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FLEXURAL RETROFIT OF CIRCULAR REINFORCED BRIDGE COLUMNS BY STEEL JACKETING

COLRET - A Computer Program for Strength and Ductility Calculation

by

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Final Report on a Research Project funded through Caltrans under Contract Number F88SD06 and Agency Grant Number RTA-59G267

October 1991

Department of Applied Mechanics and Engineering Sciences
University of California, San Diego
La Jolla, California

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Structural Systems Research Project

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Graduate Research Assistant

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16. Abstract

This report presents the development of an analytical model, based on moment-curvature analysis aT the critical section, for the prediction of flexural strength and ductility in prismatic circular bridge columns. Columns are treated as vertical cantilevers subjected to axial compression and lateral inertial force. Passive confinement of the concrete core by the transverse steel is accounted for using a realistic model for confined concrete. Post-yield deformation of the column is predicted on the basis of an elasto-plastic idealization with plasticity concentrated in a localized hinged region. Failure modes such as that precipitated by shear or bond-slip at the lap-splices of the longitudinal bars are not considered in the model.

The analytical model is extended to columns with fully grouted steel jackets which are assumed to contribute to the ductility of the column by providing confinement to the column concrete without significant enhancement of the flexural strength. The increase in lateral stiffness of the column due to the steel jacketing is taken into account by considering the effective bond transfer between the jacket and column for composite action. Deformation beyond yield of the column occurs in a reduced plastic hinge length. The increase of shear strength in the encased region of the column is also considered.

The model is coded in a computer program to provide bridge engineers a simple and reliable means of assessing the perfomance of existing bridge piers so that deficient piers can be identified, and to allow an assessment of the improved column performance after retrofit. The reliability of the model is verified through experimental testing of large-scale column units.

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Table of Contents

List of Figures

List of Tables

Chapter 1

Introduction

1.1 Summary

This report outlines the development of a simple analysis program COLRET for the estimation of the flexural strength and ductility of prismatic circular bridge columns under seismic loading. Bridge columns are treated as vertical cantilevers subjected to axial compression and lateral inertial force. Passive confinement of the columns by internal hoops or spirals are taken into account by a well calibrated stress-strain model for confined concrete. Post-yield flexural deformation of the column is predicted on the basis of an elasto-plastic idealization with plasticity concentrated in a localized hinge region. Strain penetration of longitudinal bars into the footing is accounted for using an increased column height. The ductility capacity of the column is assessed in terms of a displacement ductility factor. Failure modes such as that precipitated by shear or bond-slip in the laps of the longitudinal bars are not considered in the model. The footing is assumed to provide full fixity against translations and rotations.

The computer program also includes a retrofit option which assumes partial encasement in the lower critical region of the column by a steel jacket. Composite action between the column and jacket is effected by grout infill. The steel jacket is assumed to contribute to the ductility of the column by providing confinement to the concrete, but without significant enhancement to the flexural capacity. The jacket also acts as transverse reinforcement in resisting the column shear force in the encased region.

The objectives of the computer program developed in this report are:

1. to provide bridge engineers a simple and reliable mean for assessing the performance of existing bridge piers so that deficient piers can be identified from the large domain of existing bridges for retrofit, and
2. to allow an assessment of the improved column performance after retrofit.

The reliability of the program is verified by comparing predictions with results from large-scale column tests carried out at the University of California, San Diego and elsewhere [1,2]. A user guide with examples is included to illustrate the use of the program.

1.2 Background

Collapse or severe damage to a large number of highway bridges in the region of fault rupture during the 1971 San Fernando earthquake [3,4] prompted Caltrans to initiate an extensive retrofit program to upgrade the seismic resistance of highway bridges in California [5]. The greatest risk of bridge collapse was felt to be associated with the large relative displacements which occurred between bridge piers under the earthquake induced motion. The relative displacements in some cases were of sufficient magnitude to dislodge the superstructure from its seating positions and caused the entire span to fall off the supporting piers. Phase I of the retrofit program by Caltrans involved securing adjacent spans of the bridge superstructure using restrainer devices across the movement joints. Detailed descriptions on the methods of retrofitting and experimental testings of these restrainer devices have been reported [6,7]. This phase of superstructure retrofit was completed in 1988 [8].

Major deficiencies are also associated with the substructures of the older bridges, as evident by the bridge failures in the recent 1987 Whittier Narrows earthquake [9,10] and 1989 Loma Prieta earthquake [11], as well as those of the 1971 San Fernando earthquake [3,4]. Substructural deficiencies include inadequate flexural strength, as well as ductility in tall columns, inadequate shear strength in squat columns, and inadequate footing or joint strengths. These deficiencies have been discussed in a companion research report [1].

A cost-effective method for enhancing the flexural strength and ductility of deficient circular bridge columns can be achieved by encasing the critical regions of the column with a steel tube or jacket, as shown in Figure 1.1. Two half shells are welded together along a longitudinal seam to form a tubular sleeve over the bottom region of the column. The jacket is slightly oversized to allow a cement-based grout to be pressure-injected and provide composite action between the column and steel tube. It is assumed that a small vertical gap exists between the toe of the steel jacket and the footing. This is to ensure that the steel jacket does not bear against the footing when in compression and contribute further to the flexural capacity of the column. The basis for this approach was the excellent ductile response of steel-encased concrete piles tested by Park et al [12,13,14], as demonstrated in Figure 1.2.

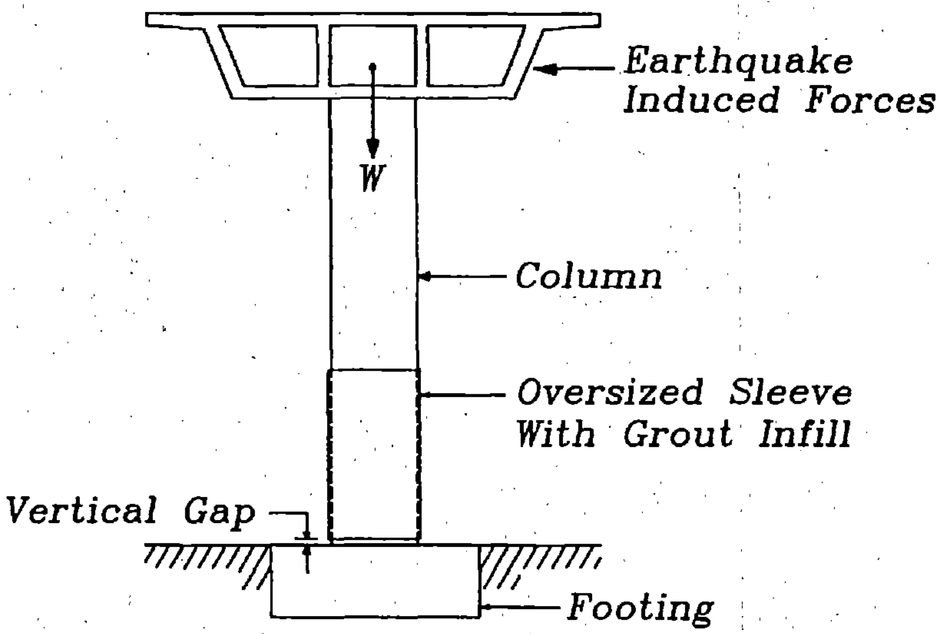


Figure 1.1: **Steel Jacketing of Circular Bridge Columns**

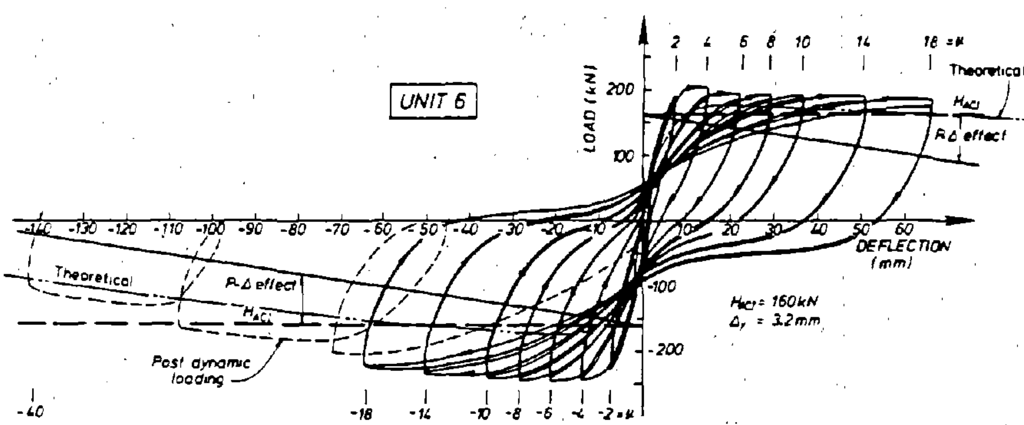


Figure 1.2: **Ductile Response of Steel Encased Concrete Piles (Park et al - Ref. 12**

Chapter 2

Constitutive Properties

2.1 Concrete

Two different regimes must be considered for the concrete portion of the column section, i.e. the core and cover portion of the section. The core portion is defined to be the circle contained by the centerline of the spiral or hoop. The behavior of this concrete core is affected by the presence of transverse reinforcement, since a passive confinement of the core is provided by the transverse reinforcement as concrete dilates laterally under compression. The cover concrete, on the other hand, responds under unconfined conditions and must be described by a different stress-strain curve. Limited ductility is associated with the cover concrete since crushing occurs at an early strain of about 0.005, and subsequent loading leads to development of longitudinal cracks and eventual separation of the cover concrete from the concrete core. The cover concrete therefore cannot be relied upon to resist stresses at high strains. Confinement however will be provided to the cover concrete if the column is encased with a steel jacket. In such cases, the column core will be subjected to two levels of passive confinement; one from the transverse steel and the other from the external steel jacket. It is further assumed that the tensile strength of both confined and unconfined concrete can be ignored for the calculation of the yield and ultimate displacements.

2.1.1 Confined Concrete

Transverse reinforcement at close spacing has been shown to enhance significantly the performance of a concrete member under compression in the inelastic load range [15,16]. A substantial increase in concrete compressive strength was observed even for columns with relatively small volumetric confinement ratio of $\rho_s = 0.6\%$ (ρ_s defined later in Eqn. 2.15). Post-peak deformation capacity were greatly enhanced with a much more gradual falling branch in its stress-strain curve, when compared to the unconfined concrete. The transverse reinforcement provide lateral confining pressure to the core concrete which delay the propagation of microcracks as stress level approaches that of the unconfined compressive strength. The presence of lateral confining pressure allows the development of much higher axial strains until first fracture of the transverse reinforcement, after which the column experiences a sudden drop in the compression capacity due to a reduction of confinement for the core concrete and a loss of restraint against compression buckling of longitudinal bars. Average axial strains as high as 0.06 have been observed in columns containing 2% volumetric confinement ratio [17]. The enhancement of compressive strength and ultimate strain in confined concrete is illustrated in Figure 2.1.

A model recently proposed by Mander et al [18] has been shown to provide excellent prediction of the compressive response of large-scale columns confined by a wide range of transverse reinforcement confinement ratios. The attractiveness of the model lies in its use of a single equation for the entire range of concrete compressive strain, and is applicable to columns confined by circular or rectangular shaped transverse reinforcement. According to the model, the

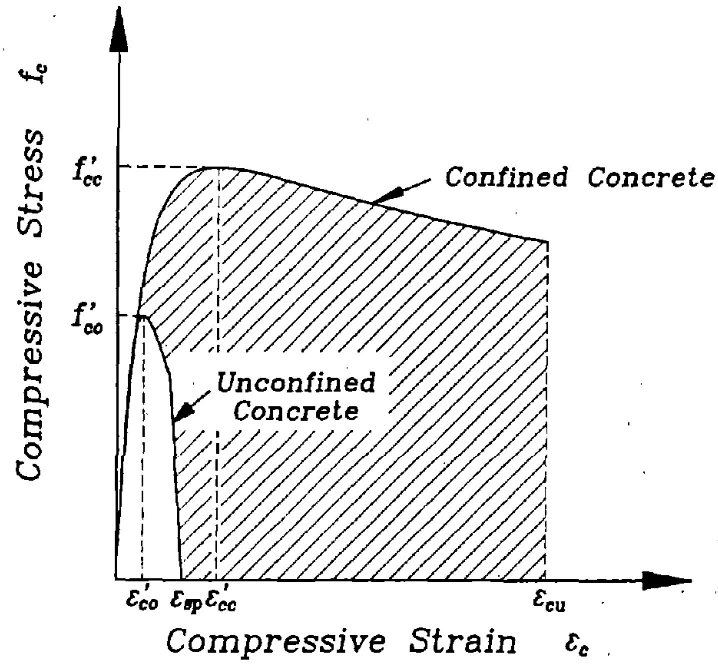


Figure 2.1: **Confining Effect on Compressive Response of Concrete**

longitudinal compressive stress of confined concrete is given by:

$$f_c = \frac{f'_{cc}.x.r}{r - 1 + x^r} \tag{2.1}$$

where f'_{cc} = compressive strength of confined concrete (defined later in Eqn. 2.6); x = longitudinal compressive strain, ϵ_c, divided by concrete compressive strain at f'_{cc} i.e. ϵ'_{cc}. The suggested expression for ϵ'_{cc} increases linearly with f'_{cc} and is given by:

$$\epsilon'_{cc} = \epsilon'_{co}\{1 + 5(\frac{f'_{cc}}{f'_{co}} - 1)\} \tag{2.2}$$

where f'_{co} and ϵ'_{co} = the unconfined concrete compressive strength and corresponding strain, respectively. A value of 0.002 is adopted for ϵ'_{co} and the parameter r is given by:

$$r = \frac{E_c}{E_c - E_{sec}} \tag{2.3}$$

where

$$E_c = 60200\sqrt{f'_{co}} \tag{2.4}$$

is the tangent modulus of elasticity for unconfined concrete (f'_{co} in psi units), and

$$E_{sec} = \frac{f'_{cc}}{\epsilon'_{cc}} \tag{2.5}$$

is the secant modulus for confined concrete, defined with respect to $(f'_{cc}, \epsilon'_{cc})$.

For the confined concrete compressive strength, f'_{cc}, Mander [19] used the five-parameter failure criterion proposed by Willam and Warnke [20] and the triaxial tests data of Schickert and Winkler [21]. In the case of circular columns confined by circular spiral or hoops, the confined concrete compressive strength f'_{cc} has been shown to be [19]:

$$f'_{cc} = f'_{co}(2.254\sqrt{1 + \frac{7.94f'_l}{f'_{co}}} - \frac{2f'_l}{f'_{co}} - 1.254) \tag{2.6}$$

where f'_l = effective confining pressure, and may be obtained from the equilibrium of internal forces acting the dissected sections shown in Figure 2.2.

For the cover concrete and grout in retrofitted columns, the equilibrium of forces assuming uniform yield of the jacket requires:

$$f'_{lj} = \frac{2f_{yj}t_j}{(D_j - 2t_j)} \tag{2.7}$$

where f'_{lj} = lateral pressure acting on the cover concrete; D_j and t_j = outside diameter and thickness of the jacket, respectively; and f_{yj} = yield strength of the steel jacket. By defining a confining ratio for the steel jacket as:

$$\rho_{sj} \equiv \frac{4t_j}{D_j - 2t_j} \tag{2.8}$$

Eqn. 2.7 may be written as

$$f'_{lj} = \frac{1}{2}\rho_{sj}f_{yj} \tag{2.9}$$

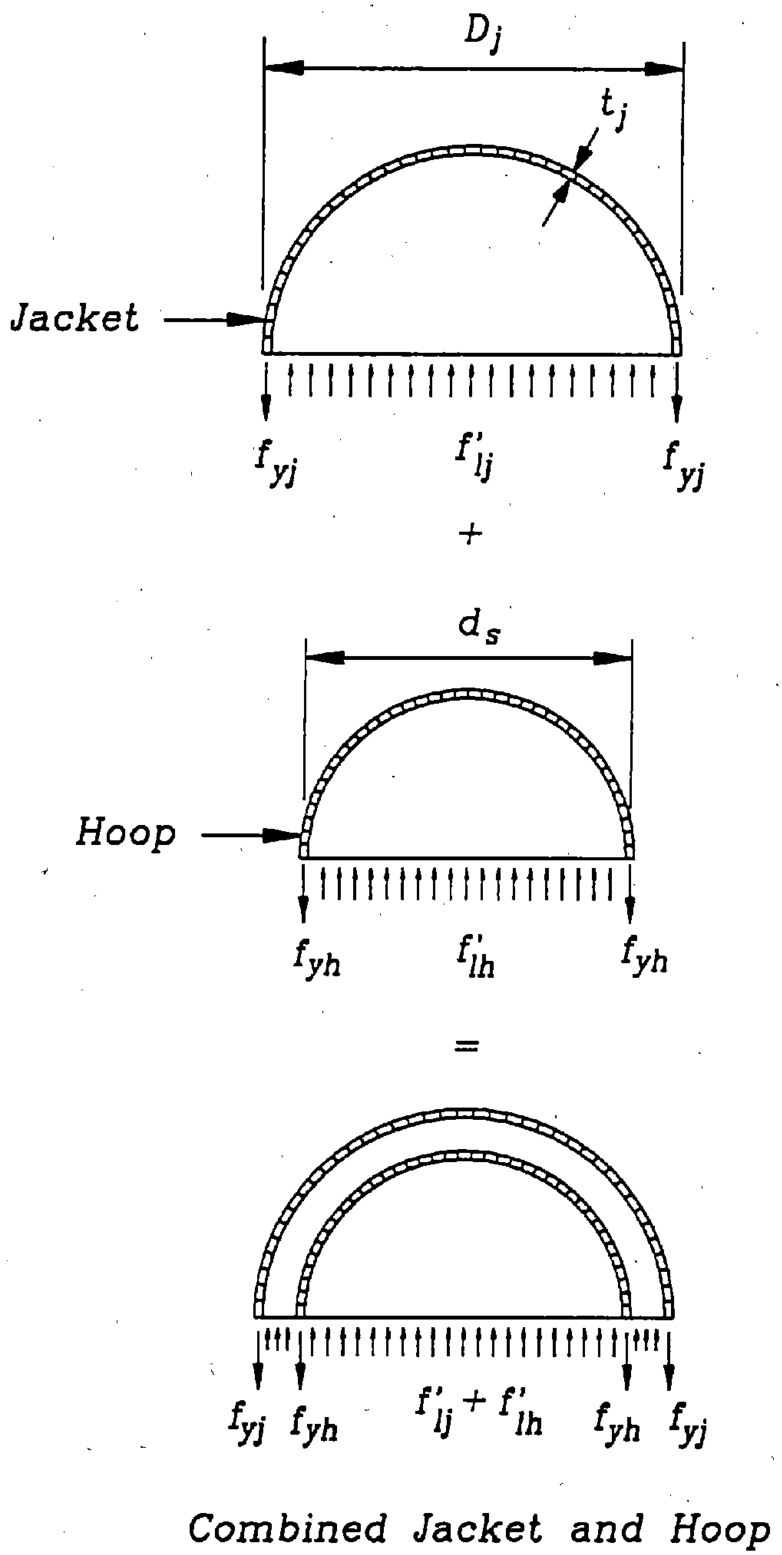


Figure 2.2: **Confining Action of Steel Jacket and Internal Hoops**

By substituting $f'_l = f'_{lj}$ into Eqn. 2.6, the compressive strength of cover concrete enhanced by steel jacket can be determined.

