}\frac{b_c}{s_l}\frac{b_c}{s}} \leq 1 \tag{3.21}$$

$$f_l = \frac{2 f_{yt} A_s}{s b_c} \tag{3.22}$$

$$K = \frac{k_1 f_{le}}{f_{co}} \tag{3.23}$$

$$\varepsilon_1 = \varepsilon_{01}(1+5K) \tag{3.24}$$

$$\varepsilon_{85} = \varepsilon_{085} + 260\rho\varepsilon_1 \tag{3.25}$$

$$\rho = \frac{4A_s}{s b_c} \tag{3.26}$$

where σ = confined stress; s_l =lateral reinforcement spacing; s =spacing of hoop reinforcement; b_c = core size of concrete; A_s = Area of transverse reinforcement; ε_1= strain corresponding to peak strength of confined concrete; ε_{01}= strain corresponding to peak strength of unconfined concrete; ε_{85}= strain corresponding to 85% of peak strength of confined concrete; ε_{085}= strain corresponding to 85% of peak strength of unconfined concrete;

3.5 PROPOSED IMPROVED MODEL

In view of the limitations of the existing models, a new model is proposed in this study. The proposed model consists of a parabolic ascending branch, followed by constant stress level and then followed by a linear descending segment (Figure 3.4). Formulation of the model is based on the calculation of lateral confinement pressure generated by circular hoops, and the resulting improvements in strength and ductility of confined concrete. The parameters of the model are calibrated through large volume of test data available in PEER database. Confined concrete strength and corresponding strain are expressed in terms of equivalent uniform confinement pressure provided by the reinforcement cage (Eqs. 3.18 and 3.24). The equivalent uniform pressure is obtained from average lateral pressure computed from sectional and material properties. Hoop / spiral reinforcement in circular sections leads to additional confined strength of the sections. In addition, progressive yield of longitudinal reinforcement in circular sections gives rise to additional ductility at the peak stress. These phenomena were taken in to account in the proposed model. The original Saatcioglu's model under-estimates the ultimate capacity significantly as observed from load deflection curve in Figure 4.4. Therefore, to improve the prediction of the Saatcioglu model, strength improvement factor was incorporated. Improved ductility and extra strength parameters were also added to the Saatcioglu model. The ductility level is adopted from the Sheikh and Uzumeri's model at the peak level of stress strain curve. Because of the composite action at the peak level, material readjustment takes place. So a minimum ductility should be maintained at this level. The descending branch is constructed by defining the strain corresponding to 85% of the peak stress. This strain level is expressed in terms of confinement parameters (Equation. 3.25). A constant residual strength is assumed beyond the descending branch, at 20% strength level.

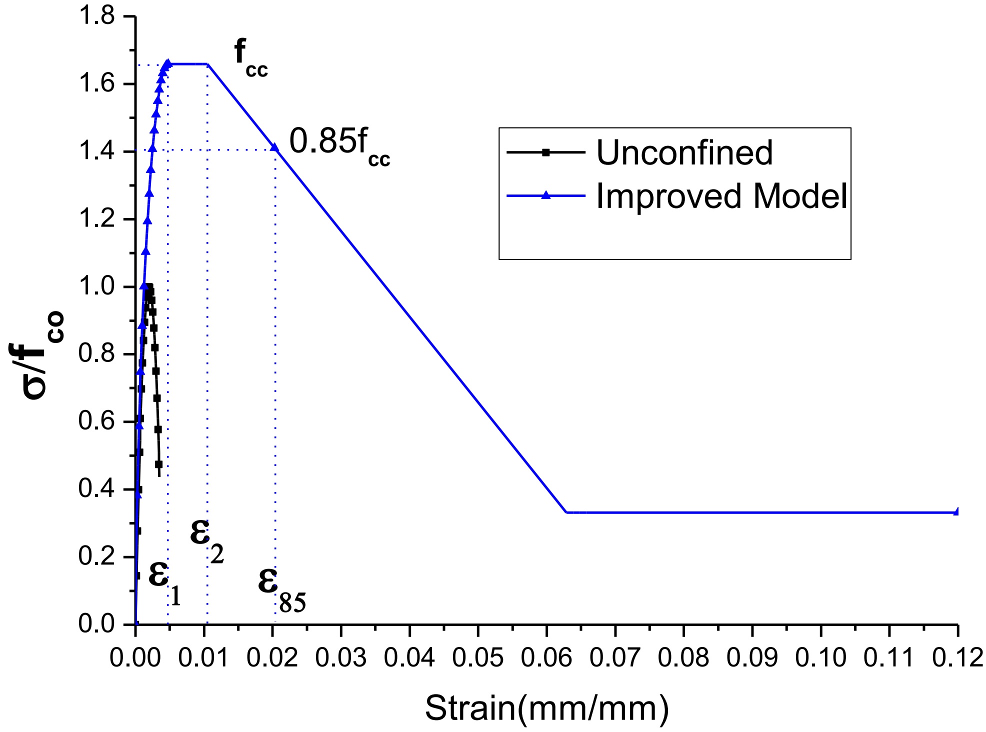


Figure 3.4. Proposed Improved Normalized Confined Stress Strain Model

Formulations:

$$f_c = 1.3 f_{cc}[2(\frac{\varepsilon_c}{\varepsilon_1}) - (\frac{\varepsilon_c}{\varepsilon_1})^2]^{\frac{1}{1+2K}} \leq f_{cc} \tag{3.27}$$

$$\varepsilon_2 = \varepsilon_1 + (\varepsilon_{s2} - \varepsilon_{s1}) \tag{3.28}$$

where σ = confined stress; ε_2 = strain at last peak point; $\varepsilon_{s2}, \varepsilon_{s1}$ are from Sheikh and Uzumeri model and the trends observed in other portions are similar to that of Saatcioglu model.

CHAPTER 4

EVALUATION OF CONFINEMENT MODELS: AT THE SECTIONAL LEVEL (Part-1)

4.1 OVERVIEW

Moment curvature at the sectional level and the load displacement curves at the member level are predicted using the new proposed model and validated with test results of eight full scale columns. Two circular columns one with an aspect ratio of ‘6’ and the other with ‘3’ tested under flexure by the second author are used in this evaluation. Out of the remaining six columns, four of them were tested by Lehman at the University of California Berkeley and the other two were tested at NIST, USA. Details of the columns used for evaluation are given in Table 4.1.

Table 4.1 Details of Specimen used for Evaluation

Specimen Name Test-H/D-ρ_l-ρ_s-A_l	ρ_l (%)	ρ_t (%)	Height (m)	Dia (mm)	Concrete Cylinder Strength (MPa)	H/D	Axial (kN)	Yield Strength of Long. Bar (MPa)	Yield Strength of Transv. Bar (MPa)
Missouri-H/D(6)-2.10%-0.73%-6.16%	2.1	0.73	3.67	609.6	30	6	592	450	450
Missouri-H/D(3)-2.10%-1.32%-6.16%	2.1	1.32	1.83	609.6	30	3	592	450	450
Lehman-H/D(4)-1.5%-0.72%-7.2%	1.5	0.72	2.4	609.6	31	4	654	462	607
Lehman-H/D(10)-1.5%-0.72%-7.2%	1.5	0.72	6.1	609.6	31	10	654	462	607
Lehman-H/D(4)-0.75%-0.72%-7.2%	0.75	0.72	2.4	609.6	31	4	654	462	607
Lehman-H/D(4)-2.98%-0.72%-7.2%	2.98	0.72	2.4	609.6	31	4	654	462	607
NIST-H/D(6)- 2.0%-1.49%-6.9%	2.0	1.49	9.1	1520	35.8	6	4450	475	493
NIST-H/D(6)- 2.0%-1.49%-9.6%	2.0	1.49	1.5	250	25.4	6	120	446	476

4.2 FAILURE MODES OF COLUMNS USED FOR EVALUATION

4.2.1 Behavior of Columns Tested in Missouri

Test data for the specimens tested in University of Missouri are obtained from Prakash (2009). Missouri columns were tested in cantilever position with a length of 3.1m and a diameter of 610 mm. These columns had same axial load ratio (P_u/ f'_c A_g= 6.16%) and longitudinal reinforcement ratio of 2.10%. Test parameters included low and high transverse reinforcement ratio (0.73%, 1.32%) and aspect ratio (H/D=6, 3). Details of the test setup and complete behavior of the columns can be found elsewhere (Prakash et al. 2009, Prakash et al. 2012).