For the concrete core, an additional confinement is provided by the transverse steel. The additional lateral pressure, f'_{lh}, may also be determined from the equilibrium of forces, assuming uniform yield of the transverse steel i.e.

$$f'_{lh} = 2k_e \frac{f_{yh} A_{sh}}{d_s s} \tag{2.10}$$

where d_s = diameter of concrete core defined along the center line of transverse steel; s = vertical spacing of the transverse steel; f_{yh} = yield strength of the transverse reinforcement; A_{sh} = cross-sectional area of the transverse steel. The parameter k_e is termed as the confinement effectiveness coefficient and is defined as:

$$k_e \equiv \frac{A_e}{A_{cc}} \tag{2.11}$$

where A_e = area of an effectively confined concrete core (see Figure 2.3); $A_{cc} = A_c(1 - \rho_{cc})$ where ρ_{cc} = ratio of area of longitudinal reinforcement to core area of the section A_c, i.e.

$$\rho_{cc} = \frac{4A_s}{\pi {d_s}^2} \tag{2.12}$$

where A_s = total longitudinal steel area. By assuming an arching action between circular hoops in the form of a second-degree parabola with an initial tangent slope of 45^0, the confinement effectiveness coefficient k_e in Eqn. 2.11 has been shown to be [19]:

$$k_e = \frac{(1 - 0.5\frac{s'}{d_s})^2}{(1 - \rho_{cc})} \tag{2.13}$$

where s' = clear distance between spiral or hoop. Similarly, the confinement

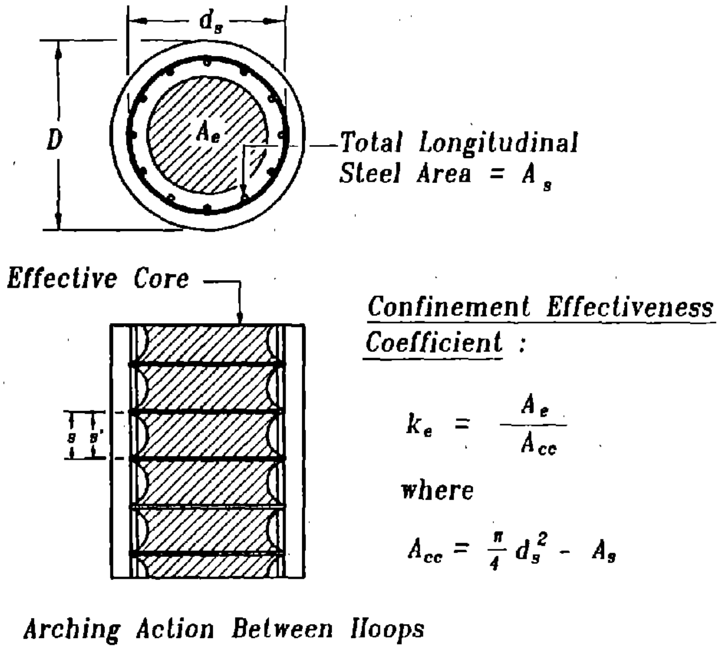


Figure 2.3: **Definition of Confinement Effectiveness Coefficient**

effectiveness coefficient for a circular spiral has been shown to be:

$$k_e = \frac{(1 - 0.5\frac{s'}{d_s})}{(1 - \rho_{cc})} \tag{2.14}$$

By introducing ρ_s as the ratio of the volume of transverse confining steel to the volume of confined core i.e.

$$\rho_s \equiv \frac{A_{sh}\pi d_s}{\frac{\pi}{4}{d_s}^2 s} = \frac{4A_{sh}}{d_s s} \tag{2.15}$$

the lateral confining pressure due to transverse steel in Eqn. 2.10 may be written as:

$$f'_{lh} = \frac{1}{2} k_e \rho_s f_{yh} \tag{2.16}$$

Thus the substitution of $f'_l = f'_{lj} + f'_{lh}$ into Eqn. 2.6 will allow the enhanced compressive strength of the concrete core to be determined.

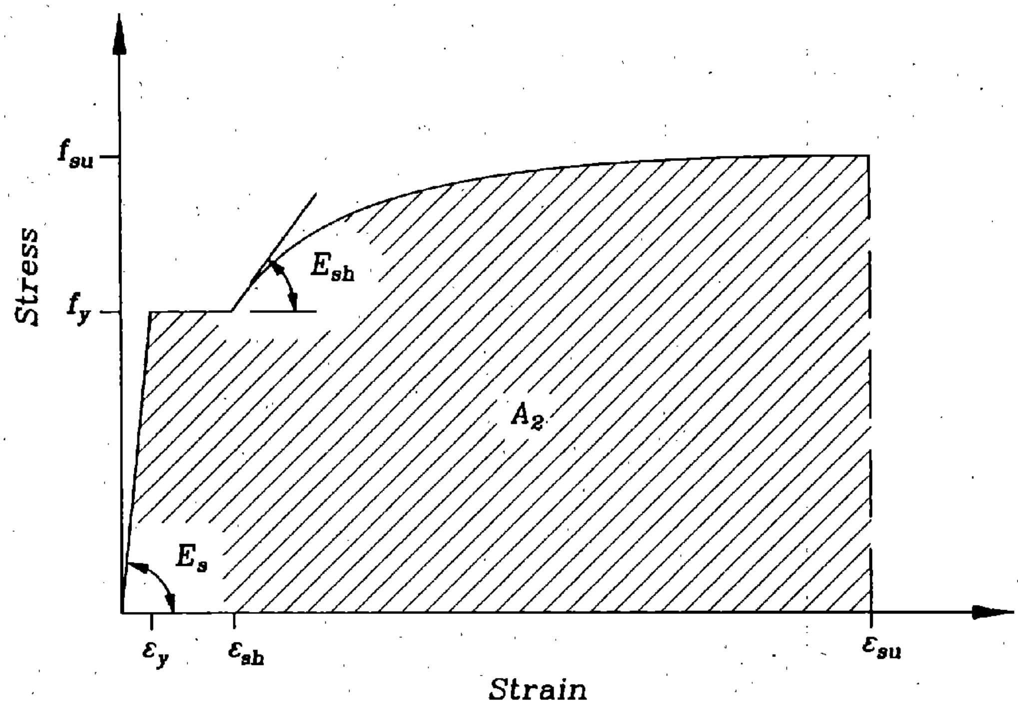


Figure 2.4: **Stress-Strain Curve for Reinforcing Steel**

2.1.2 Unconfined Concrete

For the 'as-built' columns, the unconfined condition in the cover concrete may be simulated by putting the lateral confining pressure to zero i.e. $f_l' = 0$. The following simplifications can be made to the equations for confined concrete given in Section 2.1.1:

$$f'_{cc} = f'_{co} \tag{2.17}$$

$$\epsilon'_{cc} = \epsilon'_{co} \tag{2.18}$$

$$E_{sec} = \frac{f'_{co}}{\epsilon'_{co}} \tag{2.19}$$

$$x = \frac{\epsilon_c}{\epsilon'_{co}} \tag{2.20}$$

Even though the falling branch of Eqn. 2.1 represents well the rapid drop of concrete stress with strain in the post-peak range of the unconfined concrete, the concrete stress does not decrease to zero for large concrete strains. It is therefore assumed that the stress-strain curve for unconfined concrete follows Eqn. 2.1 during the earlier stages of loading up to $2\epsilon'_{co}$. For compressive strains larger than $2\epsilon'_{co}$, the stresses are assumed to decrease linearly with strains up to the spalling strain ϵ_{sp}. A value of 0.005 has been adopted for ϵ_{sp}. Thus the longitudinal compressive stress for unconfined concrete may be written as:

For $\epsilon_c \leq 2\epsilon'_{co}$,

$$f_c = \frac{f'_{co}.x.r}{r - 1 + x^r} \tag{2.21}$$

For $2\epsilon'_{co} < \epsilon_c \leq \epsilon_{sp}$,

$$f_c = f'_{co}(\frac{2r}{r - 1 + 2^r})(1 - \frac{\epsilon_c - \epsilon'_{co}}{\epsilon_{sp} - 2\epsilon'_{co}}) \tag{2.22}$$

For $\epsilon_{sp} < \epsilon_c$,

$$f_c = 0 \tag{2.23}$$

2.2 Reinforcement

While earlier design practices tended to use large diameter bars, up to #14 or #18, to avoid congestion of reinforcement, such practice may lead to potential bond problem in cases where the column main reinforcement were lapped at insufficient length with starter bars in the plastic hinge regions [8,1]. Consequently, such columns are characterized by very rapid flexural strength degradation under the design seismic loads. The current Caltrans approach [22] has been to avoid lap-splicing of the main reinforcement in the potential plastic hinge region of bridge columns. Such deficiency however may be rectified by a fully grouted steel jacket, as shown in recent large-scale column tests carried out at the University of California, San Diego [1]. The analytical model developed here assumes full yield of the main reinforcement including strain-hardening.

2.2.1 Longitudinal Reinforcement

The monotonic uniaxial stress-strain response of a typical reinforcing steel is characterized by a distinct elastic region, a yield plateau, a strain-hardening region, followed by a falling branch after peak stress up to bar fracture. A generic stress-strain curve for the reinforcing steel up to the maximum stress is shown in Figure 2.4.

The equations describing the monotonic uniaxial stress-strain curve up to ultimate strain are:

For the elastic range, i.e. $\epsilon_s \leq \epsilon_y$,

$$f_s = E_s \epsilon_s \tag{2.24}$$

where ϵ_s, f_s = axial strain and stress in reinforcing steel, respectively; ϵ_y = yield strain of reinforcing steel; and E_s = modulus of elasticity of reinforcing steel.

For the yield plateau, i.e. $\epsilon_y < \epsilon_s \leq \epsilon_{sh}$,

$$f_s = f_y \tag{2.25}$$

where ϵ_{sh} = axial strain at the on-set of strain-hardening; and f_y = yield stress of the reinforcing steel.

For the strain-hardening range i.e. $\epsilon_{sh} < \epsilon_s \leq \epsilon_{su}$,

$$f_s = f_y \left(\frac{m(\epsilon_s - \epsilon_{sh}) + 2}{60(\epsilon_s - \epsilon_{sh}) + 2} + \frac{(\epsilon_s - \epsilon_{sh})(60 - m)}{2(30 r_s + 1)^2} \right) \tag{2.26}$$

where ϵ_{su}, f_{su} = ultimate strain and stress in reinforcing steel, respectively, and

$$m = \frac{(f_{su}/f_y)(30 r_s + 1)^2 - 60 r_s - 1}{15 r_s^2} \tag{2.27}$$

$$r_s = \epsilon_{su} - \epsilon_{sh} \tag{2.28}$$

In the study by Mirza and MacGregor [23] on the variability of reinforcing steel in North America, a mean yield strength of 48.8 ksi and 71 ksi were obtained for Grades 40 and 60 steel, with coefficients of variation of 10.7% and 9.3%, respectively. The mean modulus of elasticity, E_s, was 29200 ksi with a coefficient of variation of 3.3%. It was also found that for both grades of steel, the ratio of ultimate to yield strength was $f_{su}/f_y = 1.55$. The steel model adopted for the program assumes a modulus of elasticity of 29000 ksi, and a slightly lower ultimate to yield strength ratio of 1.50. Other mechanical properties assumed for the stress-strain model are:

For Grade 40 steel,
$$\epsilon_{sh} = 14\epsilon_y$$
$$\epsilon_{su} = 0.14 + \epsilon_{sh}$$

For Grade 60 steel,
$$\epsilon_{sh} = 5\epsilon_y$$
$$\epsilon_{su} = 0.12$$

Note that the tangent modulus at the on-set of strain-hardening, E_{sh}, may be obtained by taking the derivative of Eqn. 2.26 with respect to steel strain, ϵ_s, and operated at the strain-hardening strain, ϵ_{sh}:

$$E_{sh} = f_y \left(\frac{2m - 120}{4} + \frac{60 - m}{2(30r_s + 1)^2} \right) \tag{2.29}$$

2.2.2 Transverse Reinforcement

The provision of closely-spaced transverse reinforcement in the regions of severe inelastic actions will maintain the integrity of the concrete core and increase the rotational capacity of the column. Maintaining the integrity of the core also allows higher shear forces to be resisted by the concrete. Potential shear failure plane must intersect a larger number of transverse reinforcement which therefore increases the shear resistance. Lateral stability of the longitudinal reinforcement is also improved by the presence of the closely-spaced hoop or spiral. These hoops or spiral act as anti-buckling ties to allow full compression yield of the longitudinal steel to be developed. The integrity of the core and longitudinal steel assures the vertical load carrying capacity of the column after a severe earthquake.

The design requirement for confinement by transverse reinforcement in the potential plastic hinge region of column differs in different design codes. For example, for a column confined by circular hoops or spiral, the ACI 318 Code [24] requires a minimum volumetric ratio of:

$$\rho_s = 0.45\left(\frac{A_g}{A_c} - 1\right)\frac{f'_{co}}{f_{yh}} \tag{2.30}$$

but not less than

$$\rho_s = 0.12\frac{f'_{co}}{f_{yh}} \tag{2.31}$$

where f_{yh} = yield strength of transverse steel which is not to be taken as greater than 60 ksi; A_g = gross sectional area for the column; and A_c = sectional area for the concrete core. It should be noted that the definition of core area A_c (and hence ρ_s) by the ACI 318 Code [24] is referenced to the outside diameter, and not to the centerline diameter of the hoop of spiral, as defined earlier in Section 2.1.1. Similar expressions were adopted by the New Zealand NZ3101 Code [25] but modified to include the influence of axial load, namely:

$$\rho_s = 0.45\left(\frac{A_g}{A_c} - 1\right)\frac{f'_{co}}{f_{yh}}\left(0.5 + 1.25\frac{P_e}{\phi_a f'_{co} A_g}\right) \tag{2.32}$$

but not less than

$$\rho_s = 0.12\frac{f'_{co}}{f_{yh}}\left(0.5 + 1.25\frac{P_e}{\phi_a f'_{co} A_g}\right) \tag{2.33}$$

where P_e = column axial compression from gravity and seismic loading, and is limited to $0.7\phi_s f'_{co} A_g$; and ϕ_a = strength reduction factor = 0.9, if plastic hinging can occur, or 1.0 if the column is protected from plastic hinging by a capacity design procedure. Note that Eqn. 2.32 and 2.33 allows a 50% reduction of the transverse steel required by the ACI 318 Code [24] for no axial load ($P_e = 0$), but requires a 38% more confining steel at the upper limit of $P_e = 0.7\phi_a f'_{co} A_g$. The requirement for greater confinement for high axial load is in recognition of the larger neutral axis depth associated with the increased axial load, resulting in more dependent of the flexural strength and ductility on the stability of the concrete core.

To ensure a ductile response in the columns, confinement must be provided over sufficient length at both ends of the columns where severe inelastic actions may occur. For columns with axial loads $P_e \leq 0.3\phi_a f'_{co} A_g$, the New Zealand NZ3101 Code [25] requires confinement to be provided over a region of at least equal to the larger cross-sectional dimension (or D for circular column), or over the portion of the column where the bending moment exceeds 80% of the maximum moment at that end, whichever is greater. For higher axial loads, $0.3\phi_a f'_{co} A_g \leq P_e \leq 0.7\phi_a f'_{co} A_g$, the extent of confined region is to be increased by 50%. The current ACI 318 Code [24], however, requires confinement to be provided over a length equal to the largest column dimension, or 1/6 of the clear column height, but not less than 18 inches, regardless of the axial load level.

To restrain against compression buckling of the longitudinal reinforcement, the center to center spacing of the confining reinforcement, as required by the New Zealand NZ3101 Code [25], cannot exceed $6d_b$ where d_b is the longitudinal bar diameter, nor 1/5 of the least sectional dimension or diameter, nor 200 mm. Test results, however, have shown that such spacing requirement does not eliminate buckling of the longitudinal reinforcement but rather enable a compressive strain up to 0.04 or higher to be sustained by the longitudinal bar before excessive lateral displacement due to instability of the bar would occur [26]. The maximum spacing of the transverse steel required by the ACI 318 Code [24], on the other hand, is limited to 4 inches or 1/4 of the minimum member dimension, independent of longitudinal bar diameter.

The effective use of the transverse reinforcement also requires careful detailing of the spirals or hoops. Current usage may entail welding at the lap-splices of the spiral and hoop, or bending back of these bars into the concrete core for anchorage in order to develop full yield capacity. Design practice prefers the use of spirals since fewer anchorages are required for spirals when compared to hoops. The transverse reinforcement in earlier design practice, however, was often anchored with lap-splices in the plastic hinge regions where serious spalling of cover concrete is expected. The loss of cover concrete may initiate unwinding of the spirals or hoops and renders the transverse reinforcement ineffective. The model developed here assumes full development of the transverse steel strength at ultimate condition, and prudent use of the computer program pertaining to lap detailing is advised.

2.2.3 Steel Jacket

The role of a steel jacket for retrofit of bridge columns are considered to be the same as that of the transverse reinforcement. The jacket prevents the spalling of cover concrete and allows the development of large compressive strain in the longitudinal steel without buckling. The shear strength of the column in the encased region is also enhanced (see Chapter 5).

Although the commercially available structural steel for steel jackets has yield strengths ranging from 36 ksi to 50 ksi or higher, the level of confining pressure required for retrofit does not generally require yield strength greater than 36 ksi [1]. A suitable steel for the jacket is the A36 hot-rolled which has relatively low carbon content (from 0.25 to 0.29% depending on the thickness). The low carbon content provides a good welding property which is important for on-site welding of the steel jacket. The average static yield strength of A36 steel, as reported in [27], is 37.1 ksi with a modulus of elasticity averaged around 29500 ksi. The on-set of strain-hardening occurs at strain of 0.020 with a strain-hardening modulus of 450 ksi. The ultimate stress is about 56 ksi occurring at 0.20 strain. Using these properties, the stress-strain curve for A36 steel can be constructed from Eqn. 2.24 to 2.28.

2.2.4 Grout

The analytical model developed here assumes an injection of a cement-based grout into the gap between the steel jacket and column to facilitate composite action. The grout infill provides a certain degree of composite action between the column and jacket depending on the available bond strength at the steel jacket and grout interface, thus increasing the flexural rigidity of the column.

Considerations of the column flexural rigidity are discussed in Chapter 4.

The development of the lateral confining pressure f'_{lj} given in Eqn. 2.7 may be limited by the compressive strength of the infilling grout i.e.

$$f'_{lj} \leq f'_{g} \tag{2.34}$$

Chapter 3

Laminar Analysis

3.1 General

Since the constitutive relations given in Chapter 2 cannot be easily integrated in closed form to give the internal forces for steel and concrete, a numerical approach involving laminar analysis as outlined by King [28] will be used.

The column critical section is divided into two regions for confinement consideration, namely the cover and core concrete. The centerline of the hoop or spiral steel defines the boundary of the two regions. For the program, the critical section is discretized into a total of 100 slices; with 5 slices in the top and bottom cover and 90 slices in the core portion. The discretization is considered adequate for flexural strength and ductility assessment. The discretized column section is shown in Figure 3.1.

3.2 Discretization of Concrete Section

A convenient method for defining the area associated with the discretization process is by use of the area formula for a sector in a circle, shown shaded in Figure 3.2, and is given by:

$$\begin{aligned} A_{sec} &= \frac{D^2}{8}(\theta - \sin\theta) \\ &= \frac{D^2}{4}\cos^{-1}(1 - \frac{2y}{D}) - (\frac{D}{2} - y)\sqrt{D.y - y^2} \end{aligned} \tag{3.1}$$

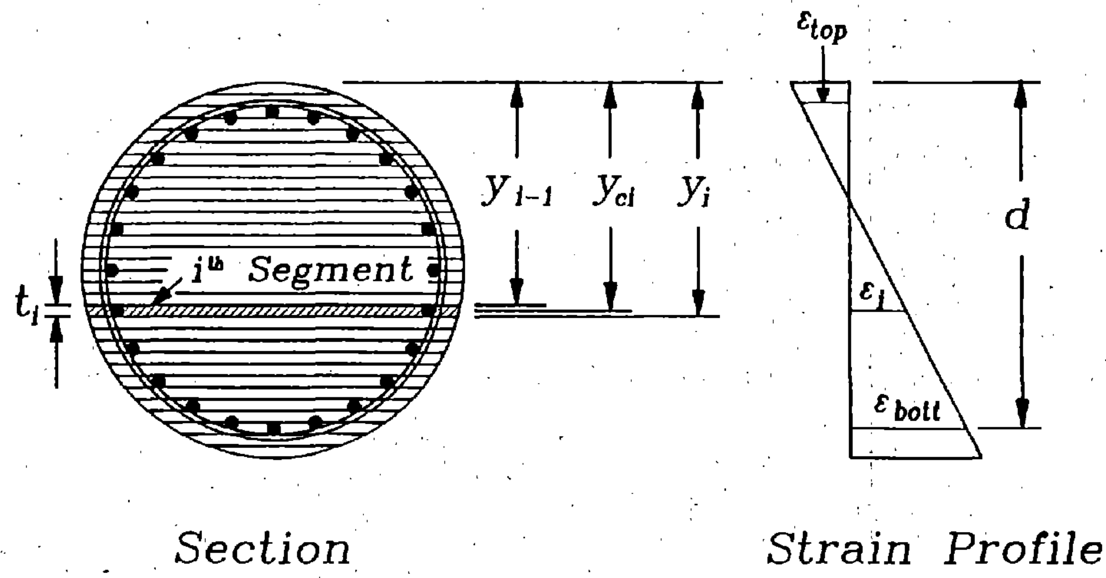


Figure 3.1: **Discretization of Circular Section**

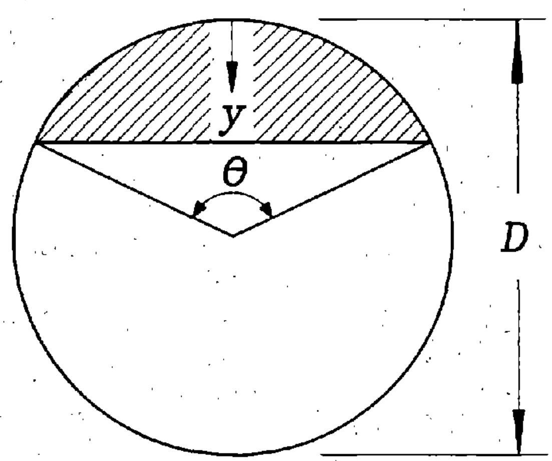


Figure 3.2: **Area for a Sector of Circle**

where D = diameter of column; θ = subtended angle; and y = distance from top edge of column. Note that the diameter of the core concrete d_s is:

$$d_s = D - 2c + d_{sh} \tag{3.2}$$

where c = clear cover measured to longitudinal bar; and d_{sh} = bar diameter of hoop or spiral.