<u>Missouri-H/D(6)-2.10%-0.73%-6.16%:</u> This column had a lower transverse reinforcement ratio of 0.73 % but tested with higher V/M or H/D ratio of six indicating flexure dominated behavior. Failure of the specimen began with the formation of a flexural plastic-hinge at the base of the column, followed by core degradation, and finally by the buckling of longitudinal bars on the compression side. The progression of damage is shown in Figure 4.1. The flexural resistance was maintained at a nearly constant bending strength of 850 kN-m.

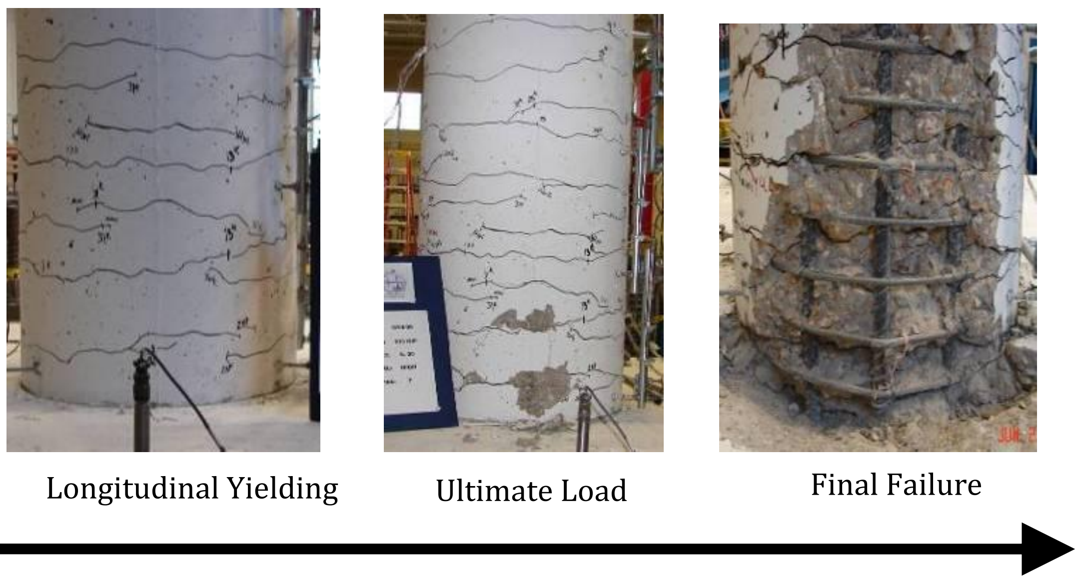


Figure 4.1. Damage to Column Missouri-H/D(6)-2.10%-0.73%-6.16% under Flexure

Missouri-H/D (3)-2.10%-1.32%-6.16%: This column had a higher transverse reinforcement ratio of 1.32% but tested with lower V/M or H/D ratio of three. Failure of the column began with the formation of a flexural plastic-hinge at the base of the column, followed by core degradation, and finally by the buckling of longitudinal bars on the compression side at a drift of about 5.1%. Though the column was tested at a lower H/D ratio of 3, the failure was dominated mainly by flexure due to the relatively low longitudinal ratio of the column and increased confinement from spiral reinforcement due to a higher spiral ratio of 1.32%. Thus, the increase in spiral ratio may have helped to change the failure mode from brittle shear to ductile flexural failure as a result of the increased level of shear resulting from a reduction in the shear span ratio. The progress of the failure is shown Figure 4.2.

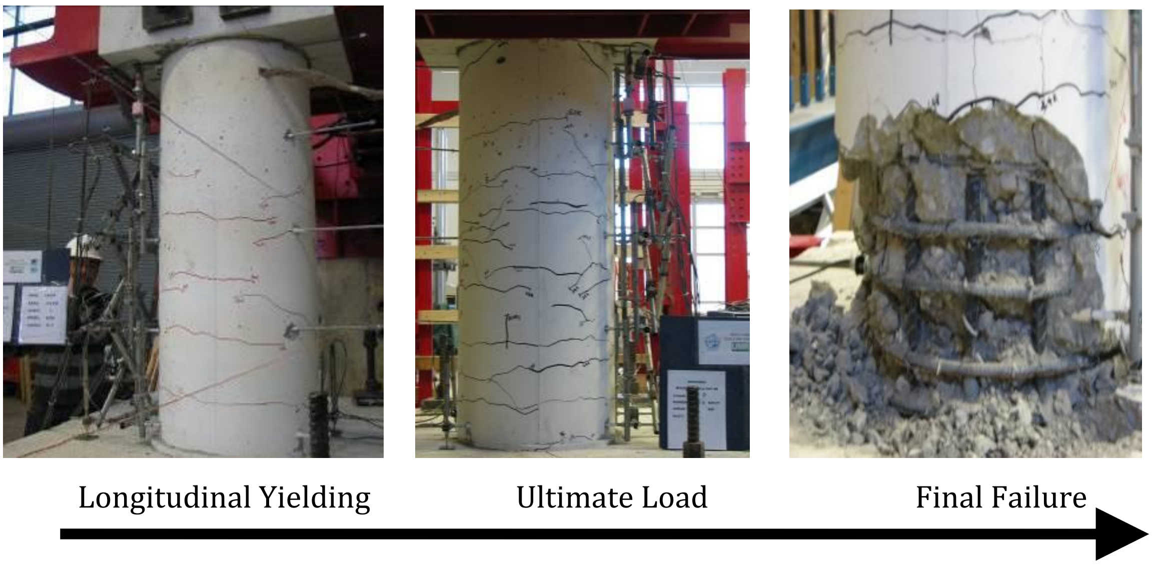


Figure 4.2. Damage of Column Missouri-H/D(3)-2.10%-1.32%-6.16%: under Flexure

4.2.2 Behavior of Columns Tested by Lehman

Lehman tested five columns in two series with column longitudinal reinforcement ratio and aspect ratio as the parameters. The first series studied the influence of longitudinal reinforcement ratio. All columns had aspect ratio of four with three different longitudinal reinforcement ratios of 0.75%, 1.5% and 3.0%. Second series investigated the influence of different aspect ratios such as 4,8, and 10 with a constant longitudinal reinforcement ratios of 1.5%. All the columns had a diameter of 610 mm with a spiral reinforcement ratio of 0.7% at a constant axial load ratio of 7.2% (P_u/ f'_c A_g). All the columns tested by Lehman (1998) failed in a flexure dominant mode. More details on the test results can be found elsewhere (Lehman et al. 1998, Lehman and Moehle 2000).