In a similar manner, the sectorial area associated with the core diameter d_s may be written as:

$$A'_{sec} = \frac{(d_s)^2}{4}\cos^{-1}(1 - \frac{2y}{d_s}) - (\frac{d_s}{2} - y)\sqrt{d_s.y - y^2} \tag{3.3}$$

By substituting the distances y_{i-1} and y_i into Eqn. 3.1, the corresponding sectorial area $A_{(sec)_{i-1}}$ and $A_{(sec)_i}$ may be obtained. The difference between the two areas gives the area of the i^{th} slice for the outside diameter (Figure 3.3) i.e.

$$A_{(slice)_i} = A_{(sec)_i} - A_{(sec)_{i-1}} \tag{3.4}$$

Similarly the area of the i^{th} slice for the core diameter d_s is given by:

$$A'_{(slice)_i} = A'_{(sec)_i} - A'_{(sec)_{i-1}} \tag{3.5}$$

Within each slice, the area is further divided into the cover area, the core area and the steel area. In the cover region:

$$A_{(cover)_i} = A_{(slice)_i} \tag{3.6}$$

$$A_{(core)_i} = 0 \tag{3.7}$$

In the core region:

$$A_{(cover)_i} = A_{(slice)_i} - A'_{(slice)_i} \tag{3.8}$$

$$A_{(core)_i} = A'_{(slice)_i} - (A_s)_i \tag{3.9}$$

where $(A_s)_i$ is the steel area assigned to the i^{th} slice defined in the next section.

3.2.1 Longitudinal Steel

The program assumes a uniform distribution of longitudinal bars around the perimeter so that the total steel area A_s can be 'smeared' into a continuous ring of reinforcement formed by a circle passing through the centers of the bars. The steel area assigned to each slice is in proportion to the arc length subtended by that slice (see Figure 3.4) Thus the steel area for the i^{th} slice is:

$$(A_s)_i = A_s(\frac{\theta_i - \theta_{i-1}}{2\pi}) \tag{3.10}$$

For simplicity, the steel area in i^{th} slice $(A_s)_i$ is assumed to act at the center of each slice.

3.3 Section Analysis

3.3.1 Strain Profile

It is assumed that plane section before bending remains plane after bending so that the linear longitudinal strain profile shown in Figure 3.1 may be used to define the deformation of the column critical section. The strain ϵ_i at center of i^{th} slice may be written as:

$$\epsilon_i = \epsilon_{top} - (\epsilon_{top} - \epsilon_{bott})\frac{y_{ci} - c + 0.5d_{sh}}{d - c + 0.5d_{sh}} \tag{3.11}$$

where ϵ_{top} = strain in the outermost fiber of the core (compression is +ve); ϵ_{bott} = strain in the extreme tension steel; d = distance from top cover fiber to extreme steel; and y_{ci} = distance of the center of i^{th} slice from top cover fiber.

From the known strain profile, the longitudinal stresses in both the concrete and steel are computed using constitutive equations developed in Chapter 2.

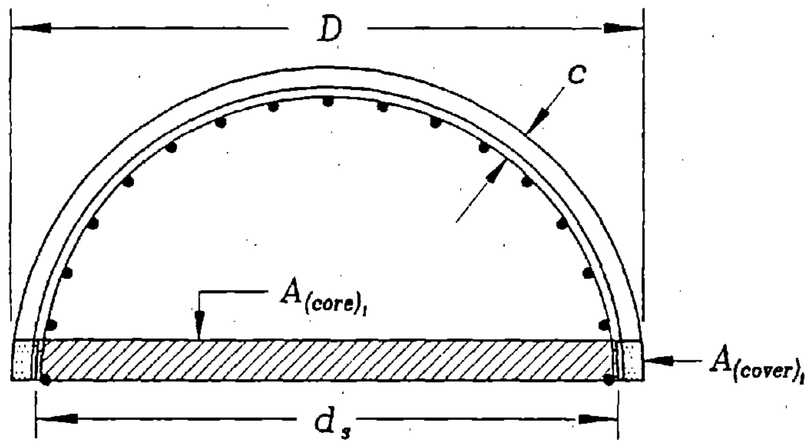


Figure 3.3: **Definition of the i^{th} Slice**

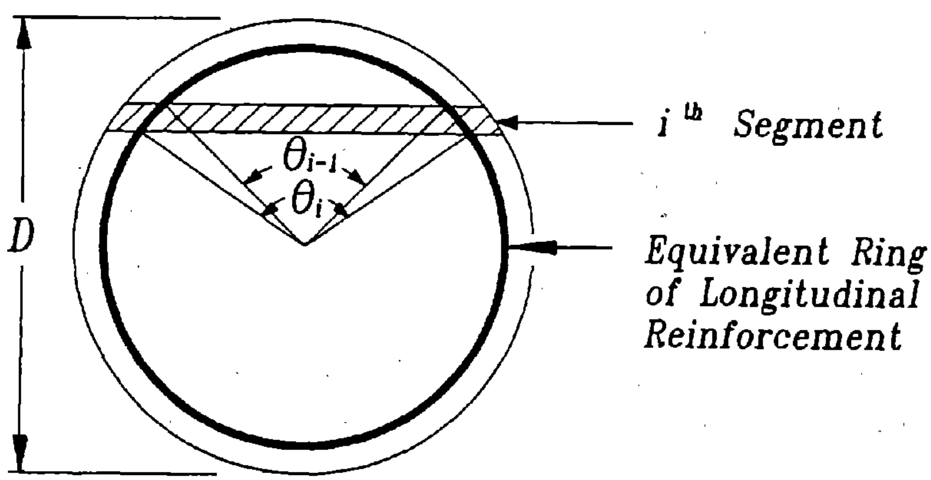


Figure 3.4: **Assignment of Steel Area to Each Slice**

3.3.2 Equilibrium of Internal Forces

For static equilibrium, the sum of forces in concrete and steel at the critical section must equal to the applied axial force. The total concrete force C_c consists of forces in the cover and core concrete, and is given by:

$$C_c = \sum_{i=1}^{N}((f_c)_{cover}A_{cover} + (f_c)_{core}A_{core})_i \tag{3.12}$$

where $(f_c)_{cover}$, $(f_c)_{core}$ = longitudinal stresses in the cover and core concrete, respectively; and N = total number of slices. It is to be re-emphasized that the analysis is carried out for load stages above cracking of the cover concrete and that the tensile stresses in concrete can be ignored. The net steel force is given by:

$$C_s - T_s = \sum_{i=1}^{N}(f_s)_i(A_s)_i \tag{3.13}$$

where C_s, T_s = compressive and tensile steel forces, respectively; $(f_s)_i$ = longitudinal steel stress at the i^{th} slice; and $(A_s)_i$ = steel area assigned to the i^{th} slice.

An iteration procedure is employed by varying the extreme strain value of the concrete core until the following convergence criterion for the equilibrium of internal forces is satisfied:

$$|C_c + C_s - T_s - P| \leq \Delta P \tag{3.14}$$

where P = applied axial force; and ΔP = convergence limit, and is the taken as 0.05% of the balanced axial force P_{bal}. The balanced axial force P_{bal} is defined as the column force, acting in conjunction with a bending moment, required to induce simultaneously an ultimate compression strain in the extreme core fiber and yield strain in the extreme tension steel. The estimation of ultimate compressive strain is discussed further in the next section.

Upon convergence of internal forces, the bending moment about the column centerline is obtained using:

$$M = \sum_{i=1}^{N}((f_c)_{cover}A_{cover} + (f_c)_{core}A_{core} + f_sA_s)_i(\frac{D}{2} - y_{ci}) \qquad (3.15)$$

where y_{ci} is the distance from top fiber to the center of the i^{th} slice.

3.4 Ultimate Concrete Compressive Strains

3.4.1 Previous Research

In limit state design, a realistic estimation of the ductility capacity of a member must be made. Code computation of the flexural strength for reinforced concrete members, for example by ACI [24], assumes an extreme compressive strain of 0.003. Such values lead to very conservative assessment of the ductility capacity for the member and cannot easily satisfy the ductility demand imposed by seismic loading.

Various empirical expressions have been proposed to improve the prediction of the ultimate compressive strain. For example, earlier expressions by Baker [29]:

$$\epsilon_{cu} = 0.0015\left[1 + 150\rho_s + (0.7 - 10\rho_s)\frac{d}{c}\right] \leq 0.01 \qquad (3.16)$$

and Corley [30]:

$$\epsilon_{cu} = 0.003 + 0.02\frac{b}{z} + \left(\frac{\rho_s' f_{yh}}{20}\right)^2 \qquad (3.17)$$

were intended for calculation of the plastic hinge rotation in reinforced concrete beams. The terms in Eqn. 3.16 and 3.17 are ρ_s = ratio of volume of transverse confining reinforcement to volume of concrete core (similar to Eqn. 2.15); ρ_s' = ratio of total volume of transverse plus longitudinal compression reinforcement to volume of concrete core; d = effective depth of beam section; c = neutral axis depth; b = width of beam section, z = distance from critical section to point of contraflexure; and f_{yh} = yield strength of hoop (ksi). These expressions have been shown to be very conservative, especially for well confined columns [31].

Tests on near full-scale square columns by Scott et al [31] suggested that the maximum compressive strain in the core concrete could be taken conservatively as the longitudinal strain at which the first hoop fractures. The concrete core compressive strain at first hoop fracture varied between 0.02 to 0.038 in concentrically loaded columns, and between 0.061 and 0.074 for the eccentrically loaded columns. The larger compressive strain in eccentrically loaded columns indicated strong dependence of ultimate compressive strain on strain gradient across the section. The strain at first hoop fracture was found to increase with increasing volumetric ratio of transverse reinforcement and decrease with higher rates of loading. Based on these results, a less conservative expression was proposed by Scott et al [31], which in U.S. Customary Units, is given by:

$$\epsilon_{cu} = 0.004 + 0.0207\rho_s f_{yh} \tag{3.18}$$

where ρ_s = volumetric ratio of transverse steel to concrete core; and f_{yh} = yield strength of transverse steel. Note that in the limit where no transverse steel is provided, the prediction of ultimate concrete compressive strain is 0.004.

3.4.2 Energy Balance Method

A more rational approach to the prediction of ultimate compressive strain for confined concrete was recently proposed by Mander et al [18] based on an energy balance method. It was suggested that the additional ductility of confined concrete is provided by the strain energy capacity of the transverse reinforcement. For a confined column, the available strain energy of the transverse hoops, U_{sh}, is considered to be equal to the external work done on the column to fracture the hoops, U_g, minus the work done to cause failure of an equivalent column of unconfined concrete, U_{co}, i.e.

$$U_{sh} = U_g - U_{co} \tag{3.19}$$

In the original approach by Mander et al [18], the external work done on the column to fracture the hoops, U_g, was taken to be the sum of the work done on the confined concrete core, U_{cc}, and the longitudinal steel, U_s, i.e.

$$U_g = U_{cc} + U_s \tag{3.20}$$

where U_s, for a unit length of column, is determined by the area beneath the longitudinal steel force-strain curve i.e.

$$U_s = \int_0^{\epsilon_{cu}} A_s f_s d\epsilon \tag{3.21}$$

However, as Tanaka and Park have shown [32], Eqn. 3.21 overestimates the amount of strain energy absorbed by the hoop steel to develop the compressive strain of the longitudinal reinforcement, since it is the energy to prevent buckling of the longitudinal, rather than the energy to fully compress the longitudinal steel that is required for energy balance. For practical range of longitudinal steel ratios, the strain energy required of the hoop steel to prevent buckling the longitudinal steel, U_s, is small compared to the strain energy absorbed by the confined concrete, U_{cc}. It is thus assumed that U_s can be ignored so that Eqn. 3.19 can be approximated by:

$$U_{sh} \approx U_{cc} - U_{co} \tag{3.22}$$

This approximation will be shown to provide a conservative estimate of the ultimate compressive strain. A more refined treatment of the strain energy required to prevent buckling of the longitudinal steel can be found in Tanaka and Park [32].

The additional strain energy absorbed by the confined concrete (per unit core area) is given by the shaded area, A_1, between the stress-strain curves of the unconfined and confined concrete, as shown in Figure 2.1, and may be written as:

$$A_1 = \gamma_1 f'_{cc}(\epsilon_{cu} - \epsilon_{sp}) \tag{3.23}$$

where γ_1 denotes the coefficient of integration; f'_{cc} = confined concrete compressive strength; ϵ_{cu} = ultimate concrete compressive strain; and ϵ_{sp} = spalling strain of the unconfined concrete.

The strain energy density of transverse steel is given by the area, A_2, under the stress-strain curve of transverse steel, shown in Figure 2.4, and may be written as:

$$A_2 = \gamma_2 f_{yh} \epsilon_{su} \tag{3.24}$$

where γ_2 is the coefficient of integration; f_{yh} = yield strength of transverse steel; and ϵ_{su} = ultimate tensile strain of transverse steel. It should be noted that the strain energy is only calculated up to the ultimate strain and does not include the energy reserve beyond ultimate stress to final fracture. The approach would lead to a lower bound value for the ultimate compressive strain of confined concrete.

The balance of strain energies between the core concrete and transverse steel (Eqn. 3.22) requires:

$$\gamma_2 f_{yh} \epsilon_{su} A_{sh} \pi d_s = \gamma_1 f'_{cc}(\epsilon_{cu} - \epsilon_{sp})(\frac{\pi}{4} d_s^2 - A_s) k_e s \tag{3.25}$$

where k_e is the confinement effectiveness coefficient; A_s = longitudinal steel area. Eqn. 3.25 can be rearranged into:

$$\epsilon_{cu} = \epsilon_{sp} + \frac{\gamma_2}{\gamma_1} \frac{f_{yh}}{f'_{cc}} \frac{\rho_s \epsilon_{su}}{(1 - \rho_{cc}) k_e} \tag{3.26}$$

where ρ_s and ρ_{cc} have been defined earlier (Eqn. 2.15 and 2.12).

Since the longitudinal steel content is generally less than 5% for a typical bridge column, and the confinement effectiveness coefficient k_e is close to unity, it would be acceptable, for design or assessment purposes, to further simplify Eqn. 3.26 into:

$$\epsilon_{cu} = \epsilon_{sp} + \rho_s \epsilon_{su} \frac{\gamma_2}{\gamma_1} \frac{f_{yh}}{f'_{cc}} \tag{3.27}$$

Note that the above simplification compensates for the assumption of no strain energy being absorbed by the longitudinal steel.

Numerical integrations were carried out to determine the values of the integration coefficients, γ_1 and γ_1, for both Grade 40 and 60 transverse steels. The integrations were carried out using the stress-strain curves presented in Section 2.1 and 2.2 for the concrete and reinforcing steel. For Grade 40 transverse steel, strain-hardening was assumed to occur at a strain of $\epsilon_{sh} = 0.0193$, while the ultimate strain was taken to be $\epsilon_{su} = 0.159$. For Grade 60 steel, a shorter yield plateau of $\epsilon_{sh} = 0.0103$ and smaller ultimate strain of $\epsilon_{su} = 0.12$ were assumed. The ratio of ultimate tensile strength to yield strength of transverse steel was taken as $f_{su}/f_y = 1.5$. Even though a large variation of steel properties is expected in practice, these steel properties were considered representative and would lead to a reasonable, if not conservative, estimate of the ultimate compressive strains. The range of concrete compressive strength investigated in this study were 4, 5 and 6 ksi.

The variation of integration coefficient γ_1 with confining steel ratio ρ_s is shown in Figure 3.5. It can be seen from Figure 3.5 that the value of γ_1 rises rapidly for small value of ρ_s. For a confining steel ratio ρ_s of less than 0.001, the influence of confinement is small and the term $(\epsilon_{cu} - \epsilon_{sp})$ in Eqn. 3.23 approaches zero. But because the stress-strain curve for unconfined concrete has been assumed to be linear from $2\epsilon'_{co}$ to ϵ_{sp} (see Section 2.1.2), there exists a non-zero area between the curves of the confined and unconfined concrete, and the integration coefficient γ_1 must increase in order to maintain the finite area between the two stress-strain curves as $(\epsilon_{cu} - \epsilon_{sp})$ approaches zero. It can also be seen from Figure 3.5 that the value of γ_1 is larger for Grade 60 transverse steel than for Grade 40 transverse steel, and decreases with increased concrete compressive strength. This can be expected since a greater confining pressure is available from a higher grade of transverse steel, and the integration coefficient γ_1 is essentially inversely proportional to f'_c. Numerical integration gave the same value of the integration coefficient, $\gamma_2 = 1.35$, for both grades of steel.