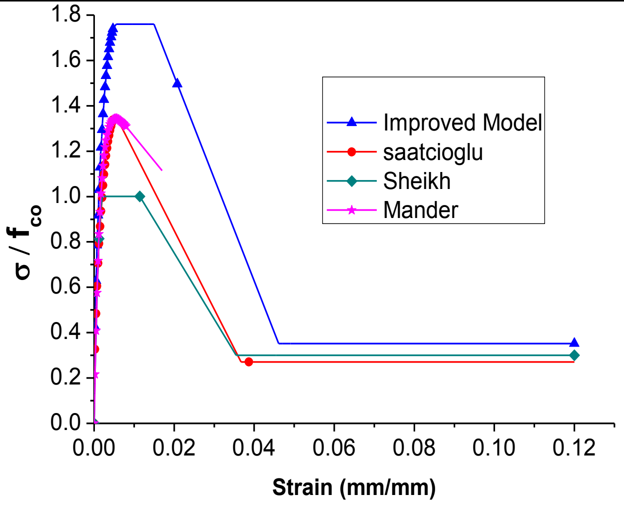
4.2.3 Behavior of Columns Tested at NIST

NIST tested three columns in two series with axial load ratio and aspect ratio as the parameters. The first series studied the influence of axial load ratio. All columns had aspect ratio of six with two different axial load ratios of 6.9%and 9.6%. Second series investigated the influence of different aspect ratios such as 3 and 6 with a constant axial load ratios of 7%. All the columns had a spiral reinforcement ratio of 1.5% at a constant longitudinal reinforcement ratio of 2% . All the columns considered in this study failed in flexure dominant failure mode. More details on the test results can be found elsewhere (Taylor and Stone 1993).

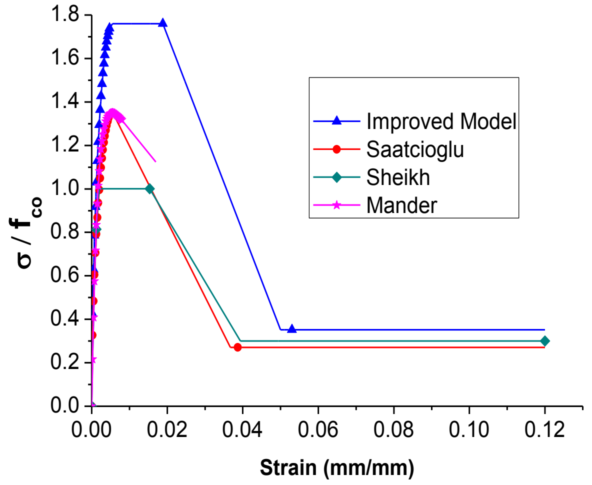
4.3 CONFINED STRESS STRAIN BEHAVIOUR BY DIFFERENT MODELS

Typical confined concrete stress strain models were shown for the particular sectional properties (Figure 4.3). As shown in Figure 4.3(a) and 4.3(b), longitudinal reinforcement is found to influence the confined stress-strain behaviour. The model Sheikh and Uzumeri's model having very low strength improvement factor for all the columns

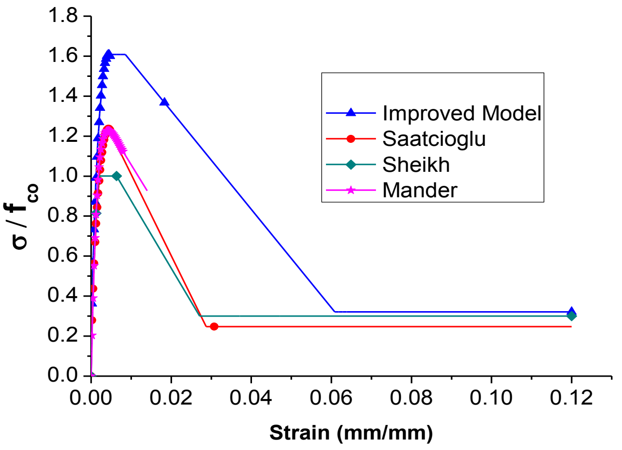
(Figure 4.3). A comparison of different models for the column tested by the author is also shown in Figure 4.3(d). For different values of longitudinal and axial load ratios, the confined stress strain curves were developed which is shown in column (c). In all the cases, Mander and Saatcioglu models predict the same level of confined strength. However, the ductility levels of these models are very different. The proposed improved model is having higher strength improvement.



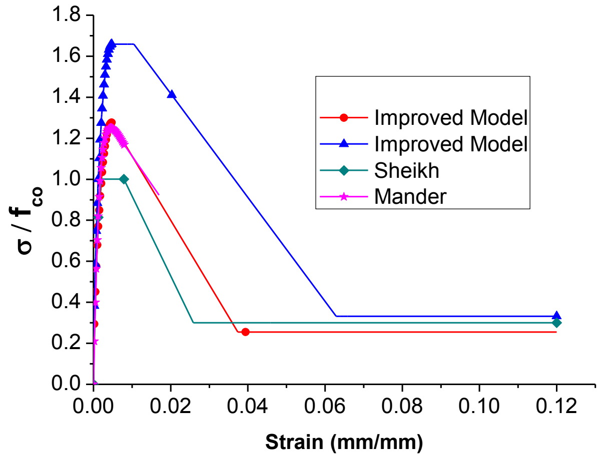
Column(a): Lehman-H/D(4)- 1.5%-0.72%-7.2%



Column(c): Lehman-H/D(4)- 2.98%-0.72%-7.2%



Column (c): NIST-H/D(6)- 2.0%-1.49%-6.9%



Column (d): Missouri-H/D(6)-2.10%-0.73%-6.16%

Figure 4.3. Comparison of Confined Stress Strain Behaviour for Different Models

4.4 SECTIONAL ANALYSIS - MOMENT CURVATURE BEHAVIOUR

The sectional behaviour of reinforced concrete columns can be reliably estimated once the proper constitutive relationships for concrete and steel can be established. A MATLAB program was developed for the sectional analysis and the algorithm is shown in Figure 4.4. The predicted moment curvature behaviour for different set of column data is shown in the Figure 4.5. Experimental data is available only for the columns (a), (b) and (c). The graphs for NIST (column (d)) and Missouri (column (e)) columns are also presented. It is shown that improved model is able to predict very closely the experimental behaviour. The predicted behaviour is very close upto the yield point and its slightly over predicting the ultimate moment.