The variation of the ratio of integration coefficient, γ_2/γ_1, with the transverse steel ratio ρ_s is plotted in Figure 3.6(a) and (b). It can be seen that for both grades of steel, the ratio γ_2/γ_1 rises rapidly for small transverse steel ratio ρ_s, but levels off to values of about 1.4 and 1.35 for Grade 40 and 60 steel, respectively. The Grade 40 steel however shows a larger variation of γ_2/γ_1 ratio with concrete

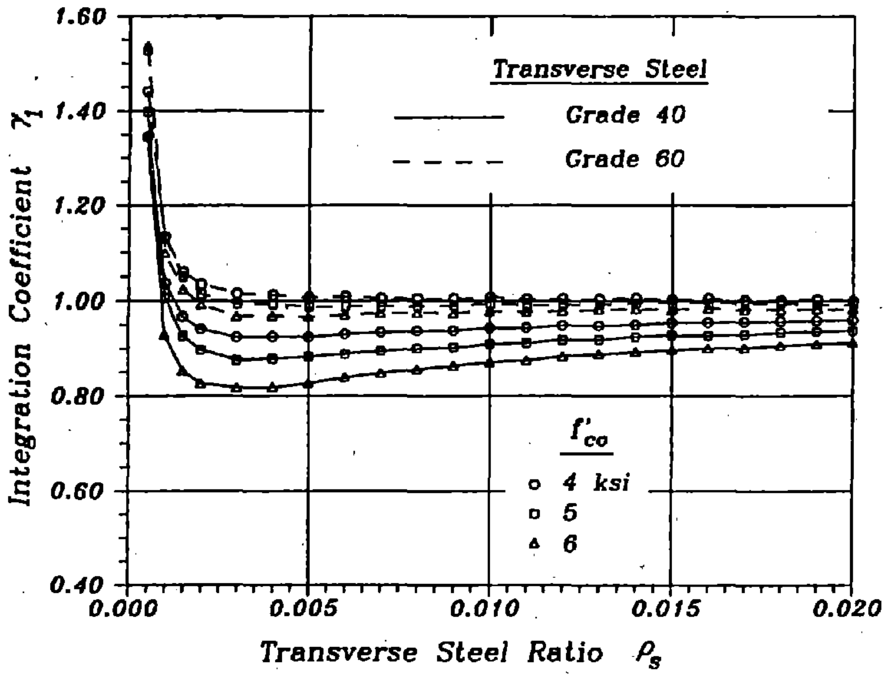


Figure 3.5: **Variation of Integration Coefficient γ_1 with Confining Steel Ratio ρ_s**

compressive strength when compared to Grade 60 steel. For limit state design, a conservative estimate of the γ_2/γ_1 ratio may be desirable, and it is thus proposed that the integration coefficient ratio be given by:

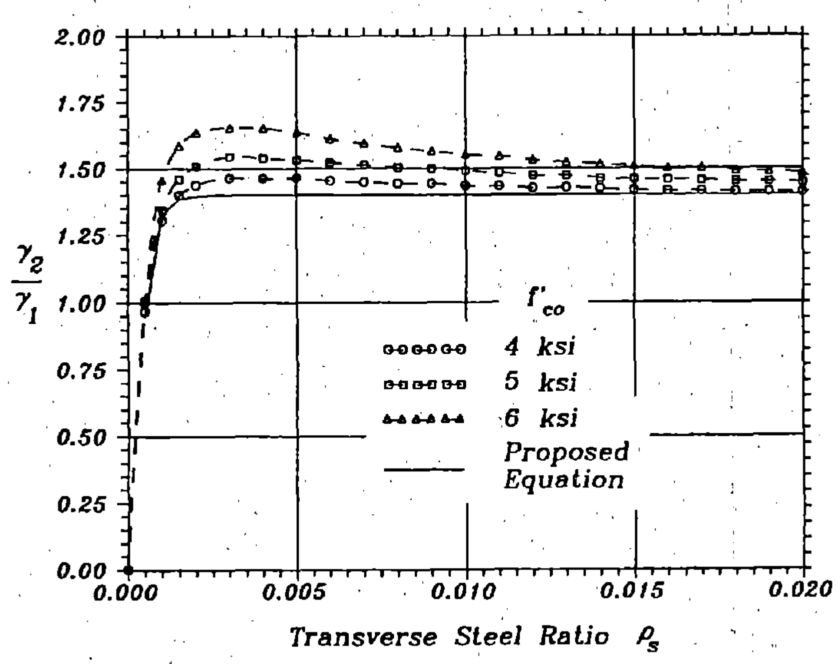
For Grade 40 steel,

$$\frac{\gamma_2}{\gamma_1} = \frac{2000\rho_s}{\left(1 + (1428\rho_s)^4\right)^{0.25}} \tag{3.28}$$

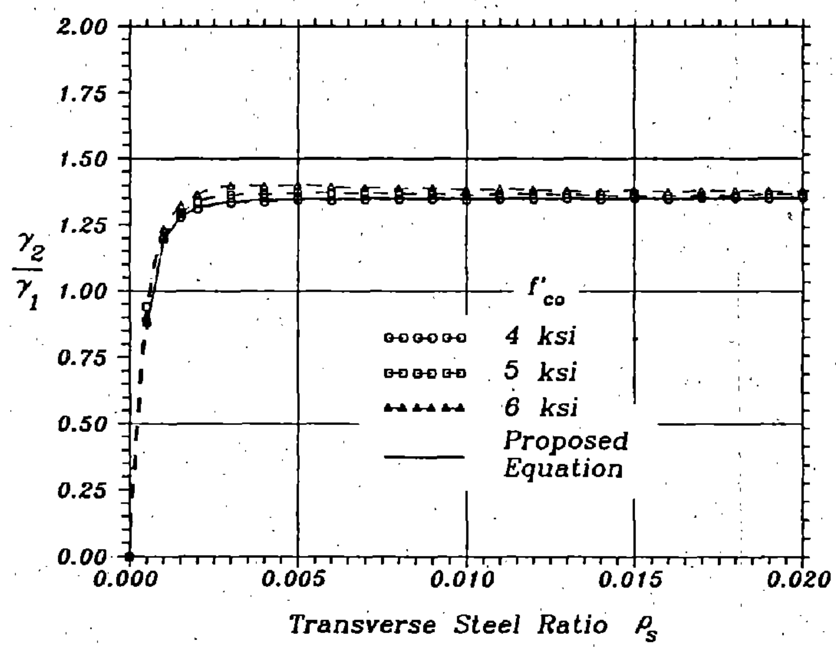
For Grade 60 steel,

$$\frac{\gamma_2}{\gamma_1} = \frac{2000\rho_s}{\left(1 + (1480\rho_s)^{2.5}\right)^{0.4}} \tag{3.29}$$

The two proposed equations are superimposed as solid lines in Figure 3.6(a) and (b). It should be re-emphasized that the proposed equations for γ_2/γ_1 ratio are only applicable for Eqn. 3.27, since the reduction of concrete core area by the longitudinal steel has been ignored and the confinement effectiveness coefficient k_e has been assumed to be unity.



(a) Grade 40 Transverse Steel



(b) Grade 60 Transverse Steel

Figure 3.6: Variation of Integration Coefficient Ratios

Mander et al [19] conducted concentric load tests on circular columns of 19.7 inches in diameter by 59.1 inches in height to determine the response of axially loaded columns confined by circular spirals. A relative high loading rate of 0.013 strain per second was used. The transverse steel content for the columns, ρ_s, was between 0.6% and 2.5%, and longitudinal steel content, ρ_t, based on gross sectional area, was between 1.23% and 3.69%. The axial compressive strains of the columns were measured by the linear potentiometers over a gage length of 17.7 inches in the central region. The energy balance method was shown to provide good prediction of the average axial compressive strain at first hoop fracture. In calculating the strain energy capacity of the transverse reinforcement, however, Mander et al [19] included the energy up to the fracture strain, assumed to be 8% above the ultimate tensile strain of the transverse steel. The approach may overestimate the energy capacity of the transverse hoop, as hoop fracture occurs in a localized region at a stress level below the ultimate strength of the transverse steel and is accompanied by stress-relief in other part of the steel.

Table 3.1 summaries the experimental ultimate compressive strains as reported by Mander et al [19] and the theoretical prediction of the ultimate compressive strain by Eqn. 3.27. Implicit in Eqn. 3.27 is the confinement effectiveness coefficient $k_e = 1$, whereas the actual k_e factor for the test columns varied between 0.890 and 1.002. The spalling strains of unconfined concrete ϵ_{sp} shown in Table 3.1 are those obtained experimentally for a corresponding set plain concrete cylinders by Mander et al [19].

Figure 3.7(a) shows the ratio of experimental to predicted ultimate compressive strain given in Table 3.1 as a function of the confining steel ratios for the test columns. It can be seen from Figure 3.7(a) that the proposed equa-

Table 3.1: **Prediction of Ultimate Compressive Strains**

- Based on column tests by Mander et al (Reference 19)

Unit	ϵ_{sp}	f_{yh} (MPa)	f'_{cc} (MPa)	ρ_s	ϵ_{su}	γ_2/γ_1	ϵ_{cu} Theory	ϵ_{cu} Experiment
a	0.0040	310	40.3	0.020	0.18	1.4	0.0427	0.060
b	0.0040	340	48.3	0.020	0.18	1.4	0.0395	0.039
c	0.0055	340	50.5	0.020	0.18	1.4	0.0394	0.058
1	0.0080	340	51.0	0.025	0.18	1.4	0.0500	0.058
2	0.0080	340	43.0	0.015	0.18	1.4	0.0379	0.056
3	0.0080	340	38.5	0.010	0.18	1.4	0.0303	0.055
4	0.0080	320	34.5	0.006	0.14	1.4	0.0189	0.035
5	0.0080	320	46.5	0.020	0.14	1.4	0.0350	0.058
6	0.0080	307	45.1	0.020	0.14	1.4	0.0347	0.057
7	0.0045	340	50.8	0.020	0.18	1.4	0.0382	0.060
8	0.0045	340	48.6	0.020	0.18	1.4	0.0398	0.057
9	0.0045	340	50.8	0.020	0.18	1.4	0.0382	0.060
10	0.0045	340	48.5	0.020	0.18	1.4	0.0398	0.058
11	0.0045	340	48.8	0.020	0.18	1.4	0.0396	0.043
12	0.0045	340	50.7	0.020	0.18	1.4	0.0383	0.043

tion (Eqn. 3.27) gives reasonable prediction of the ultimate compressive strain even though conservatism is pronounced for column with low confining steel ratio, possibly due to longitudinal strain gradient invariably present at the section when the transverse steel fractures in a localized region. As will be seen in the next section, the influence of strain gradient on the ultimate compressive strain is significant for columns with low confining steel ratios.

The same set of test data is presented in Figure 3.7(b) with the ultimate compressive strains being plotted directly on the y-axis. The prediction of ultimate compressive strain in this case is based on the following properties for the columns: unconfined compressive strength of concrete $f'_{co} = 4350$ psi; spalling strain of unconfined concrete $\epsilon_{sp} = 0.005$; yield strength of transverse steel $f_{yh} = 49.3$ ksi; and ultimate tensile strain of transverse steel $\epsilon_{su} = 0.18$.

The prediction of ultimate compressive strains by Scott et al (Eqn. 3.18) is also included in Figure 3.7(b) [31] for comparison. It can be seen that, even though the proposed equation (Eqn. 3.27) is conservative, it gives a better prediction of the ultimate concrete compressive strains when compared to Eqn. 3.18 proposed by Scott et al [31].

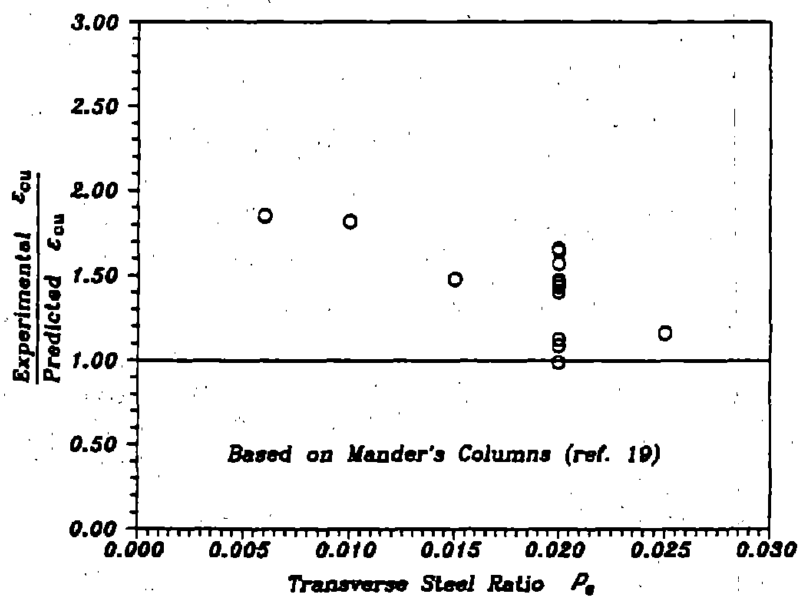
3.4.3 Influence of Strain Gradient

In the eccentric column tests by Scott et al [31], a two to three fold increase of the ultimate compressive strains were observed at first hoop fracture. The test columns were subjected to a strain gradient of having the neutral axis near one face of the column.

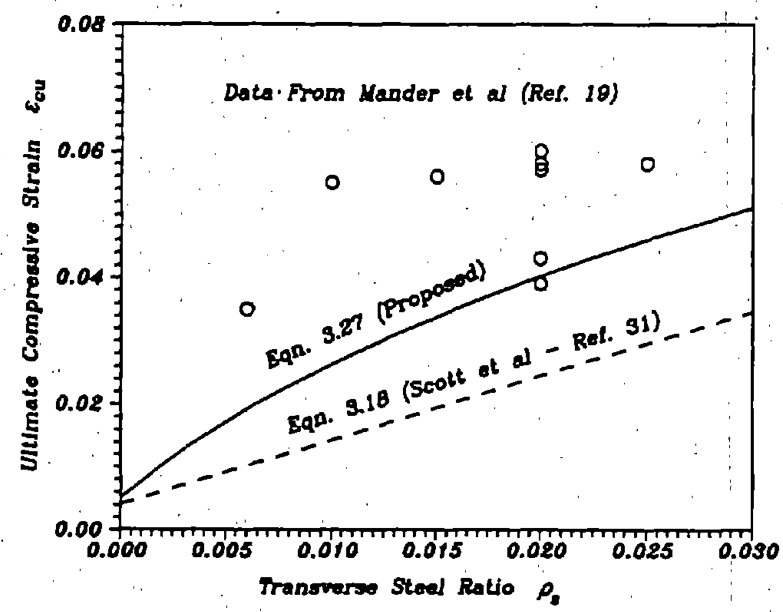
The influence of strain gradient on the ultimate compressive strain, ϵ_{cu}, can be studied using the energy balance method but carried out on discretized core concrete, as shown in Figure 3.8. The concrete core, as characterized by the diameter, d_s, is divided into a finite number of slices and prescribed with a linear longitudinal strain profile across the section. For the i^{th} slice, the strain energy required to change the concrete from an unconfined to confined state is:

$$\mathcal{E}_{ci} = \left[\int_0^{\epsilon_{ci}} (f_{cc} - f_c) d\epsilon\right] (A_c)_i \qquad (3.30)$$

where ϵ_{ci} = longitudinal strain at mid-point of the i^{th} slice; $(A_c)_i$ = net concrete area for i^{th} slice; and f_{cc}, f_c = confined and unconfined concrete compressive stresses, respectively. The net concrete area for each slice is taken as the area of each slice minus the assigned longitudinal steel area for that slice. The longitudinal steel is 'smeared' into an equivalent continuous ring of reinforcement, and assigned to each slice in proportion to the arc length contained in the slice,



(a) Prediction of Ultimate Compressive Strain



(b) Comparisons of Proposed Equations

Figure 3.7: Ultimate Compressive Strains of Confined Concrete

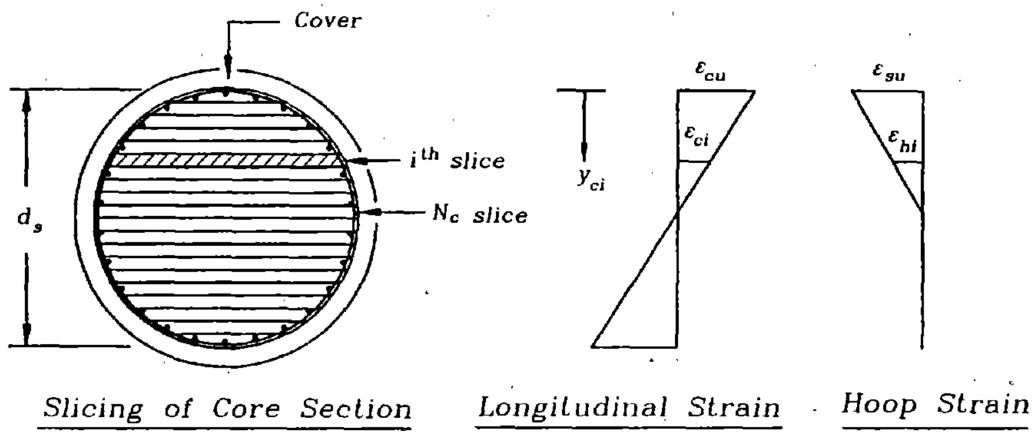


Figure 3.8: **Laminar Analysis for Ultimate Compressive Strain**

similar to that discussed earlier in Section 3.2. The longitudinal steel is assumed, as before, to absorb no strain energy.

The strain energy available from the transverse steel in the i^{th} slice is given by:

$$\mathcal{E}_{hi} = \left[\int_0^{\epsilon_{hi}} (f_s) d\epsilon\right] (v_h)_i \tag{3.31}$$

where ϵ_{hi} = tension strain in transverse steel for i^{th} slice; f_s = transverse steel stress; and $(v_h)_i$ = transverse steel volume assigned to i^{th} slice, similar to that adopted for longitudinal steel, i.e., proportional to the arc length contained in that slice. The distribution of tensile strain in the transverse steel is assumed to be linear with ϵ_{su} at the extreme fiber, and decreases to zero at the location of the neutral axis (Figure 3.8).

Assuming a confinement effectiveness coefficient of $k_e = 1$, the balance of energies between concrete and transverse steel requires:

$$s \sum_{i=1}^{N_c} \mathcal{E}_{ci} = \sum_{i=1}^{N_c} \mathcal{E}_{hi} \tag{3.32}$$

where N_c = number of compressive slices; and s = transverse steel spacing. A numerical procedure is carried out so that, for a given dimensionless curvature i.e. curvature times core diameter (ϕd_s), the value of ϵ_{cu} is iterated until the balanced condition in Eqn. 3.33 is satisfied within a prescribed tolerance limit. A convergence of solution is assumed if the inequality below is satisfied:

$$\left|\frac{s \sum_{i=1}^{N_c} \mathcal{E}_{ci}}{\sum_{i=1}^{N_c} \mathcal{E}_{hi}} - 1\right| \leq 0.02 \tag{3.33}$$

The influence of strain gradient on the ultimate compressive strain of concrete ϵ_{cu} was studied using Grade 40 steel for the transverse reinforcement, with the following properties: strain-hardening strain ϵ_{sh} = 0.0193; ultimate strain ϵ_{su} = 0.159; yield strength f_{yh} = 40 ksi and ultimate strength $f_{su} = 1.5 f_{yh}$ = 60 ksi. The concrete strength chosen was f'_{co} = 5000 psi. Figure 3.9 shows the influence of strain gradient on the ultimate compressive strain for the case of ρ_{cc} = 2% longitudinal steel, where ρ_{cc} is defined in Eqn. 2.12.

The presence of strain gradient causes a significant increase in the ultimate compressive strain, as can be seen in Figure 2.9. For a given dimensionless curvature, the increase of ultimate compressive strain is large for small transverse steel ratios. It can be seen that ϵ_{cu} may double for columns with small volumetric transverse steel ratio. For example, consider a column of 60 inches core diameter and reinforced with transverse steel of ρ_s = 0.2% and longitudinal steel ratio ρ_{cc} = 2%. For an axial compression of $0.05 f'_{co} A_g$, the ultimate curvature to be expected is 0.00119 rad/in, with a corresponding ultimate compressive

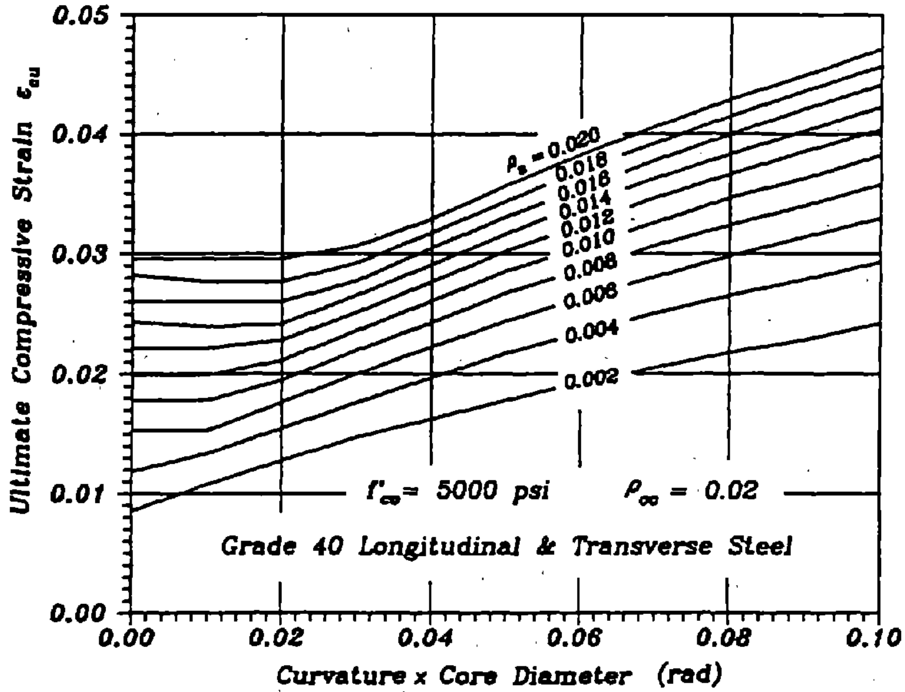


Figure 3.9: **Influence of Strain Gradient on Ultimate Compressive Strain**

strain of 0.0205. The prediction of ϵ_{cu} without strain gradient is however only about 0.0084. Thus the presence of strain gradient caused an increase of 2.44 times in ultimate compressive strain.