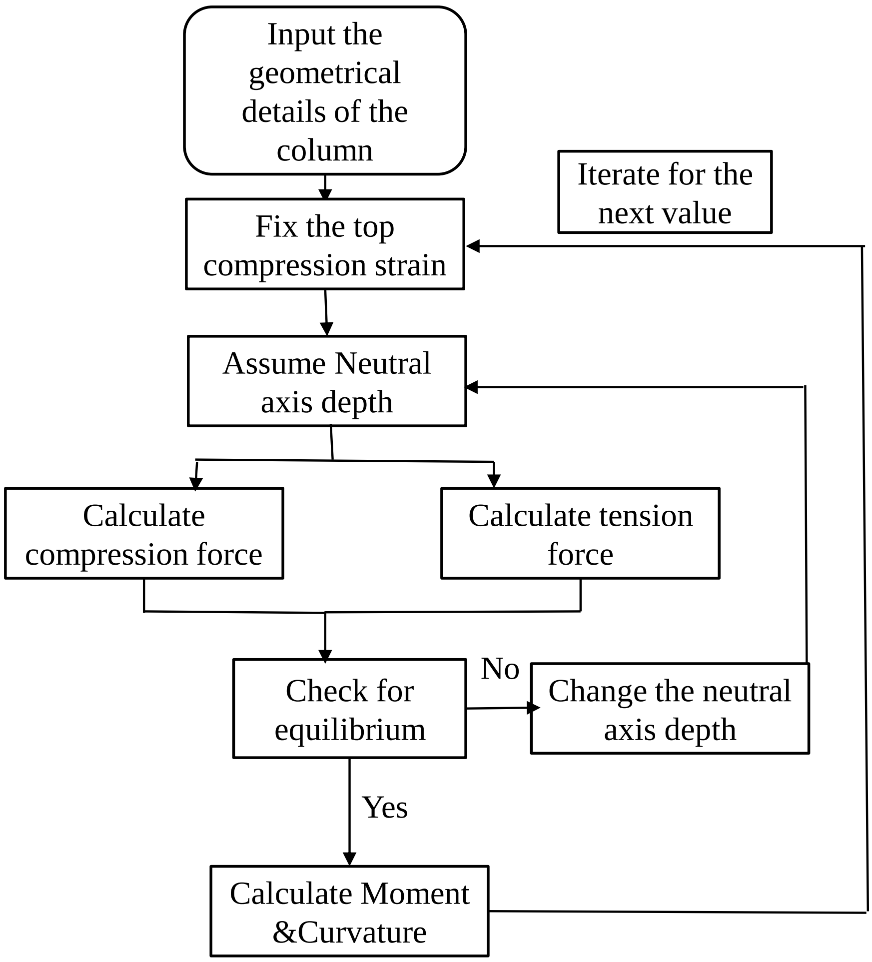
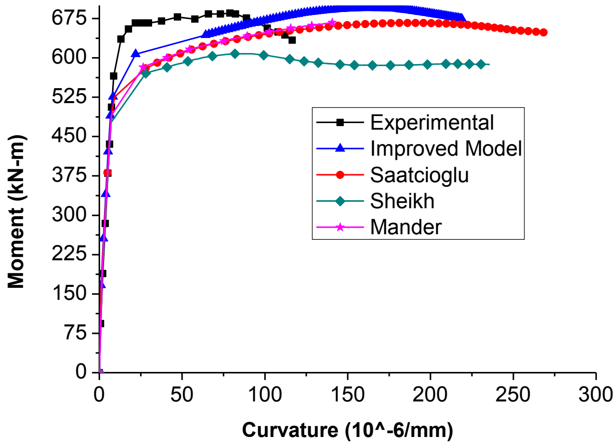
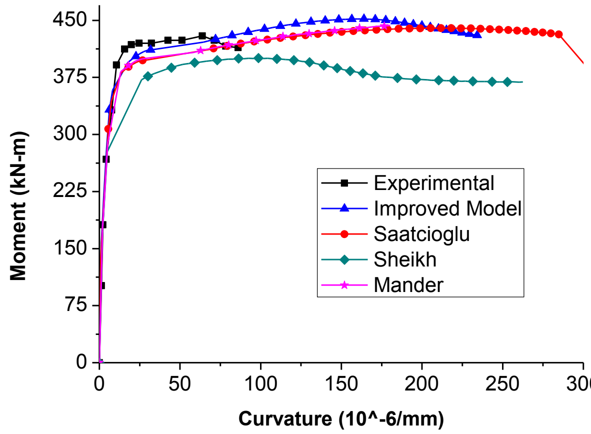


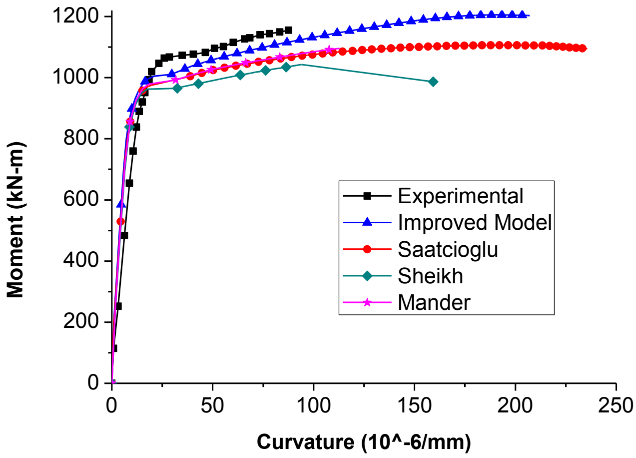
Figure 4.4. Algorithm for Moment Curvature Analysis for Circular Secitons



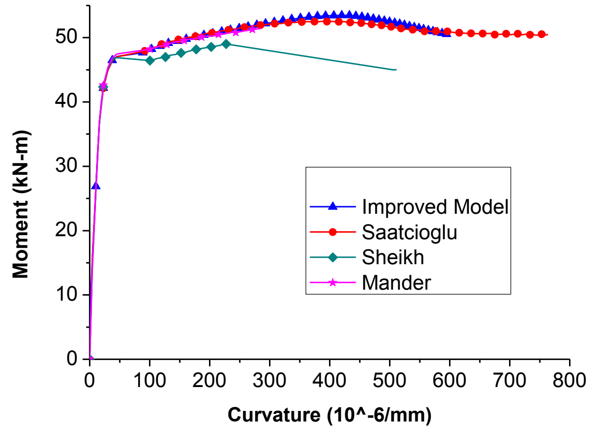
Column(a): Lehman-H/D(4)- 1.5%-0.72%-7.2%



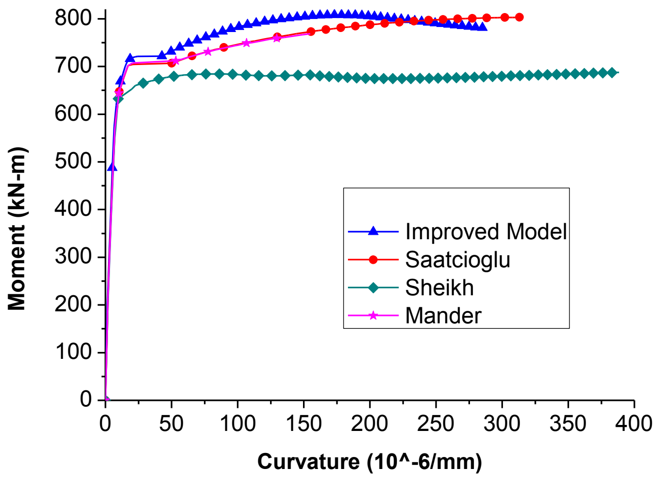
Column(b): Lehman-H/D(4)- 0.75%-0.72%-7.2%



Column(c): Lehman-H/D(4)- 2.98%-0.72%-7.2%



Column(d): NIST-H/D(6)- 2.0%-1.49%-9.6%



Column(e): Missouri-H/D(3)-2.10%-1.32%-6.16%

Figure 4.5. Comparison of Moment Curvature Behaviour using Different Models

CHAPTER 5

EVALUATION OF CONFINEMENT MODELS: AT THE MEMBER LEVEL (Part-2)

5.1 INTRODUCTION

After getting the Sectional behaviour of columns which is M-ϕ curve, using the second moment area theorem is used to get flexural displacement. This does not include additional moment caused by axial load which is P- delta effect and slip of the longitudinal bars on the tension side. The behaviour will also change with respect to H/D ratio by transforming from shear to flexure. At the peak loads, very high and nonlinear curvatures occurs due to yielding of longitudinal bars at the base. This is called as plastic-hinge location.

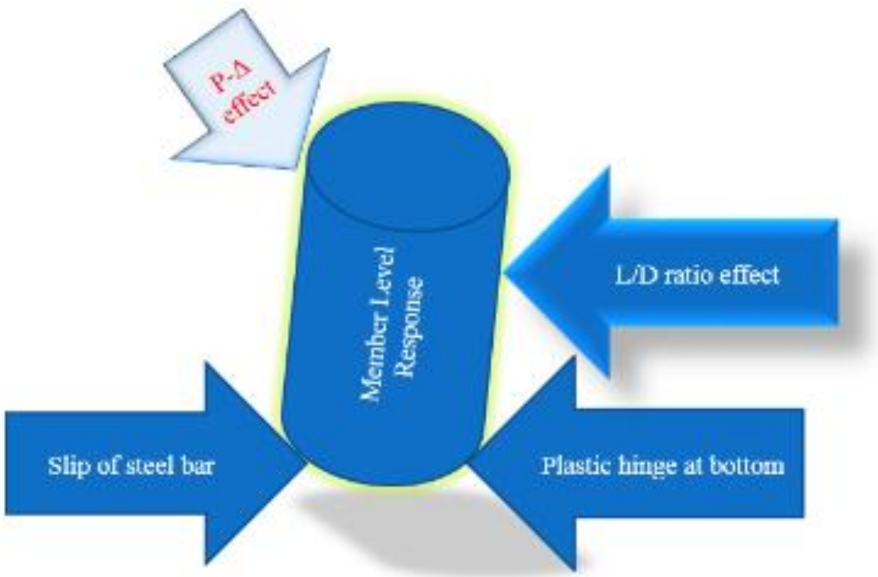


Figure 5.1. Summary of Member Level Behavior

The main contributing factors for the member level behavior considered are (1) P-delta effect (2) Slip effect (3) H/D ratio effect and (4) Plastic Hinge at bottom a shown in the Figure 5.1.

5.2 P-DELTA EFFECT

When columns supporting substantial axial loads experience lateral displacements, the gravity-induced axial loads produce pronounced secondary moments. The distribution of secondary moments is related to the deflected shape of a column along the height of a column as shown in the Figure 5.2. Therefore, combining the moment diagram by lateral loads and the *P*-Δ effect, curvatures along the column height can be obtained and the associated tip deflection can be estimated.

Since the secondary moment distribution along the column depends on the deflected shape of a column, an iterative procedure is required to get the Load Deflection diagram.

$M=VL+P\Delta$………………………………………………………………………. (5.1)

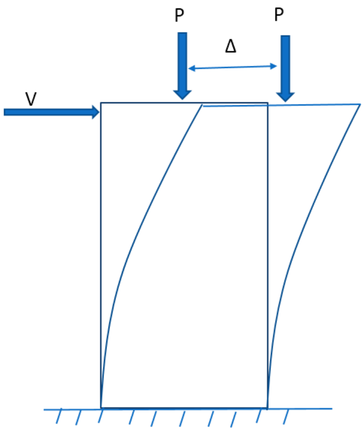


Figure 5.2. P-Δ effect

5.3 SLIP EFFECT

The formation of flexural cracks at the interface of a column and a typical beam-column joint (or foundation) strain the longitudinal bars crossing the crack. Widening of such cracks produces inelastic strains in the bar. This results in the penetration of yielding into the anchorage zone, causing extension of the bar. Hence, reinforced concrete columns experience additional rigid body rotations at their base due to bar slip.

In this study, displacement due to bar slip is computed using the analytical model proposed by Alsiwat and Saatcioglu (1992). This model incorporates yield penetration and associated inelasticity in an anchored bar, as well as the possibility of slip as shown in the Figure 5.3. Once the bar slip at the end of a column is computed, the end rotation and lateral displacement of a cantilever column due to bar slip can be determined as follows:

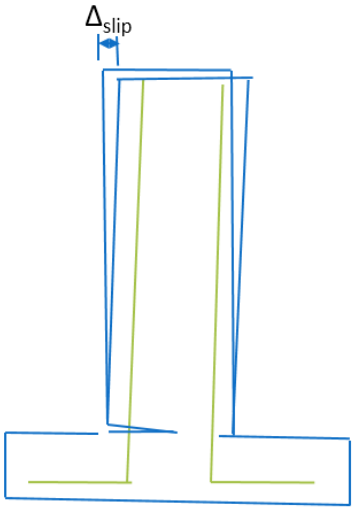


Figure 5.3. Slip Effect

$$\Delta_{slip} = L\theta_{slip} \quad (5.2)$$

$$\theta_{slip} = (0.5\phi f_y d_b)/(4*12*\sqrt{f_c}) \quad (5.3)$$

5.4 H/D RATIO EFFECT

The behavior will change according to H/D ratios. For smaller H/D ratios, shear is dominant (Figure 5.4a). For larger H/D ratios, the behavior is flexure dominant (Figure 5.4b). For intermediate H/D ratios both the flexure-shear mode will be governing the failure (Figure5.4c).

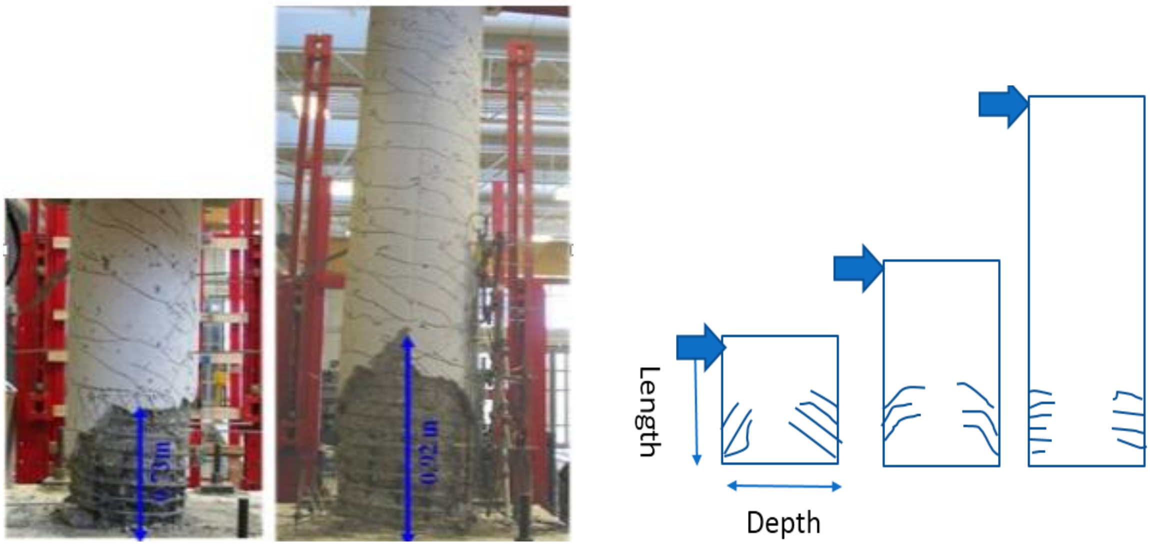


Fig.5.4 a) short column **b)** long column **c)** Variation in H/D ratios

5.5 PLASTIC HINGE LENGTH

A series of sensitivity analyses were conducted to identify the primary variables influencing the length of plastic hinges. The sensitivity analyses showed that axial load, shear span-to-depth ratio and amount of longitudinal reinforcement had significant influences on the length of plastic hinges. Based on the analysis results, linear relationships between these parameters (*P*/*Po*, *L*/*h* and *As*/*Ag*) and the plastic hinge length are used in calibrating the plastic hinge length expression for simplicity. Equation (5.4) is the result of a series of least squares analyses conducted on the UW/PEER column database (proposed by Sung-jin bae, 2005).Accurate estimation of the length of a plastic hinge formation in a reinforced concrete column plays an important role in estimating the displacement capacity of the column. Given the moment-curvature response of a column section, the lateral load-tip deflection response of the column can be obtained with relative ease if the plastic hinge length is known. The formation of plasic hinge portion and practical damaged portions in the column are shown in the Figures 5.7 and 5.8.