3.4.4 Confinement by Steel Jackets

The use of a steel jacket for column retrofit inhibits early spalling of the cover concrete, and enhances the ultimate concrete compressive strain ϵ_{cu} in the manner similar to that of the confinement provided by internal transverse reinforcement to the core concrete. The ultimate compressive strain ϵ_{cu} may be estimated from the balance of strain energies between the steel jacket and the cover concrete which changes from an unconfined to a confined state. For simplicity, let us consider the enhancement of ultimate compressive strain in a column of concrete encased by a steel jacket. Rewriting the area A_2 under the stress-strain curve for the steel jacket as:

$$A_2 = \gamma_2 f_{yj} \epsilon_{suj} \tag{3.34}$$

where γ_2 is the coefficient of integration as before, and f_{yj}, ϵ_{suj} are now the yield stress and ultimate strain for the steel jacket, respectively, the balance of strain energies for a column of concrete encased by steel jacket requires:

$$\gamma_1 f'_{cc}(\epsilon_{cu} - \epsilon_{sp})\frac{\pi}{4}(D_j - 2t_j)^2 = \gamma_2 f_{yj}\epsilon_{suj}(D_j - t_j)t_j\pi \tag{3.35}$$

which reduces to

$$\epsilon_{cu} = \epsilon_{sp} + \frac{\gamma_2}{\gamma_1}\frac{f_{yj}}{f'_{cc}}\epsilon_{suj}\frac{4t_j(D_j - t_j)}{(D_j - 2t_j)^2} \tag{3.36}$$

For practical application, since the thickness of the jacket will be small compared to the diameter i.e. $t_j \ll D_j$, we can write:

$$\frac{4t_j(D_j - t_j)}{(D_j - 2t_j)^2} \approx \frac{4t_j}{D_j - 2t_j} = \rho_{sj} \tag{3.37}$$

where ρ_{sj} denotes the confining ratio of steel jacket defined in Eqn. 2.8.

Thus the limiting concrete compressive strain ϵ_{cu} for concrete confined by steel jacket may be written as:

$$\epsilon_{cu} = \epsilon_{sp} + \frac{\gamma_2}{\gamma_1}\frac{f_{yj}}{f'_{cc}}\epsilon_{suj}\rho_{sj} \tag{3.38}$$

The program assumes that the stress-strain curve for steel jacket resembles that of Grade 40 reinforcing steel so that the ratio of γ_2/γ_1 may be obtained from Eqn. 3.28.

It should be noted that the increase in ultimate concrete compressive strain ϵ_{cu}, as a result of confinement from the steel jacket, causes a corresponding increase in the extreme tension steel strain. A possible limit state then exists in which the behavior of retrofitted column may be governed by fracturing of the longitudinal steel. Large inelastic load reversals can cause serious reduction of the fracturing strain; a phenomenon associated with low-cycle fatigue of metal. The reduction in fracturing strain results in a smaller cyclic displacement ductility factor, and is discussed further in Section 4.3.2.2.

Chapter 4

Column Deformation

4.1 General

The approximate prediction of column flexural response used by COLRET is outlined in this chapter. The lateral displacement at the center of seismic mass is obtained from a moment-curvature analysis carried out at the critical section of the column. Instead of establishing a distribution of curvatures up the column and then integrating that curvature with respect to column height to obtain the lateral displacement, the simple approach of using an equivalent plastic hinge length developed in [33] is adopted. The same approach is extended to retrofitted column. Shear deformation is ignored in this formulation.

4.2 Moment-Curvature Relations

Typical behavior of bridge columns under lateral loading is characterized by a near-linear elastic response up to first yield of the extreme reinforcement at the critical section of the column. The moment and curvature at this stage of the loading are termed as the yield moment M_y and the yield curvature ϕ'_y, respectively. Further lateral loading on the column causes a spread of steel yielding over a larger area of the cross-section and decreases the slope of the moment-curvature curve until a peak moment is achieved. A descending branch then follows in the post-peak region until an ultimate curvature ϕ_u is reached. The ultimate curvature ϕ_u is defined by the extreme compression fiber of the concrete core reaching an ultimate concrete compressive strain ϵ_{cu}. The moment corresponding to ϕ_u is termed the ultimate moment M_u.

In the case of retrofitted columns, the ultimate compression strain ϵ_{cu} reached by extreme concrete fiber is significantly increased by the confining action of the encasing steel jacket. The initial elastic response of the retrofitted columns however resembles that of the 'as-built' columns since a minimal lateral expansion of the column concrete occurs at the early stages and the steel jacket essentially remains inactive. At higher lateral loads, a significant lateral expansion of concrete occurs, inducing hoop tension in the steel jacket which reacted by providing lateral pressure to the expanding concrete. The spalling of cover concrete is then delayed until a much larger concrete strain is developed. The increase in concrete compression strain causes a corresponding increase in the curvature ductility and may produce a positive moment-curvature gradient as the longitudinal steel reaches strain-hardening. The ultimate curvature may be limited by the extreme tension steel reaching the ultimate tensile strain ϵ_{su}, as pointed out in Section 3.4.4. Typical moment-curvature relations for 'as-built' and retrofitted columns are shown in Figure 4.1.

4.3 Lateral Displacement

4.3.1 'As-built' Columns

The computer program COLRET assumes an elasto-plastic approximation for the actual moment-curvature relation at the critical section. For the 'as-built' columns, the elasto-plastic yield curvature is approximated by:

$$\phi_y = \phi'_y \frac{M_u}{M_y} \tag{4.1}$$

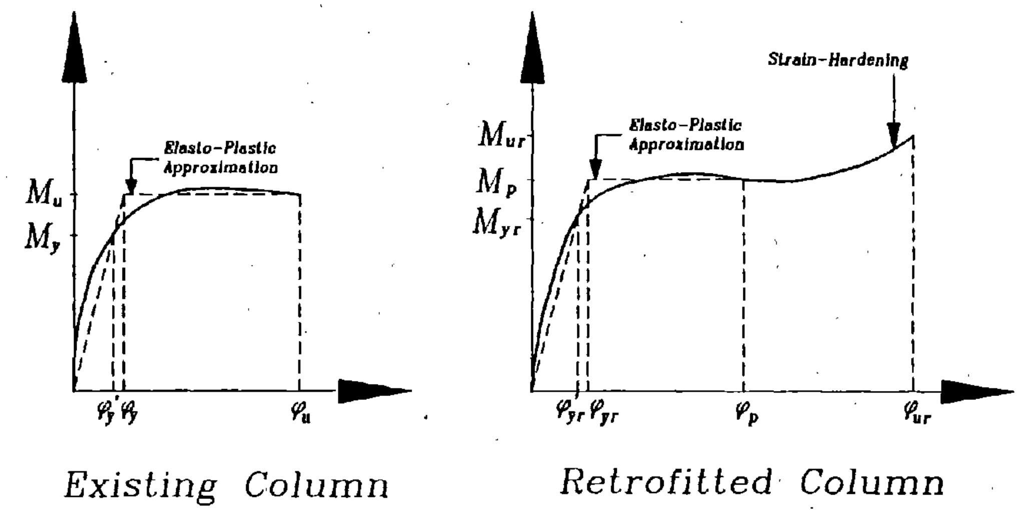


Figure 4.1: **Moment-Curvature Relations**

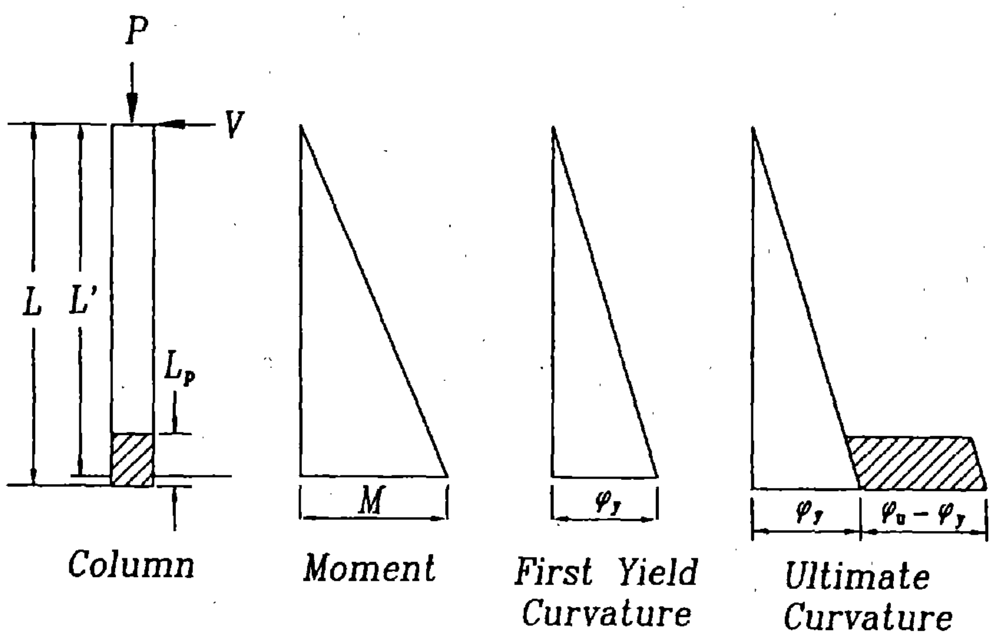


Figure 4.2: **Lateral Displacement of 'As-built' Column**

where M_y and M_u = first yield moment and ultimate moment of 'as-built' column, respectively; and ϕ'_y = curvature at first yield of the longitudinal reinforcement. A linear curvature distribution is assumed for the column up to the equivalent elasto-plastic yield curvature, ϕ_y, as shown in Figure 4.2. The first moment of the curvature distribution about the top of the column gives the equivalent elasto-plastic yield displacement i.e.

$$\Delta_y = \phi_y \frac{(L')^2}{3} \tag{4.2}$$

where L' = original height of the column; and ϕ_y= elasto-plastic yield curvature at the column base.

It should be recognized that the assumption of full base fixity in a vertical cantilever does not prevail in actual columns. The lateral displacement of a column is increased by strain penetration of longitudinal bars into the footing. To account for the increase in lateral displacement, the effective height of the column is increased by 6 times the longitudinal bar diameter i.e.

$$L = L' + 6d_b \tag{4.3}$$

where L, L' = effective and original height of the column, respectively, and d_b is the longitudinal bar diameter. The increase of column height is only effected for the prediction of ultimate deflection in the program.

The lateral displacement beyond first yield of the column is assumed to occur by a plastic rotation over an equivalent plastic hinge length L_p. Assuming that the plastic rotation θ_p is concentrated at the center of the plastic hinge, it has been shown [33] that the lateral displacement at ultimate curvature ϕ_u is given by:

$$\Delta_u = \Delta_y + \theta_p(L - 0.5L_p) \tag{4.4}$$

where θ_p = plastic rotation = $(\phi_u - \phi_y)L_p$.

The displacement ductility factor μ is defined as the ratio of the ultimate displacement to the equivalent elasto-plastic yield displacement i.e.

$$\mu \equiv \frac{\Delta_u}{\Delta_y} \tag{4.5}$$

Combining Eqn. 4.2, 4.4 and 4.5 gives:

$$\mu = 1 + 3\left(\frac{\phi_u}{\phi_y} - 1\right)\frac{L_p}{L'}\left(\frac{L}{L'} - 0.5\frac{L_p}{L'}\right) \tag{4.6}$$

The equivalent plastic hinge length adopted by COLRET follows that proposed in [33]:

$$L_p = 0.08L' + 6d_b \tag{4.7}$$

where d_b denotes the longitudinal bar diameter. The second term on the right-hand side of the equation signifies the increase in effective plastic hinge length with strain penetration into the footing, which is assumed to be proportional to the bar diameter.

4.3.2 Retrofitted Columns

4.3.2.1 Yield Displacement

In addition to enhancing the strength and ductility of the column, the presence of the steel jacket also increases the flexural stiffness of the column. In the encased region of the column, full composite action of the steel jacket cannot be realized until a finite bond transfer length is developed. Depending on the length of the jacket and the bond strength between the grout and jacket, two possibilities arises: case (i) involves a long jacket where sufficient length exists for bond transfer to effect a region of full composite action, and case (ii) involves a relatively short length of jacket in which full composite action cannot be developed. The two cases are identified in Figure 4.3.

The lateral displacement of a retrofitted column at first yield of the extreme tension steel may be estimated by assuming a suitable variation of the flexural rigidities up the column. At the base of the retrofitted column where a vertical gap is provided, an equivalent confined flexural rigidity of the column can be defined as:

$$(EI)_b \equiv \frac{M_{yr}}{\phi'_{yr}} \tag{4.8}$$

where M_{yr} and ϕ'_{yr} represent the moment and curvature, respectively, at first yield of the extreme tension reinforcement, and are assessed by sectional analysis carried out at the base section.

If the steel jacket is considered to be extended to the base of the column and full composite action assumed at that level, an upper-bound composite flexural rigidity $(EI)_c$ can be defined by a sectional analysis carried out to satisfy the equilibrium condition imposed by the external axial load, P, and retrofit yield moment, M_{yr}, i.e.

$$(EI)_c \equiv \frac{M_{yr}}{\phi_c} \tag{4.9}$$

where ϕ_c denotes the curvature for a fully composite section. The fully composite condition requires strain compatibility between the steel jacket, grout infill, column concrete and reinforcing steel. The difference between $(EI)_c$ and $(EI)_b$ thus represents the maximum stiffening effect of the steel jacket and grout infill on the column.

If full composite action exists over the entire length of the jacket, the vertical stresses in the jacket will vary in a near-linear manner since a linear

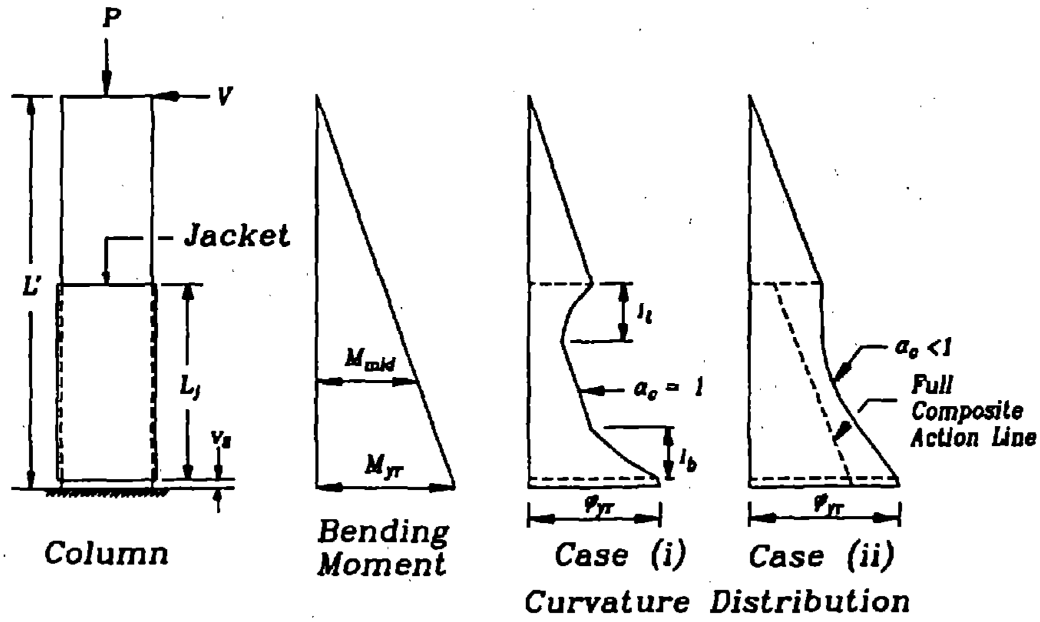


Figure 4.3: **Stiffening of Column by Steel Jacket**

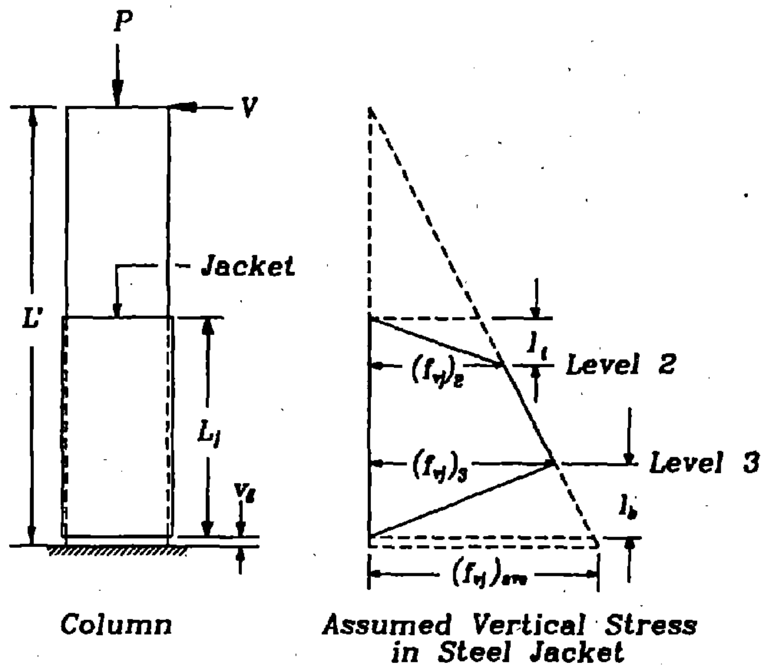


Figure 4.4: **Bond Transfer Lengths For Steel Jacket**

distribution of bending moment is imposed on the column. The linear stress variation will be known since the extreme vertical jacket stresses are calculated from a full composite sectional analysis carried out at the base level to determine $(EI)_c$. In reality, the vertical stresses in the jacket corresponding to full composite action cannot be effected until a finite bond transfer length is developed, resulting in a non-linear variation. The bond transfer lengths are however different for the compression and tension regions of the column. On the flexural compression side, lateral expansion of the concrete is significant especially at high axial strain. The expansion of concrete increases the normal contact stress which in turn increases the bond strength. On the other hand, lesser expansion of the concrete occurs on the tension side requiring a larger bond transfer length. The approach adopted here computes an average value for the absolute magnitude of the compressive and tensile vertical stresses and uses a linear variation of the average value up the jacket, as shown in Figure 4.4. The average jacket vertical stress at the base, assuming an extension of the jacket to top of footing and full composite action at that level, may be written as:

$$(f_{vj})_{ave} = \frac{|(f_{vj})_t| + |(f_{vj})_c|}{2} \tag{4.10}$$

where $(f_{vj})_t$ and $(f_{vj})_c$ are the vertical stresses in the extreme tension and compression generators of the jacket. In case (i) where the jacket length is long enough for a full composite action to occur, the vertical stresses at levels 2 and 3 (see Figure 4.4) may be written in the form:

$$(f_{vj})_2 = \frac{(L' - v_g - L_j + l_t)}{L'}(f_{vj})_{ave} \tag{4.11}$$

$$(f_{vj})_3 = \frac{(L' - v_g - l_b)}{L'}(f_{vj})_{ave} \tag{4.12}$$

where L' = original height of the column; v_g = vertical gap provided between jacket and top of footing; L_j = length of steel jacket; l_t, l_b = bond transfer lengths from top and bottom of jacket, respectively.

If constant bond stress exists at the grout and jacket interface, the vertical jacket stresses developed by bond transfer at levels 2 and 3 may be written as:

$$(f_{vj})_2 = \frac{\bar{u}_o l_t}{t_j} \tag{4.13}$$

and

$$(f_{vj})_3 = \frac{\bar{u}_o l_b}{t_j} \tag{4.14}$$

where $\bar{u}_o$ = average bond strength between grout and steel jacket; t_j = thickness of the steel jacket.

Equating Eqns. 4.11 to 4.13 and 4.12 to 4.14 gives the top and bottom bond transfer lengths:

$$l_t = \frac{(L' - v_g - L_j)t_j}{(\bar{u}_o L' - (f_{vj})_{ave} t_j)}(f_{vj})_{ave} \tag{4.15}$$

and

$$l_b = \frac{(L' - v_g)t_j}{(\bar{u}_o L' + (f_{vj})_{ave} t_j)}(f_{vj})_{ave} \tag{4.16}$$

The distribution of flexural rigidities in the column is assumed to vary linearly within the bond transfer regions. Thus case (i) assumes the form of a trapezoid having $(EI)_b$ at the free ends of the jacket and attaining $(EI)_c$ after developing the transfer lengths of l_b and l_t. A constant rigidity of $(EI)_b$ is used in the uncased region of the column. The distribution of column flexural rigidities for case (i) is shown in Figure 4.5.