In order to evaluate lateral load-tip deflection response of concrete columns, a Mat lab program, developed during the course of this research study, is used.

$$\frac{l_p}{h} = [0.3(\frac{P}{P_0}) + 3(\frac{A_s}{A_g}) - 0.1]\frac{L}{h} + 0.25 \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots (5.4)$$

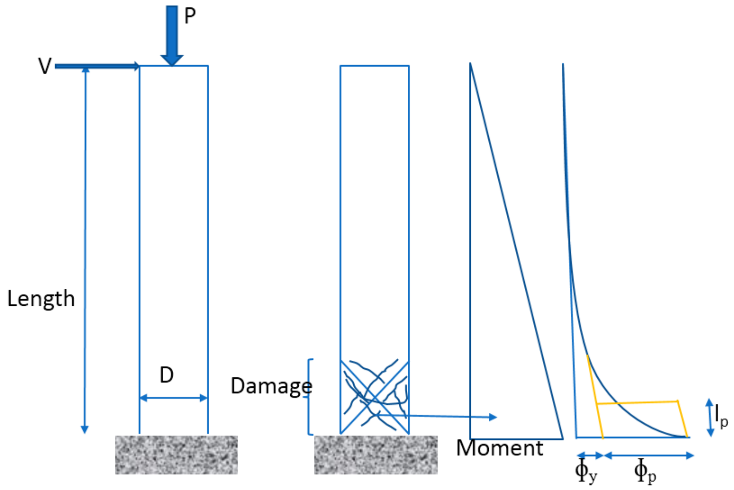


Figure 5.5. Effective plastic hinge length

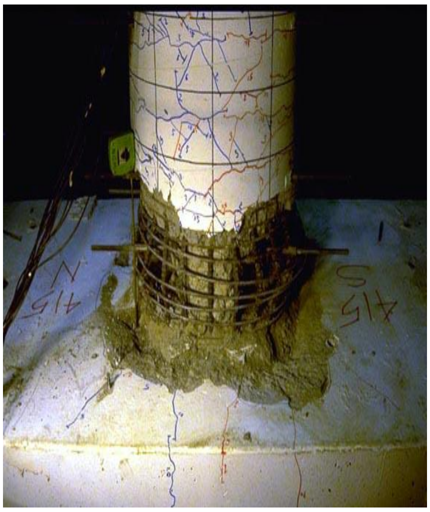


Figure 5.6. Plastic Hinge(column tested by Lehman)

5.6 MEMBER LEVEL BEHAVIOUR

For a given sectional performance, the member behaviour of a column can be estimated if lp is known. As such, estimating the length of a plastic hinge establishes a key step in predicting the lateral load-drift response of a column. A computer program in MATLAB was developed to predict the load deflection behaviour for different set of column data as shown in the Figure.5.10. The algorithm shown in Figure 5.9 is followed. Plastic hinge length method proposed by Priestley et al. (2007) is used in this study. Bae (2005), proposed an improved expression for plastic hinge length as shown in Equation. 5.5. Sensitivity analysis was carried out to propose a new equation for circular columns by validating with experimental data. Based on the analysis, this equation was scaled down by 0.75 to capture the test data well. The proposed model captured the behaviour very well compared to all other models before and after yield point (Figure 5.10). The slip and shear deflections were taken according to the

Equations.(5.5) and (5.6). Columns (a) and (d) were tested by Prakash et.al. (2009). Column (b) was tested by Lehman and column (c) was taken from NIST.

$$\frac{l_p}{h} = [0.3(\frac{P}{P_0}) + 3(\frac{A_s}{A_g}) - 0.1]\frac{L}{h} + 0.25 \ldots\ldots (5.5)$$

$$\Delta_{slip} = L\theta_{slip} \ldots\ldots (5.6)$$

$$\theta_{slip} = (0.5\phi f_y d_b)/(4*12*\sqrt{f_c}) \ldots\ldots (5.7)$$

where A_s = Area of longitudinal reinforcement, P=axial load, P_0 =balanced load, h=least lateral dimension of the column, l_p =plastic hinge length, ϕ =curvature, θ_{slip} =rotation w.r.t neutral axis and Δ_{slip} =slip displacement. Sectional moment capacity and the corresponding member level load levels for all the columns are shown in Table 5.1.

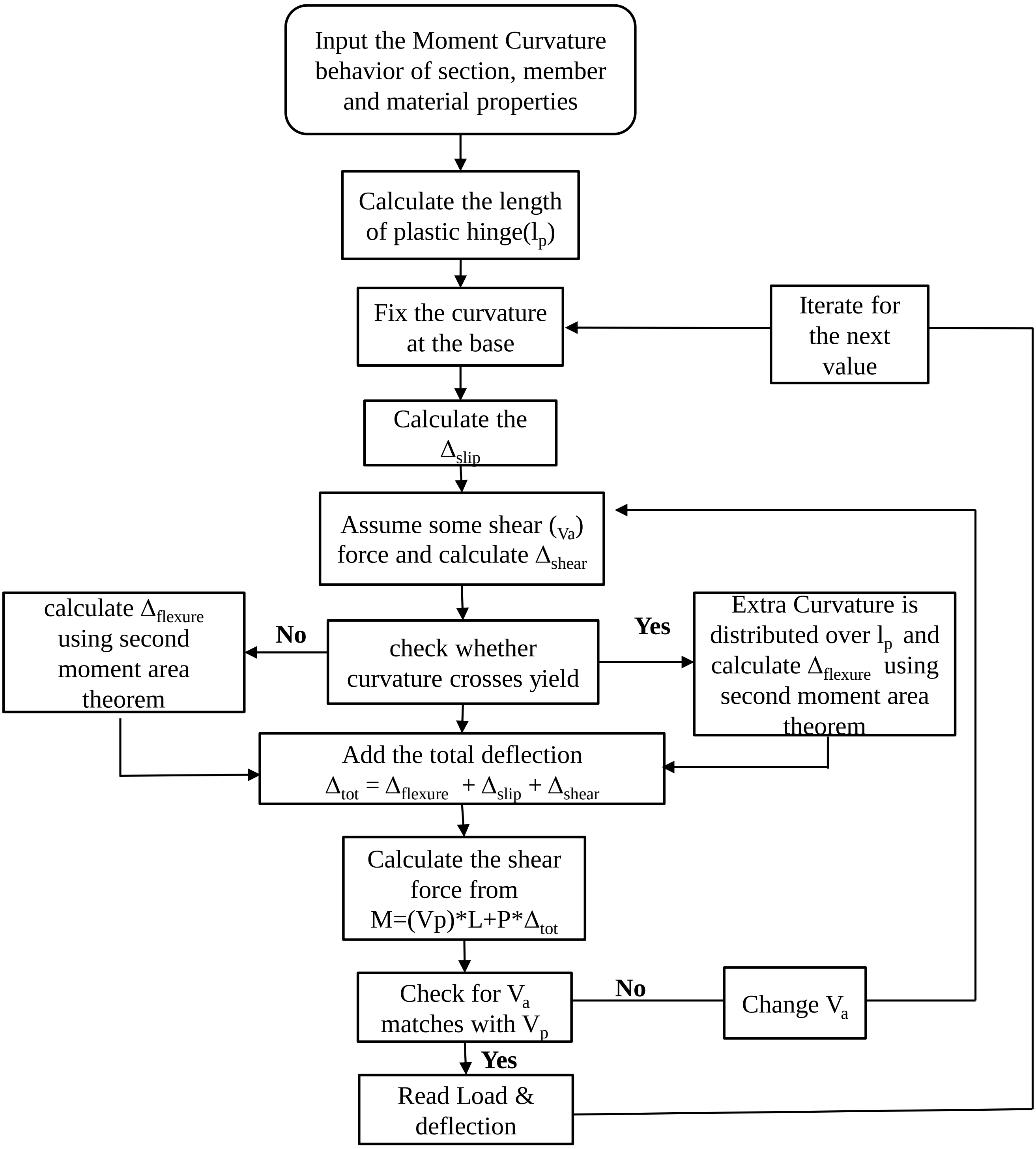
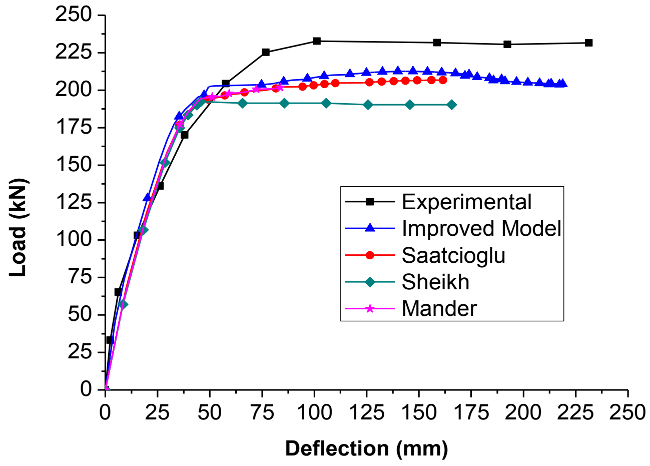
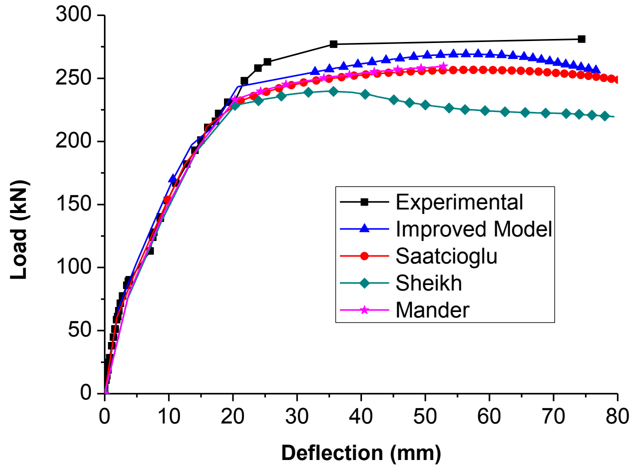