In case (ii) where the steel jacket is of insufficient length for full bond transfer, the sum of the computed bond lengths would exceed the jacket length

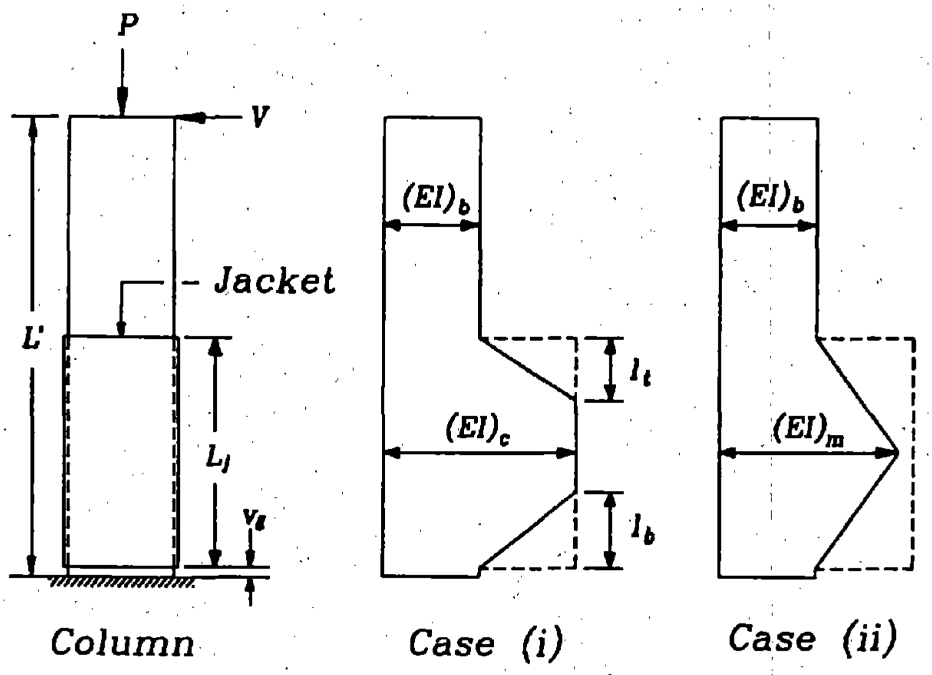


Figure 4.5: **Distribution of Column Flexural Rigidities**

i.e. $l_t + l_b > L_j$. This condition prevents the fully composite flexural rigidity $(EI)_c$ from developing even at mid-height of the jacket. It is proposed herein that the flexural rigidity at mid-height of the jacket be given by:

$$(EI)_m \equiv (EI)_b + \alpha_c((EI)_c - (EI)_b) \tag{4.17}$$

where $0 \leq \alpha_c \leq 1$. The degree of participation by the steel jacket and grout ring in stiffening up the column is accounted for by the composite action coefficient α_c. A suitable definition of α_c is given by:

$$\alpha_c \equiv \frac{(f_{vj})_m}{(\bar{f}_{vj})_m} \tag{4.18}$$

where $(f_{vj})_m$ represents the average vertical stress that is developed by bond at mid-height of jacket and $(\bar{f}_{vj})_m$ represents the average vertical stress at the same level of jacket if full composite action could occur. It is noted that the case where $\alpha_c = 1$ corresponds to full composite action at mid-height and $\alpha_c = 0$ corresponds to no composite action. Mathematically, the two mid-height vertical jacket stresses may be written as:

$$(f_{vj})_m = \frac{\bar{u}_o L_j}{2t_j} \tag{4.19}$$

$$(\bar{f}_{vj})_m = \frac{(L' - v_g - \frac{L_1}{2})}{L'}(f_{vj})_{ave} \tag{4.20}$$

Combining the above two equations gives the composite action coefficient α_c as:

$$\alpha_c = \frac{\bar{u}_o L_j L'}{2t_j (f_{vj})_{ave}(L' - v_g - \frac{L_1}{2})} \tag{4.21}$$

The distribution of flexural rigidities in this case is taken to vary linearly from $(EI)_b$ at the free end of the jacket to the maximum $(EI)_m$ at mid-height. The uncased region of the column is assigned the flexural rigidity $(EI)_b$. The variation of column flexural rigidities for case (ii) is also shown in Figure 4.5.

The curvature distribution up the column is obtained by dividing the bending moment by the column flexural rigidities. The first moment of the curvature distribution about the center of seismic mass gives the first yield lateral displacement for the retrofitted column i.e.

$$\Delta'_{yr} = \int_0^{L'} \frac{M(y)}{EI(y)}(L' - y)dy \tag{4.22}$$

where y denotes the vertical distance measured from top of the footing. Integration of Eqn. 4.22 is described in Appendix A.

4.3.2.2 Ultimate Displacement

It was pointed out in Section 3.4.4 that two possible limit states exist for steel jacketed columns; one corresponds to the enhanced ultimate compressive strain of the cover concrete as a result of confinement by the steel jacket, and the other corresponds to a low-cycle fatigue fracture of longitudinal steel.

For the ultimate limit state in which the enhanced compressive strain of cover concrete governs, the ultimate displacement Δ_{ur} may be obtained in a manner similar to the 'as-built' column by assuming a plastic hinge rotation at the base of the column. Rewriting Eqns. 4.4 to give the ultimate displacement for retrofitted column:

$$\Delta_{ur} = \Delta_{yr} + \theta_p(L - 0.5L_{pr}) \tag{4.23}$$

where

$$\theta_p \approx (\phi_{ur} - \phi'_{yr}\frac{M_p}{M_{yr}})L_{pr} \tag{4.24}$$

and ϕ_{ur} and ϕ'_{yr} are the ultimate and first yield curvature, respectively; L = effective height of the column (Eqn. 4.3); L_{pr} = plastic hinge length for retrofitted columns; and M_p = plastic moment, and is assessed using a compressive strain of 0.005 in the extreme fiber of the concrete core. The term Δ_{yr} represents an equivalent elasto-plastic yield displacement and is related to the first yield lateral displacement by:

$$\Delta_{yr} = \Delta'_{yr}\frac{M_p}{M_{yr}} \tag{4.25}$$

Since the presence of steel jacket at the base of the column provides constraint against the expansion and spalling of concrete similar to that provided by the footing, the yielding of longitudinal reinforcement into the jacket will be confined to a very short length similar to strain penetration of longitudinal bars into the footing. Thus, plasticity can be assumed to spread into the jacket and footing by $6d_b$, giving a plastic hinge length of:

$$L_{pr} = 12d_b + v_g \tag{4.26}$$

This equation has been shown to give reasonable estimation of the plastic hinge length for steel jacketed columns [1].

The enhancement of concrete compressive strain by the steel jacket results in an increase in the tensile strain of the extreme longitudinal reinforcement. The increase in the extreme tension steel strain is large as a result of reduced plastic hinge length associated with steel jacketing. Under cyclic loading, the longitudinal reinforcement may fracture at a strain lower than the ultimate tensile strain of the longitudinal steel because of low-cycle fatigue, thus resulting in a smaller displacement ductility factor. The assessment of reduced displacement ductility factor based on cumulative damage model was recently proposed in [34]. Such damage model however cannot been directly incorporated in COLRET, since the model depends on the load history to be imposed on the column. The assessment of monotonic displacement ductility factor by COLRET for steel jacketed columns is based on the extreme cover fiber strain reaching the ultimate compressive strain as predicted by Eqn. 3.38, or the extreme tension steel reaching the ultimate tensile strain ϵ_{su}, whichever corresponds to a smaller displacement ductility factor. For the possible damaging effect on column ductilities due to repeated cyclic loads during a severe earthquake earthquake, a 25% reduction of the displacement ductility factor computed by COLRET based on examples in Chapter 6 appears to be reasonable at this stage.

4.3.3 Bond Strength

The amount of lateral stiffening on the column resulted from retrofitting depends on, among other factors, the bond strength between jacket and grout infill. It is implicit in the development of the computer program that the interface between grout and jacket will be critical for bond transfer. The distribution of flexural rigidity up the column can be determined if the average bond strength $\bar{u}_o$ is established.

A large variation of bond strengths have been reported by various investigators [14,35,36,37]. Experimental bond strengths were found to vary from about 22 psi to 270 psi. It must be recognized that the bond strength depends on the contact surfaces of the steel, presence of shrinkage strains, properties of the grout or concrete constituents (aggregate size, use of expanding additive, etc.), loading history and experimental procedures for establishing bond strength.

Experimental testings by Virdi and Dowling [37] involved displacing a column of concrete core through a steel tube. The loading condition caused a lateral expansion of the core which increases the frictional resistance between the tube and concrete interface. Experimental results revealed a relatively high value of bond strength; the mean value and standard of deviation of 275 and 73 psi, respectively. Similar experiments were carried out by Morishita et al [35,36] in which bond strength in circular, square and octagonal concrete-filled steel columns were investigated. Axial loads were applied to the column through the rim of the steel tube at the top. The tubes were supported at the bottom by thick steel plates which inhibits relative slips between concrete and tube. Experimental results showed a slightly higher bond strength for circular columns than for square columns. Octagonal columns showed a bond strength intermediate of the two. Overall values for bond strength were considerably lower (between 28 and 56 psi) when compared to that obtained by Virdi and Dowling [37]. The low values may be explained by lateral expansion of the steel tube away from the concrete when axial compression is applied. The expansion of the tube causes a reduction in the adhesion at the concrete and steel tube interface which is reflected in the reduced measured bond strength. Both experiments however suggested that bond strength was not significantly influenced by the concrete strength f'_{co}. The increase in concrete strength is typically compensated by higher heat of hydration which generates larger shrinkage during hardening phase of the concrete. Thus a higher concrete strength produces an offsetting effects on the frictional resistance at the concrete-tube interface.

More recent works by Park [14] on steel-encased concrete piles subjected to simulated seismic loading showed that bond strength between concrete and steel did not degrade significantly under cyclic loading. An average bond strength of 165 psi was obtained and was intermediate of the values obtained by Virdi and Dowling [37], and Morishita et al [35,36]. The variation of predicted elasto-plastic yield displacements, Δ_{yr}, with the average bond strength, $\bar{u}_o$, is shown in Figure 4.6. A value of $\bar{u}_o = 110$ psi appeared appropriate for the steel jacketed columns reported in [1], and this value has been assumed by the program COLRET.

The theoretical curvature within the steel jacket region may be determined once an average bond strength $\bar{u}_o$ is defined. Figure 4.7 compares the experimental curvatures within the steel jacket region at displacement ductility factor $\mu = 1$ with the theoretical curvatures obtained from an average bond strength of $\bar{u}_o = 110$ psi. It is noted that the experimental curvatures measured

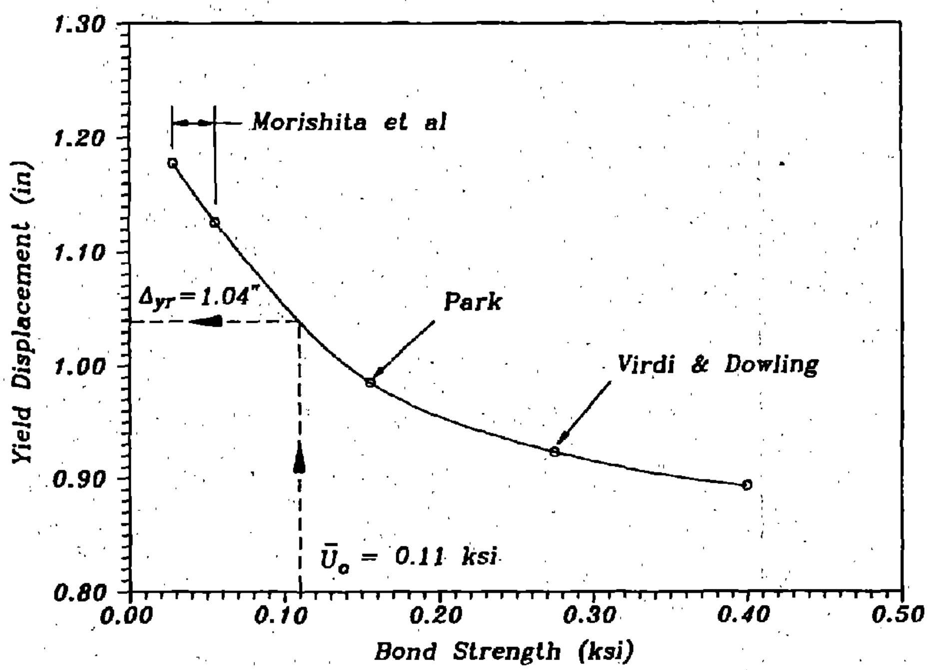


Figure 4.6: Variation of Yield Displacement with Bond Strength

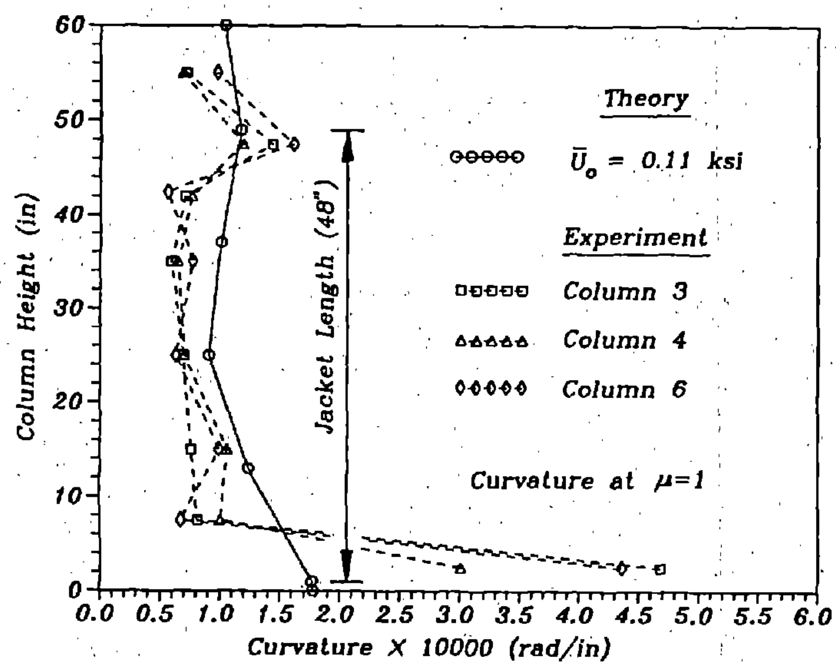


Figure 4.7: Curvature Distribution within Steel Jackets at $\mu = 1$

at the base of the column were affected by strain-penetration of the longitudinal bars into the footing and were not representative of the true curvature at that location. Additional deformations were measured by the pair of linear potentiometers which were targeted on top of the footing. Reasonable prediction of curvatures is achieved within the steel jacket, although slight over-prediction is evident in the central region of the jacket. The over-prediction however is compensated by an under-prediction observed near the top of the jacket.

Chapter 5

Shear Strength

5.1 General

The desirable ductile behavior of bridge columns depends on the proper development of flexural hinges at the critical regions of the column. Flexural yielding of longitudinal reinforcement in plastic hinge region provides a reliable means of dissipating the seismic energy that is imparted to the bridge structure. It is importance that premature shear failures do not occur either outside or within the plastic hinges because shear failures tend to be non-ductile especially under large axial compression. Rapid strength and stiffness degradation occurs during a shear failure and will jeopardize the vertical load carrying capacity of the column. To insure against shear failure, the shear capacity of the column must be higher than the shear force corresponding to the development of maximum feasible flexural strength; an approach termed capacity design.

5.2 Design Shear Force

The acceptance by bridge engineers to design for force levels smaller than the theoretical elastic inertial forces implies that inelastic actions will invariably occur under the design earthquake even though the actual flexural capacity may exceed the design values due to higher material strengths or excess reinforcement provided above the design requirements. The design shear force for bridge columns therefore does not correspond to the reduced level of response inertial force but to the shear force at the development of the actual flexural strength of the column.

It has been pointed out by Ang et al [38] that the maximum feasible flexural strength will exceed the design flexural strength because of the following practices: (1) flexural strength reduction factors commonly employed in flexural strength design calculations, (2) material strengths (concrete and longitudinal reinforcement) exceeding their nominal design values, (3) the conservative nature of design equations in various codes for flexural strength estimation, (4) the provisions by designers of excess flexural reinforcement above that required by the code level of bending moment, and (5) possible higher dynamic modes during response of the structure, causing a deviation in the position of the point of contraflexure in the columns from that predicted by elastic static analysis using the code-specified distribution of lateral loads. As outlined by Ang et al [38], the maximum feasible design shear force, V_D, may then be related to the code-specified column shear force, V_{CL}, via the equation:

$$V_D = \frac{\omega_v k_1 k_2 k_3}{\phi_f} V_{CL} \leq V_E \tag{5.1}$$

where ϕ_f is the flexural strength reduction factor; k_1 is the material overstrength factor; k_2 is the factor reflecting the conservatism in code expression for flexural strength; k_3 represents the excess flexural strength resulting from provision of excess reinforcement; ω_v defines the shear amplification due to higher dynamic modes; and V_E is the shear force corresponding to elastic response. Although Eqn. 5.1 may be used to relate the maximum feasible design shear force V_D to the shear force V_u which corresponds to the development of ultimate moment, the above factors must be re-examined when assessment or retrofit of existing structures is considered.

The flexural strength computed by this program corresponds to the ideal capacity of the column which implies a strength reduction factor of $\phi_f = 1$. Instead of using the ACI [24] equivalent stress-block for the compression zone of the column section, the flexural strength is predicted using a discretized section described in Chapter 3. The confinement of concrete core is taken into account using a realistic confined concrete model and the ultimate limit state of the column is defined by a maximum compressive strain predicted from balance of strain energies between concrete core and confining steel. Also the possibility of flexural strength increase due to strain-hardening in the longitudinal steel is accounted for. Thus the conservatism expressed by factor k_2 is unwarranted and a value of unity is appropriate. Since the computer program models the transverse response of the bridge structure as a single-degree-of-freedom oscillator, the value of $\omega_v = 1$ is implicit. Since the program is intended for assessment purposes, k_3 also has the value of unity, and the only enhancement is that resulting from material strengths possibly exceeding the assumed values. Hence

$$V_D = k_1 V_u \tag{5.2}$$

where V_u = shear force corresponding to maximum moment predicted by COL-RET. In the absence of information on the actual material strengths and reinforcement provided, a value of 1.15 is deemed satisfactory for possible shear force enhancement.

5.2.1 Column Shear Strength

The current ACI 318 Code [24] and NZ3101 Code [25] consider the column ideal shear strength V_{COL} to be the sum of shear forces carried by the concrete shear resisting mechanism, V_c, and a truss mechanism, V_s, involving the transverse reinforcement and 45^0 concrete compression struts i.e.

$$V_{COL} = V_c + V_s \geq \frac{V_D}{\phi_s} \tag{5.3}$$

where ϕ_s is the shear strength reduction factor. A shear strength reduction factor of $\phi_s = 0.85$ is recommended by the ACI 318 Code [24], whereas NZ3101 Code [25] uses $\phi_s = 1.0$ when the shear force is based on the capacity design approach indicated in Eqn. 5.1. The term V_c is also called the 'concrete contribution' to the column shear strength.

5.2.1.1 'As-built' Columns

Experimental studies by Ang et al [39] have shown that the concrete contribution, V_c, in circular columns are considerably higher than that allowed by current codes e.g. ACI318 and NZ3101 [24,25]. Code expressions for concrete contribution V_c are typically based on tests of rectangular beams and are not representative of bridge columns which generally contain well-distributed longitudinal reinforcement and significant axial forces. Under seismic conditions, the shear resistance in the plastic hinge region may decrease as a result of excessive crack opening reducing the effectiveness of aggregate interlock. Despite the reduction, the concrete contribution V_c to column shear strength may still be significant. The current approach by most codes, however, ignores the concrete contribution within the potential plastic hinge region if axial compression is small. For example, the ACI318 Code [24] assumes $V_c = 0$ in the plastic hinge region if the axial compression P is less than $0.05f'_{co}A_g$, and the NZ3101 Code [25] has a similar requirement for $P \leq 0.1f'_{co}A_g$. Even though the concrete contribution V_c degrades under large column ductilities, the angle of crack inclination (relative to the column longitudinal axis) tend to increase with column displacements. The reduced crack inclination increases the column shear force to be resisted by the transverse reinforcement, V_s, thereby compensating for the degradation of concrete contribution V_c.

An assessment of ideal shear strength for circular columns under cyclic loading was recently proposed by Ang et al [38]. The degradation of ideal column shear strength V_{COL} with flexural ductility factor μ within the plastic hinge region, is shown in Figure 5.1. At low ductilities, for example $\mu \leq 2$, the column shear strength V_{COL} attains a maximum and is denoted by the initial shear strength V_I. At higher ductilities, the column ideal shear strength degrades linearly with μ until a final shear strength $V_{COL} = V_F$ is attained at $\mu = 6$.