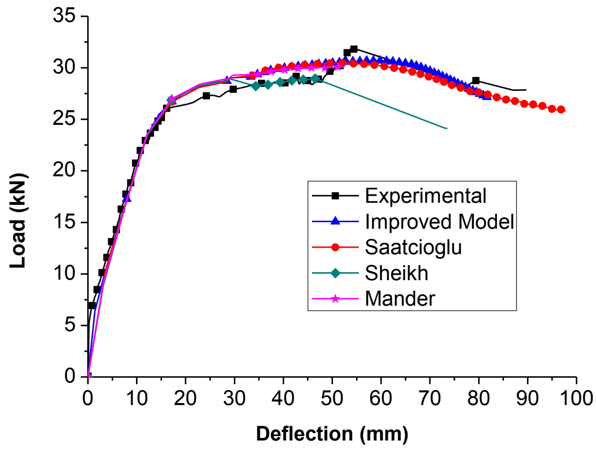
Figure 5.7. Algorithm for Load deflection Behaviour



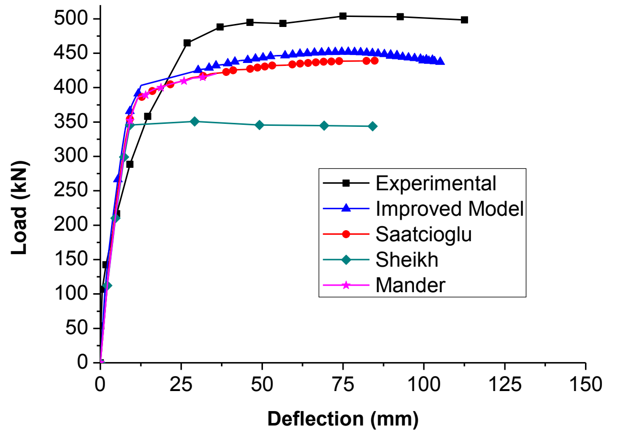
Column(a):Missouri-H/D(6)-2.10%-0.73%-6.16%



Column(b): Lehman-H/D(4)- 1.5%-0.72%-7.2%



Column(c):NIST-H/D(6.01)- 2.0%-1.49%-9.6%



Column(d):Missouri-H/D(3)-2.10%-1.32%-6.16%

Figure 5.8. Comparison of Load Displacement Behaviour with the different Confinement Model

Table 5.1 Evaluation of Proposed Model for Different Test Results

Parameter	Missouri -H/D(6)- 2.10%- 0.73%- 6.16%	Missouri -H/D(3)- 2.10%- 1.32%- 6.16%	Lehman- H/D(4)- 1.5%- 0.72%- 7.2%	Lehman- H/D(10)- 1.5%- 0.72%- 7.2%	Lehman- H/D(4)- 0.75%- 0.72%- 7.2%	Lehman- H/D(4)- 2.98%- 0.72%- 7.2%	NIST- H/D(6.01)- 2.0%- 1.49%- 6.9%	NIST- H/D(6.01) - 2.0%- 1.49%- 9.6%
Sectional Evaluation $M_u / M_{u(exp)}$	0.88	0.88	1.01	1.01	1.05	1.04	1.03	1.11
Member Evaluation $P_u / P_{u(exp)}$	0.89	0.88	0.98	0.94	0.91	0.99	0.95	0.96
Δ_u / Δ_{exp}	0.95	0.95	1.04	1.01	1.2	1.1	0.99	0.89

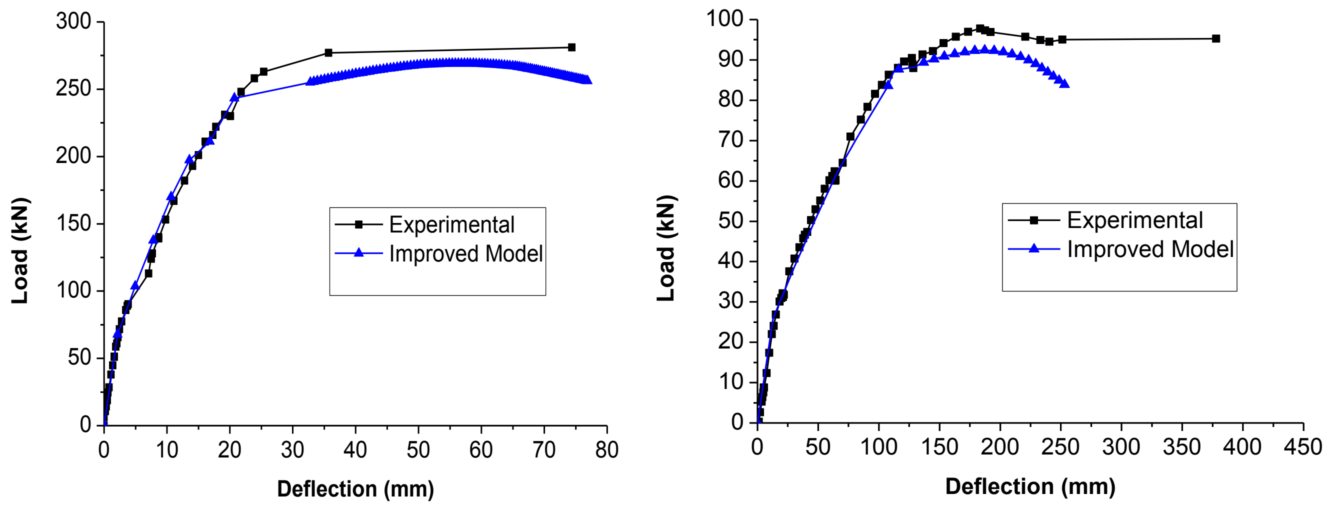
CHAPTER 6

EVALUATION OF THE PROPOSED MODEL FOR DIFFERENT PARAMETERS

The proposed improved model is evaluated over a range of parameters like different longitudinal reinforcement, axial load ratio and shear span or H/D ratios and the results are presented in the following sections.

6.1 EFFECT OF SHEAR SPAN ON BEHAVIOUR

Two columns tested by Lehman (1998) with same sectional parameters and axial load but with different aspect ratios (H/D) of four and ten are used for evaluation. This helps to validate the proposed model for capturing the behaviour of columns with different flexure to shear ratios. Improved model is able to capture the change in behaviour due to different H/D ratios (Figure 6.1). It is closely capturing the behaviour up to yielding point. The ultimate strength is also matching with the test data.