Full flexural response is assured if the design shear force V_D is less than or equal to the final shear strength V_F. Otherwise an adjustment to the displacement ductility factor μ is made (Line 1 in Figure 5.1). The adjusted ductility factor can be written as:

$$\mu = 2 + 4\left(\frac{V_I - V_D}{V_I - V_F}\right) \tag{5.4}$$

In the region where $2 < \mu < 6$, the column shear strength V_{COL} is interpolated from the displacement ductility factor (Line 2 in Figure 5.1). This level of ductility can be maintained if the design shear force is less than the ideal column shear strength i.e. $V_D \leq V_{COL}$. In cases where the design shear force exceeds the

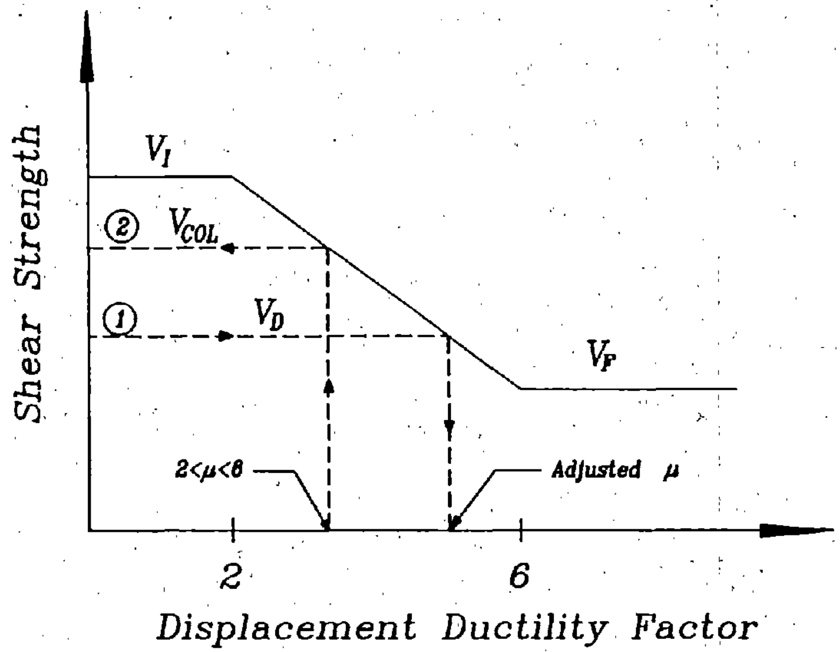


Figure 5.1: **Column Shear Strength versus Displacement Ductility Factor μ**

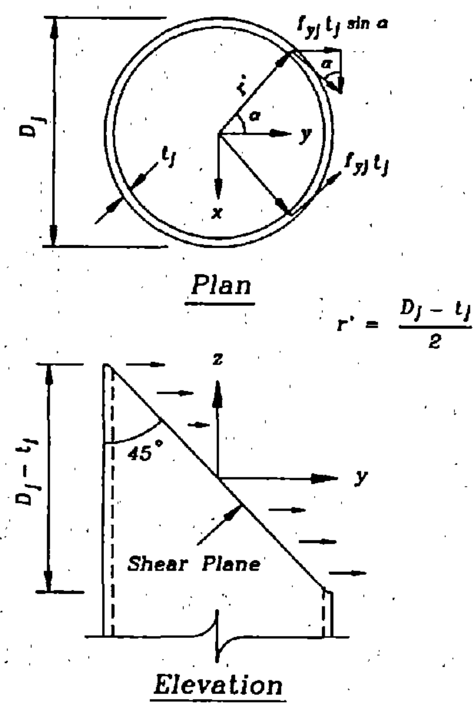


Figure 5.2: **Shear Strength Contribution from Steel Jacket**

ideal column shear strength i.e. $V_D > V_{COL}$, the displacement ductility factor is reduced in accordance with the above equation.

In the region where $\mu < 2$, the column shear strength V_{COL} is assigned the initial shear strength V_I. Limited ductility is implied in this region if the design shear force is less than the initial shear strength i.e. $V_D \leq V_I$. Brittle shear failure may occur if the design shear force exceeds the initial shear strength, i.e., $V_D > V_I$.

The initial shear strength V_I proposed by Ang et al [38] is given by (in U.S. Customary Units):

$$V_I = V_{CI} + V_{SI} \tag{5.5}$$

where the concrete contribution is:

$$V_{CI} = 4.45\alpha_s \left(1 + \frac{3P}{f'_{co}A_g}\right)\sqrt{f'_{co}}A_e \tag{5.6}$$

and the shear force resisted by the transverse steel, assuming a 45^0 analogous truss mechanism, is:

$$V_{SI} = \frac{\pi}{2}A_{sh}f_{yh}\frac{d_s}{s} \tag{5.7}$$

The initial shear strength given by Eqn. 5.6 applies within the plastic hinge zone for $\mu \leq 2$, and outside the plastic hinge regions for all ductility levels. The term A_e in Eqn. 5.6 is called the effective shear area and is recommended by Ang et al [38] as $0.8A_g$ where A_g is the gross sectional area; and P is the factored axial force occurring simultaneously with the design shear force V_D; and

$$\alpha_s = \frac{2}{M/VD} \geq 1.0 \tag{5.8}$$

represents the aspect ratio of the column, and implies an increase of the initial shear strength when the aspect ratio is reduced below 2. Setting $\alpha_s = 1.0$ and

$P = 0.0$ in Eqn. 5.6 gives an initial concrete stress of $v_{ci} = 4.45\sqrt{f'_{co}}$, which is significantly higher than the familiar ACI's expression of $2\sqrt{f'_{co}}$ for concrete contribution [24]. Even though the initial shear strength equation proposed by Ang et al [38] was based on experimental testings of small diameter columns (16 inches in diameter), the expression appeared to be supported by recent large-scale shear column tests carried out at the University of California, San Diego [40].

In a similar manner, the final degraded shear strength of the column V_F within the plastic hinge region is written as the sum of the concrete and steel contribution:

$$V_F = V_{CF} + V_{SF} \tag{5.9}$$

An increase in the shear resistance of the transverse reinforcement, however, can occur as the concrete shear resisting mechanism degrades under large ductilities, resulting in inclination steeper than 45^0 for the diagonal compression field. To account for the increase in column shear strength due to steeper diagonal strut, Ang et al [39] multiplied the initial shear resistance of the transverse steel by $\cot\theta$, where θ denotes the inclination of the diagonal strut relative to the column longitudinal axis. Using a lower bound solution in the plasticity theory, Ang et al [39] proposed that the inclination angle of the diagonal strut to be given by:

$$\theta = \cot^{-1}\sqrt{\frac{1-\psi}{\psi}} \quad \geq 25^0 \tag{5.10}$$

where the ψ is termed the mechanical ratio for the shear reinforcement, and is given by [39]:

$$\psi = \frac{\rho_s f_{yh}}{\nu f'_{co}} \tag{5.11}$$

The factor ν in Eqn. 5.11 is called the web effectiveness factor and is introduced to account for the reduction of the concrete compressive strength in the presence of transverse tensile strain. Based on test results, a value of $\nu = 0.2$ was suggested for the web effectiveness factor. Thus, the shear force that can be resisted by the transverse reinforcement for $\mu \geq 6$ is given by:

$$V_{SF} = \frac{\pi A_{sh} f_{yh} d_s}{2s}\sqrt{\frac{1-\psi}{\psi}} \leq \frac{2.15\pi A_{sh} f_{yh} d_s}{2s} \tag{5.12}$$

As implied by Eqn. 5.10, the inclination angle for the diagonal compression strut is limited to 25^0, thus giving rise to the factor 2.15 in Eqn. 5.12.

From the residual strengths measured at large ductilities for a series of test columns, Ang et al [39] proposed that the final concrete contribution, V_{CF}, be given by one-half of the initial concrete contribution when the transverse steel ratio is $\rho_s \geq 0.01$. Otherwise, for $\rho_s < 0.01$, it was proposed that the final concrete contribution V_{CF} be given as a linear increase with ρ_s [39]. Thus, the final concrete contribution to column shear strength is:

$$V_{CF} = 222.7\rho_s\sqrt{f'_{co}}A_e \leq 2.227\sqrt{f'_{co}}A_e \tag{5.13}$$

5.2.1.2 Retrofitted Columns

The use of steel jackets in retrofitted columns should increase the shear strength of the column in the encased region. Figure 5.2 shows the shear resistance of a steel jacket assuming a 45^0 failure plane and that the jacket is in a state of uniaxial hoop stress. The failure plane will expose a tension resultant $f_{yj}t_j$ tangential to the steel jacket. For an infinitesimal jacket height, dz, the shear force resisted by the steel jacket is:

$$dV_{sj} = 2f_{yj}t_j \sin\alpha \, dz \tag{5.14}$$

In the coordinate system shown, the shear failure plane is given by $z = -y$. Since $y = r' \cos\alpha$ where $r' = (D_j - t_j)/2$, the infinitesimal height dz may be written as:

$$dz = r' \sin\alpha \, d\alpha \tag{5.15}$$

Substituting back into Eqn. 5.14 gives:

$$V_{sj} = \int_0^{\pi} 2 f_{yj} t_j r' \sin^2\alpha \, d\alpha \tag{5.16}$$

Noting that

$$\int_0^{\pi} \sin^2\alpha d\alpha = \frac{\pi}{2} \tag{5.17}$$

the shear force resisted by the steel jacket is:

$$V_{sj} = \frac{\pi}{2} f_{yj} t_j (D_j - t_j) \tag{5.18}$$

If the steel jacket is extended to the full height of the column, the governing column shear strength will be increased to:

$$V_{COL} = V_F + V_{sj} \tag{5.19}$$

where V_F = final shear strength given by Eqn. 5.9. If however the steel jacket does not extend to the full height of the column, the ideal shear strength of the column will be the lesser of $V_{COL} = V_I$ or $V_F + V_{sj}$. The shear strength above the steel jacket will be given by the initial shear strength V_I (Eqn. 5.5) whereas the shear strength within the potential plastic hinge region of retrofitted column is $V_F + V_{sj}$. The latter will govern when an inadequate jacket thickness is provided.

By substituting Eqn. 5.18 into Eqn. 5.19 and equating the design shear force V_D to the column shear strength V_{COL}, an expression for minimum jacket thickness $(t)_{min}$ to satisfy shear requirement may be obtained:

$$(t_j)_{min} = \frac{1}{2}\left(D_j - \sqrt{D_j^2 - \frac{8(V_D - V_F)}{\pi f_{yj}}}\right) \tag{5.20}$$

Chapter 6

Prediction of Column Response

6.1 General

This Chapter provides a comparison between the envelope curves of test columns reported in [1,2] and theoretical predictions by the computer program COLRET developed in this report. The comparison is divided into separate sections for 'as-built' and retrofitted columns.

6.2 'As-built' Columns

6.2.1 Full-Scale Flexure Column

The first example analyzed by the program COLRET was the full-scale (60" diameter) flexure column tested by the National Institute of Standards and Technology (NIST) [2]. The column represented the current ductile design for bridge columns. Table 6.1 summarizes the parameters for the test column which was

Table 6.1: **Design Details for Full-Scale Flexure Column**

Diameter D	60"
Height L'	30'
Cover to Main Bar	4"
Concrete Strength f'_{co}	5.2 ksi
Longitudinal Steel Yield Strength f_y	25#14 68.9 ksi
Transverse Steel Yield Strength f_{yh}	#5 Spiral at 3.5" 71.5 ksi
Axial Force	1000 kips

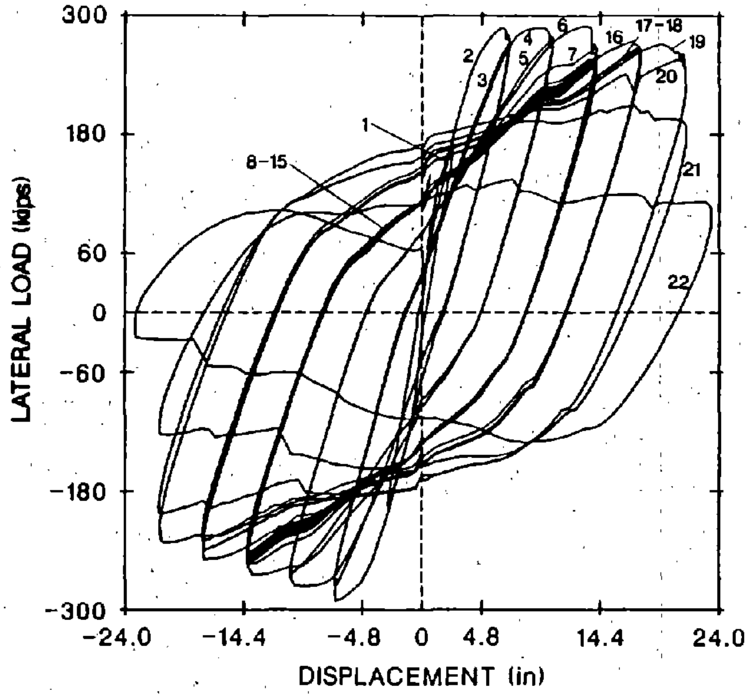


Figure 6.1: **Hysteretic Response of Full-Scale Flexure Column - Ref. 2**

subjected to an axial compression force of 1000 kips and a lateral cyclic displacement of increasing amplitudes until failure of column. Figure 6.1 shows the hysteretic response of the column. Detailed descriptions of the full-scale test are reported elsewhere [2].

The analytical results predicted by COLRET are shown in Table 6.2. The predicted elasto-plastic yield displacement for the column was $\Delta_y = 4.211$ inches. The reported experimental yield displacement was however only 3.53 inches and was low for two reasons. The definition of experimental yield displacement was based on the ACI [24] estimation of the moment capacity (see Figure 6.2) i.e.

$$\Delta_y = \Delta_{expt} \frac{M_{ACI}}{0.75 M_{ACI}} \tag{6.1}$$

where Δ_{expt} is the experimental displacement measured at 75% of the ACI moment capacity [24]. The ACI method uses an ultimate concrete strain of 0.003

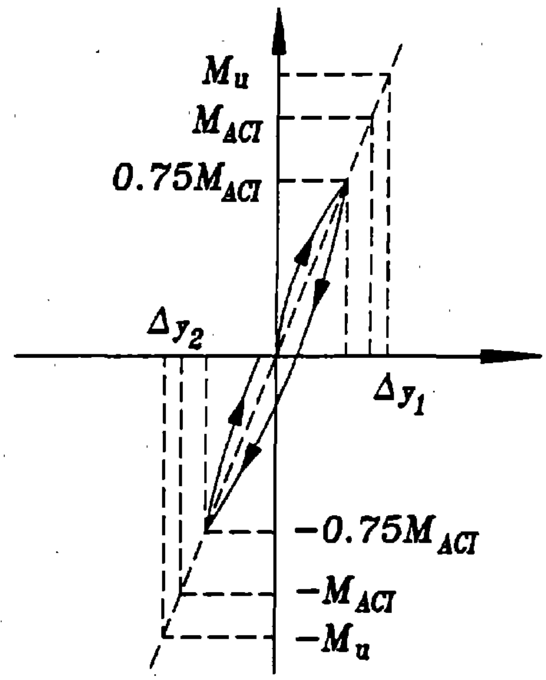


Figure 6.2: Experimental Definition of Yield Displacement by NIST

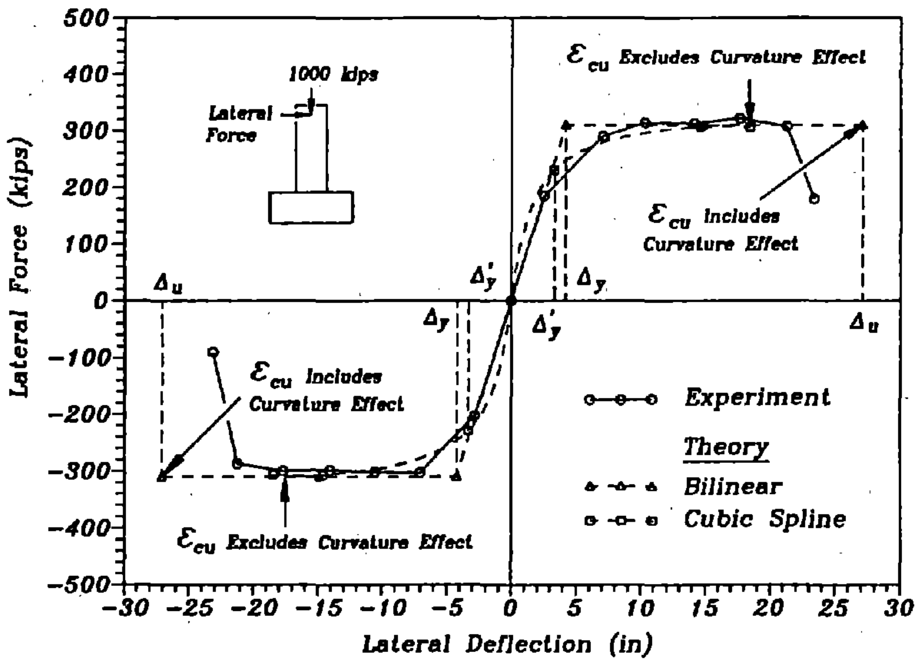


Figure 6.3: Envelope Curves for 'As-Built' Full-Scale Flexure Column

and gives a moment capacity of 8041 kip.ft which is considerably smaller than the actual experimental moment capacity of 9643 kip.ft. The nominal moment capacity predicted by COLRET was 9292 kip.ft; much closer to the measured moment capacity. It was also reported in [2] that because of the difficulties in measuring the frictional force between the column base and test floor, the actual lateral load applied to the column may be under-estimated. It was suggested that the experimental yield displacement be increased by 2%. Thus the corrected yield displacement should be:

$$\begin{aligned} \Delta_y &= 3.53 \times \frac{9292}{8041} \times 1.02 \\ &= 4.16 \quad \text{inches} \end{aligned}$$

It can be seen that the yield displacement predicted by COLRET is within 2% of the experimental value.

Figure 6.3 shows a comparison of the envelope curve predicted by COLRET and that reported in [2]. Two predictions are shown in the figure; (i) a bilinear approximation through the equivalent elasto-plastic yield displacement Δ_y and the ultimate displacement Δ_u, and (ii) a curve fitted by cubic-spline through the first yield displacement Δ'_y and the ultimate displacement Δ_u, where the first yield displacement is $\Delta'_y = 4.211 \times 6890/9292 = 3.122$ inches. Figure 6.3 shows a good prediction of the envelope curve by COLRET for both the initial and post-yield load range. The ultimate displacement predicted by COLRET was $\Delta_u = 14.864$ inches and significantly under-estimated the ultimate displacement of the test column. The response of the test column, shown in Figure 6.1, indicated a degradation of strength of $\approx 13\%$ between the second and third cycle during the lateral displacement to 21.19 inches. Compared to this value,

Table 6.2: **Analytical Results for 'As-Built' Full-Scale Flexure Column**

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*     UNIVERSITY OF CALIFORNIA   SAN DIEGO      *
*                                               *
*   STRENGTH AND DUCTILITY OF CIRCULAR COLUMN   *
*                                               *
*          VERSION 1.1  (MAR 1991)              *
*                                               *
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JOB TITLE : NATIONAL BUREAU OF STANDARDS FULL SCALE FLEXURE COLUMN

                  ORIGINAL COLUMN PARAMETERS
                  **************************

                  DIAMETER OF COLUMN :     60.000 in
COLUMN HEIGHT TO PT OF CONTRAFLEXURE :     30.000 ft
                PLASTIC HINGE LENGTH :     38.958 in
                   COVER TO MAIN BAR :      4.000 in

                            MAIN BAR :       #14
                      NUMBER OF BARS :        25
       YIELD STRENGTH FOR MAIN STEEL :     68.9 ksi   (H.S.)
          ULTIMATE STRENGTH OF STEEL :    103.4 ksi
            YOUNGS MODULUS FOR STEEL :  29000.0 ksi
               YIELD STRAIN OF STEEL :      .00238
        STRAIN AT HARDENING OF STEEL :      .01188
            ULTIMATE STRAIN OF STEEL :      .12000

                            TIE SIZE :       # 5      (Spiral)
                               PITCH :    3.500 in
               YIELD STRENGTH OF TIE :     71.5 ksi   (H.S.)