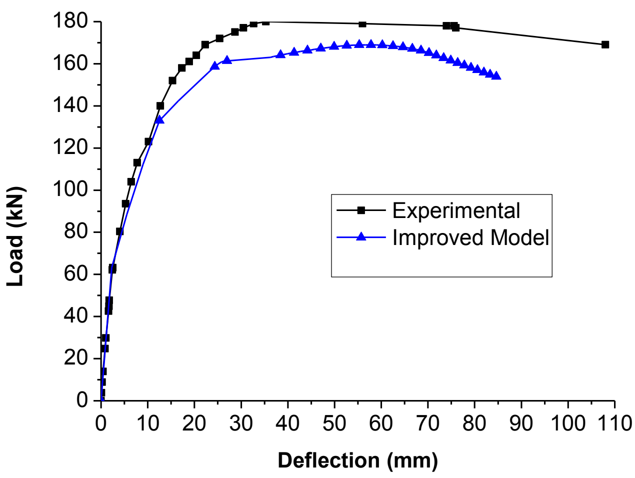


Column (a): Lehman-H/D(4)- 1.5%-0.72%-7.2% **Column (b): Lehman-H/D(10)- 1.5%-0.72%-7.2%**

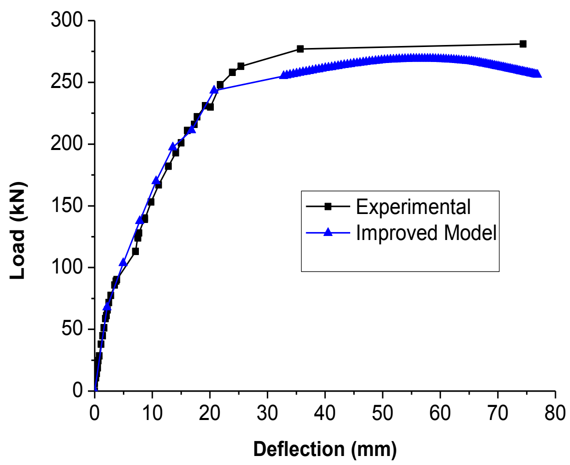
Figure 6.1. Comparison of Load Displacement Behaviour using Proposed Model with varying H/D Ratios

6.2 EFFECT OF LONGITUDINAL REINFORCEMENT ON BEHAVIOUR

Two columns tested by Lehman (1998) with same sectional parameter, aspect ratio and axial load but with different longitudinal reinforcement ratios (0.75%, 1.5%) are used for evaluation. This will help to validate the proposed model for capturing the behaviour of columns with different longitudinal reinforcement ratios. As shown in the Figure 6.2, the proposed improved model is very well capturing the change in the longitudinal reinforcement ratios. It is closely capturing the behaviour up to yielding point. The ultimate strength is also closely predicted.



Column(a): Lehman-H/D(4)- 0.75%-0.72%-7.2%

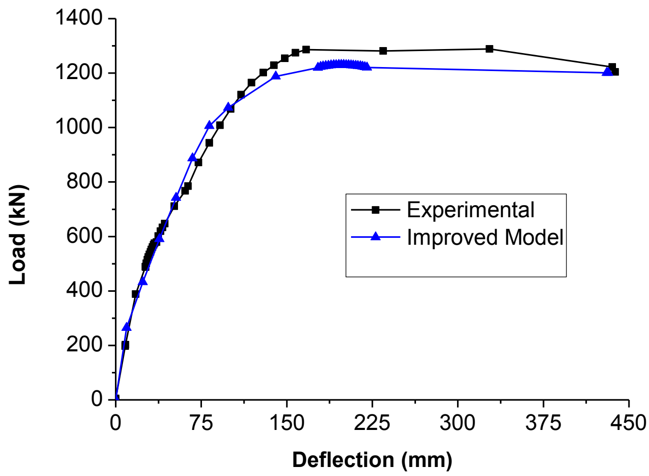


Column (b): Lehman-H/D(4)- 1.5%-0.72%-7.2%

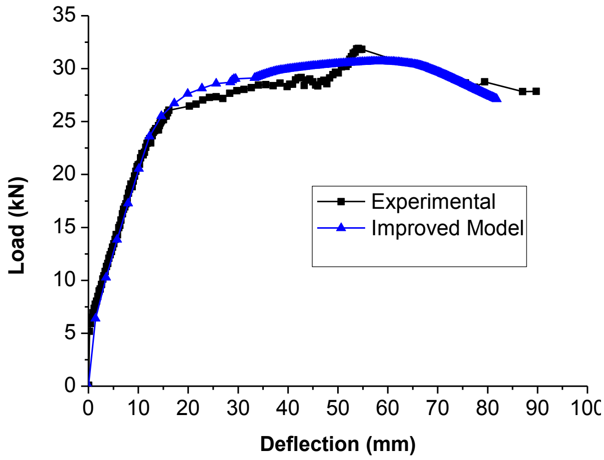
Figure 6.2. Comparison of Load Displacement Behaviour using Proposed Model with varying Longitudinal Reinforcement Ratios

6.3 EFFECT OF AXIAL LOAD RATIOS ON BEHAVIOUR

Two columns tested by Taylor and Stone (1993) at NIST with same sectional parameter, aspect ratio and but with different axial load ratios (6.9%, 9.6%) are used for evaluation. This helps to validate the proposed model for capturing the behaviour of columns with different axial load ratios. As shown below in the Figure 6.3, the improved model is very well capturing the change in the axial load ratios. The ultimate strength also closely predicted. It is worth mentioning that the proposed model is not validated for high axial load ratios. A very investigations in the past has investigated the effect of high axial load ratios on the flexure behaviour of RC columns. Future research should focus on this aspect.



Column(a): NIST-H/D(6.01)- 2.0%-1.49%-6.9%



Column(b): NIST-H/D(6.01)- 2.0%-1.49%-9.6%

Figure 6.3. Comparison of Load Displacement Behaviour using Proposed Model with varying Axial Load Ratios

CHAPTER 7
CONCLUSIONS

Confinement models proposed by various researchers were studied. Three widely used confinement models were evaluated with experimental behaviour of specimens tested by the author and other investigators. A new confined stress strain curve for RC circular section was proposed based on the evaluation of various sectional and member level properties. Columns used for evaluation were tested under combined effects of axial compression, bending moment and shear forces for a wide range of parameters. Different levels of bending moment to shear (or) shear span ratios, reinforcement ratios, and axial load levels were considered for evaluation. Based on the results presented in this study, the following major conclusions can be drawn:

- Simplified approach based on plastic hinge length method using the new confinement model replicated the force displacement behaviour of columns with different aspect ratio, axial load levels and longitudinal reinforcement ratios closely.
- The influence of confinement ratio is found to be significant on the performance of RC circular columns under flexure. Increase in transverse reinforcement ratio increased the peak confined concrete strength and the ultimate strain and post-peak stiffness of confine stress strain curves.
- Increase in longitudinal reinforcement ratio increased the peak confined concrete strength and the ultimate strain and post-peak stiffness of confined stress strain curves.

- P-Delta effect is a function of the lateral displacement and the level of axial load. Therefore, improving the sectional performance at high levels of axial load may not increase the lateral deformation capacity of a column.
- The level of axial compression influenced the ultimate strength and displacement significantly. The loss of lateral load carrying capacity increased due to P-Delta effect with increase in axial compressive load.
- Experimental data on behaviour of RC circular columns under flexure and shear with high levels of axial compression is very limited. Future work should focus on evaluation of the proposed model under high axial compression loads and for other parameters included in this study.

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