                   CONCRETE STRENGTH :      5.200 ksi
            CONCRETE ULTIMATE STRAIN :      .017

                   COLUMN AXIAL LOAD :   1000.0 kips

                 RESULTS FOR ORIGINAL COLUMN
                 ***************************

 ORIGINAL  YIELD   MOMENT =  82678.9 kip.in
 ORIGINAL ULTIMATE MOMENT = 111510.8 kip.in

 CURVATURE AT FIRST YIELD OF EXTREME REBAR =   .00007228 Rad/in
 EQUIVALENT ELASTO-PLASTIC YIELD CURVATURE =   .00009748 Rad/in
 ULTIMATE                        CURVATURE =   .00087721 Rad/in
 CURVATURE        DUCTILITY         FACTOR = 9.0

 YIELD            DISPLACEMENT =    4.211 in
 ULTIMATE         DISPLACEMENT =   14.864 in
 DISPLACEMENT DUCTILITY FACTOR =    3.530            (FLEXURE)

              **** LIMITED DUCTILITY ****

 MAXIMUM FEASIBLE  SHEAR  FORCE =   309.8 kips ( K1 = 1.000 )
 IDEAL SHEAR STRENGTH OF COLUMN =  1172.2 kips
```

COLRET under-predicted the ultimate displacement by about 30%, and was a consequence of the use of Eqns. 3.27 and 3.29 which were intended for the prediction of ultimate compressive strains in axially loaded columns. The ultimate compressive strain predicted in this case was only $\epsilon_{cu} = 0.017$.

It was pointed out earlier 3 that the prediction of ultimate displacement can be improved by including the influence of strain-gradient on the ultimate concrete compressive strain. Even though this feature has not been incorporated directly in COLRET, the improvement can be effected by iterating between the laminar analysis outlined in Section 3.4.3 and COLRET until convergence of the ultimate compressive strain ϵ_{cu} and ultimate curvature ϕ_u. In this case, the ultimate compressive strain ϵ_{cu} converges to about 0.033, with a corresponding curvature of 0.001649 rad/in, and an ultimate displacement of 27.06 inches. The predicted ultimate displacement including strain-gradient effect exceeded the actual ultimate displacement of the column by about 27% and may be attributed to the exclusion of the strain energy absorbed by the longitudinal steel during energy balancing process, resulting in over-prediction of ultimate compressive strain, as discussed in Section 3.4.2 and 3.4.3. It should be noted that the tensile strain in the extreme longitudinal bar implied by the theoretical ultimate curvature was $\approx 5.4\%$, whereas the reported ultimate strain of the longitudinal steel was 15.5%. First fracture of the transverse spiral was reported to occur during the second cycle to 21.19 inches at about 12 inches from the top of the footing where the transverse spiral was spliced, resulting in compression buckling of the longitudinal bar [2]. Subsequent re-straightening of the buckled longitudinal bar upon load reversal precipitated fracturing of the bar in tension at a strain lower than the ultimate tensile strain. The failure mode was similar to the low-cycle fatigue fracture of longitudinal reinforcement observed for steel jacketed columns.

6.2.2 Column 3 - 'As-built', No Laps

The second example analyzed by COLRET was the 0.4 scale model of 'as-built' circular bridge columns (test column 3) described in a companion report [1]. The test column was designed with 2.53% longitudinal reinforcement anchored by 90^0 hooks into the footing. A transverse steel ratio of $\rho_s = 0.0017$, corresponding to #4 transverse hoops at 12 inches centers commonly provided in pre-1971 design, was used for the test column. Detailed descriptions of the design and test results for the column can be found in [1].

Table 6.3 summarizes the analytical results predicted by COLRET for the test column 3. It was reported that [1] the axial force applied on the column increased with lateral displacements of the column, due to a net extension of the column resulted from cracking in the column. The maximum axial force applied on the column was 440 kips occurring at $\mu = 5$, and this value was used here for the prediction of the column response.

The predicted elasto-plastic yield displacement by COLRET was $\Delta_y =$ 1.081 inches and compared well with the experimental yield displacement of 1.082 inches. The predicted ultimate lateral force was 47.8 kips; $\approx$ 10% low compared to the peak measured lateral force of 53 kips (after correcting for the horizontal component of the axial force). It should be noted that the predicted peak lateral force did not occur at the ultimate compressive strain of $\epsilon_{cu} = 0.009$, but at a lower strain. A peak lateral force of 51.5 kips was calculated for an extreme concrete compressive strain of 0.004, giving a lateral displacement of 1.878 inches. The peak lateral force in this case was determined by modifying the program

Table 6.3: **Analytical Results for Column 3 - 'As-built', No Laps**

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*                                               *
*     UNIVERSITY OF CALIFORNIA   SAN DIEGO      *
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*                                               *
*           VERSION 1.1  (MAR 1991)             *
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JOB TITLE : SEISMIC RETROFIT OF BRIDGE COLUMN - COLUMN 3 (14 Feb 1989)

                 ORIGINAL COLUMN PARAMETERS
                 **************************

                 DIAMETER OF COLUMN :     24.000 in
COLUMN HEIGHT TO PT OF CONTRAFLEXURE :     12.000 ft
               PLASTIC HINGE LENGTH :     16.020 in
                  COVER TO MAIN BAR :       .800 in

                           MAIN BAR :        # 6
                     NUMBER OF BARS :         26
       YIELD STRENGTH FOR MAIN STEEL :     45.7 ksi   (Mild Steel)
          ULTIMATE STRENGTH OF STEEL :     68.5 ksi
            YOUNGS MODULUS FOR STEEL :  29000.0 ksi
               YIELD STRAIN OF STEEL :      .00158
        STRAIN AT HARDENING OF STEEL :      .02205
            ULTIMATE STRAIN OF STEEL :      .16205

                           TIE SIZE :        # 2      (Hoops)
                        TIE SPACING :    5.000 in
               YIELD STRENGTH OF TIE :     51.0 ksi   (Mild Steel)

                  CONCRETE STRENGTH :      4.725 ksi
           CONCRETE ULTIMATE STRAIN :       .009

                  COLUMN AXIAL LOAD :    440.0 kips

                RESULTS FOR ORIGINAL COLUMN
                ***************************

   ORIGINAL  YIELD   MOMENT =   6144.2 kip.in
   ORIGINAL ULTIMATE MOMENT =   6889.9 kip.in

   CURVATURE AT FIRST YIELD OF EXTREME REBAR =   .00013941 Rad/in
   EQUIVALENT ELASTO-PLASTIC YIELD CURVATURE =   .00015633 Rad/in
   ULTIMATE                        CURVATURE =   .00097442 Rad/in
   CURVATURE         DUCTILITY        FACTOR =  6.2

   YIELD             DISPLACEMENT =    1.081 in
   ULTIMATE          DISPLACEMENT =    2.922 in
   DISPLACEMENT DUCTILITY FACTOR =     2.704             (FLEXURE)

              **** LIMITED DUCTILITY ****

   MAXIMUM FEASIBLE  SHEAR  FORCE =    47.8 kips ( K1 = 1.000 )
   IDEAL SHEAR STRENGTH OF COLUMN =   171.1 kips
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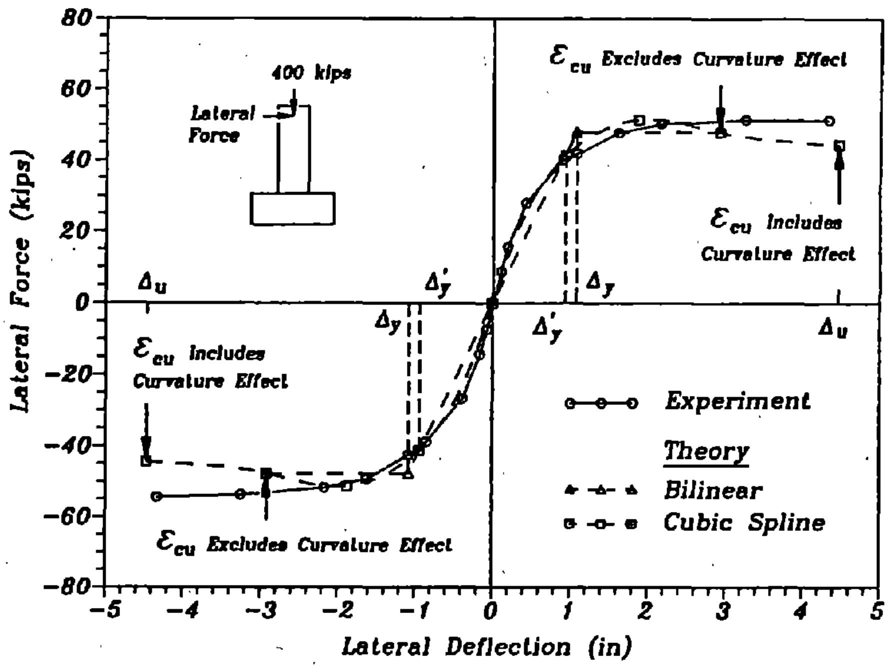

Figure 6.4: **Envelope Curve for Column 3 - 'As-built', No Laps**

COLRET to allow a direct input of the ultimate concrete compressive strain.

The ultimate displacement predicted by COLRET for column 3 was 2.922 inches, whereas the test column showed an experimental ultimate displacement of 4.328 inches ($\mu = 4$), corresponding to incipient buckling of the longitudinal reinforcement. Thus COLRET under-predicted the ultimate displacement by about 32%, and was again due to the use of Eqn. 3.27 and 3.29 intended for the prediction of the ultimate compressive strains in axially loaded columns. The corresponding ultimate compressive strain predicted by COLRET was $\epsilon_{cu} = 0.009$. The prediction of the ultimate displacement was subsequently improved by iterating between the laminar analysis for strain-gradient effect and COLRET, as was carried out in previous example. The ultimate compressive strain ϵ_{cu}, in this case, increased to about 0.0156, with a corresponding lateral force of 44.3

kips, a curvature of 0.00156 rad/in, and an ultimate displacement prediction of 4.463 inches. The improved ultimate displacement exceeded the experimental ultimate displacement by only 3%.

Figure 6.4 shows the comparison between the prediction by COLRET and the experimental envelope curve reported in [1]. Similar to the previous example, two predictions were included in Figure 6.4: (i) a bilinear approximation through the equivalent elasto-plastic yield displacement Δ_y and the ultimate displacement Δ_u, and (ii) a cubic spline fit through the following points (a) first yield lateral force of 42.7 kips at first yield displacement of $\Delta'_y = 0.964$ inches, (b) the peak lateral force of 51.5 kips at displacement of 1.878 inches, (c) the lateral force of 47.8 kips at displacement of 2.922 inches, and (d) the ultimate lateral force of 44.3 kips at displacement of 4.463 inches. It can be seen from Figure 6.4 that a reasonable representation of the column response was achieved by both predictions even though the experimental response showed a less rapid degradation of lateral strength compared to the theoretical strength after displacement to $\mu \geq 2$. It was reported in [1] that the test column exhibited a larger experimental lateral strength in the pull direction than in the push direction.

6.3 Retrofitted Columns

6.3.1 Column 4 - No Laps

Table 6.4 and 6.5 summarizes the analytical predictions for test column 4 reported in [1] before and after retrofitted with a steel jacket. Experimentally, the test column showed a larger moment capacity in the pull direction than in the push direction. The maximum measured lateral forces, after correcting for

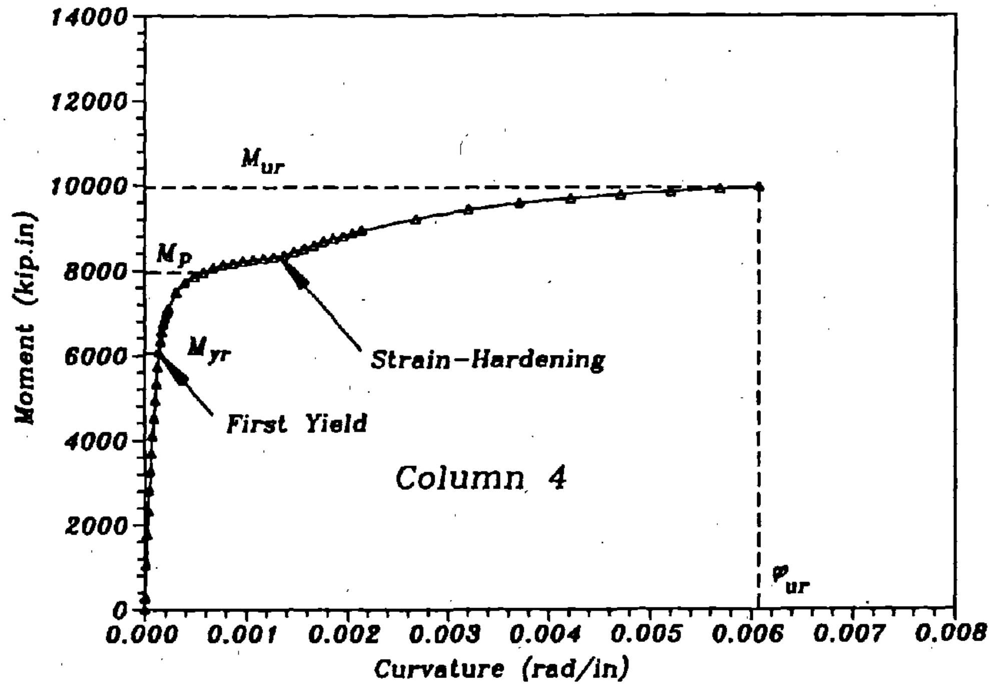

Figure 6.5: **Moment-Curvature Curve for Column 4 (No Laps)**

the horizontal component of the applied axial force, were 61.8 kips in the push direction and 67.1 kips in the pull direction. A shear force of 55.5 kips corresponding to the plastic moment, and a shear force of 68.5 kips corresponding to an ultimate compressive strain of $\epsilon_{cu} = 0.04$ were predicted by COLRET for the column after retrofit. The maximum feasible strength was 6% larger than the average experimental shear force of 64.5 kips. The equivalent elasto-plastic yield displacement predicted by COLRET was $\Delta_{yr} = 1.046$ inches, and agreed well with the measured yield displacement of 1.084 inches.

An ultimate displacement of 9.764 inches was predicted by COLRET for the column after retrofit whereas a longitudinal bar fractured during the third push cycle to a peak displacement of 8.672 inches. Moment-curvature analysis carried out at the base section of column 4 indicated the limit state was governed by the ultimate compressive strain of concrete and by not the strain in the tension

Table 6.4: **Analytical Results for Column 4 (No Laps)**

(a) Before Retrofit

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*                                               *
*     UNIVERSITY OF CALIFORNIA   SAN DIEGO      *
*                                               *
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*                                               *
*          VERSION 1.1  (MAR 1991)              *
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JOB TITLE : SEISMIC RETROFIT OF BRIDGE COLUMN - COLUMN 4 (5/15/89)

                   ORIGINAL COLUMN PARAMETERS
                   **************************

                 DIAMETER OF COLUMN :    24.000 in
COLUMN HEIGHT TO PT OF CONTRAFLEXURE :    12.000 ft
               PLASTIC HINGE LENGTH :    16.020 in
                  COVER TO MAIN BAR :      .800 in

                           MAIN BAR :       # 6
                     NUMBER OF BARS :        26
      YIELD STRENGTH FOR MAIN STEEL :     45.7 ksi   (Mild Steel)
         ULTIMATE STRENGTH OF STEEL :     68.5 ksi
           YOUNGS MODULUS FOR STEEL :  29000.0 ksi
              YIELD STRAIN OF STEEL :      .00158
       STRAIN AT HARDENING OF STEEL :      .02205
           ULTIMATE STRAIN OF STEEL :      .16205

                           TIE SIZE :       # 2      (Hoops)
                        TIE SPACING :    5.000 in
              YIELD STRENGTH OF TIE :     51.0 ksi   (Mild Steel)

                 CONCRETE STRENGTH :      5.520 ksi
          CONCRETE ULTIMATE STRAIN :       .009

                 COLUMN AXIAL LOAD :    400.0 kips

                 RESULTS FOR ORIGINAL COLUMN
                 ***************************

   ORIGINAL  YIELD   MOMENT =    6040.6 kip.in
   ORIGINAL ULTIMATE MOMENT =    6965.1 kip.in

   CURVATURE AT FIRST YIELD OF EXTREME REBAR =    .00013259 Rad/in
   EQUIVALENT ELASTO-PLASTIC YIELD CURVATURE =    .00015288 Rad/in
   ULTIMATE                        CURVATURE =    .00101044 Rad/in
   CURVATURE        DUCTILITY         FACTOR =   6.6

   YIELD            DISPLACEMENT =     1.057 in
   ULTIMATE         DISPLACEMENT =     2.987 in
   DISPLACEMENT DUCTILITY FACTOR =     2.826              (FLEXURE)

               **** LIMITED DUCTILITY ****

   MAXIMUM FEASIBLE  SHEAR  FORCE =     48.4 kips ( K1 = 1.000 )
   IDEAL SHEAR STRENGTH OF COLUMN =    165.2 kips
```

Table 6.4: **Analytical Results for Column 4 (No Laps) - Cont'd**

(b) After Retrofit

```
                RETROFIT COLUMN PARAMETERS
                **************************

         OUTSIDE DIAMETER OF JACKET :     24.875 in
                THICKNESS OF JACKET :       .188 in
                   LENGTH OF JACKET :     48.000 in
            JACKET TOE FROM FOOTING :      1.000 in

           YIELD STRENGTH OF JACKET :     47.000 ksi

         GROUT COMPRESSIVE STRENGTH :      2.000 ksi

        ULTIMATE COMPRESSIVE STRAIN :       .040

               PLASTIC HINGE LENGTH :     10.000 in


                RESULTS FOR RETROFIT COLUMN
                ***************************


           YIELD      MOMENT =   6005.7   kip.in
           PLASTIC    MOMENT =   7988.3   kip.in
           ULTIMATE   MOMENT =   9868.8   kip.in

     CURVATURE AT FIRST YIELD OF EXTREME REBAR =    .00013389 Rad/in
     EQUIVALENT ELASTO-PLASTIC YIELD CURVATURE =    .00017809 Rad/in
     CURVATURE    AT    ULTIMATE     CONDITION =    .00643418 Rad/in
     CURVATURE        DUCTILITY         FACTOR = 36.1

     YIELD            DISPLACEMENT =    1.046 in
     ULTIMATE         DISPLACEMENT =    9.764 in
     DISPLACEMENT DUCTILITY FACTOR =    9.337            (FLEXURE)

     PLASTIC      SHEAR      FORCE =     55.5 kips

           **** INADEQUATE JACKET LENGTH ****

     **** INCREASE JACKET LENGTH TO      48.834 in ****

     MAXIMUM FEASIBLE  SHEAR  FORCE =    68.5 kips ( K1 = 1.000 )
     IDEAL SHEAR STRENGTH OF COLUMN =   195.3 kips
```

Table 6.5: Moment-Curvature Results for Column 4 (No Laps)

MOMENT CURVATURE ANALYSIS FOR RETROFIT COLUMN
****

JOB TITLE : SEISMIC RETROFIT OF BRIDGE COLUMN - COLUMN 4 (5/15/89)

MOMENT (kip.in)	CURVATURE (Rad/in)	STEEL STRAIN
155.7	.00000175	.000158
863.6	.00000972	.000075
1542.5	.00001756	-.000005
2112.2	.00002620	-.000104
2587.2	.00003565	-.000221
3015.2	.00004564	-.000350
3419.6	.00005591	-.000485
3810.8	.00006671	-.000633
4193.8	.00007751	-.000781
4570.1	.00008885	-.000941
4942.5	.00009992	-.001095
5310.6	.00011126	-.001255
5675.1	.00012260	-.001415
6012.5	.00013394	-.001575
6262.6	.00014689	-.001772
6476.9	.00015985	-.001968
6660.9	.00017335	-.002178
6807.8	.00018738	-.002399
6928.9	.00020142	-.002621
7036.6	.00021545	-.002842
7397.3	.00028995	-.004048
7634.3	.00036767	-.005328
7786.9	.00044972	-.006707
7901.5	.00053391	-.008134
7988.3	.00062027	-.009611
8052.7	.00070878	-.011137
8115.5	.00079729	-.012664
8161.9	.00088688	-.014214
8189.3	.00097971	-.015839
8224.5	.00107038	-.017414
8236.0	.00116536	-.019088
8270.8	.00125603	-.020663
8310.2	.00134670	-.022239
8372.9	.00143521	-.023765
8434.2	.00152588	-.025340
8502.1	.00161439	-.026866
8573.9	.00170074	-.028343
8640.5	.00178710	-.029820
8703.0	.00187345	-.031297
8760.8	.00195981	-.032774
9030.8	.00244513	-.041044
9256.9	.00291104	-.048872
9411.7	.00337371	-.056626
9527.8	.00383099	-.064256
9605.7	.00428827	-.071887
9679.9	.00473477	-.079271
9743.3	.00518127	-.086655
9794.7	.00562776	-.094040
9834.5	.00607426	-.101424

*** CONCRETE STRAIN > ULTIMATE COMPRESSIVE STRAIN OF .03953 ***

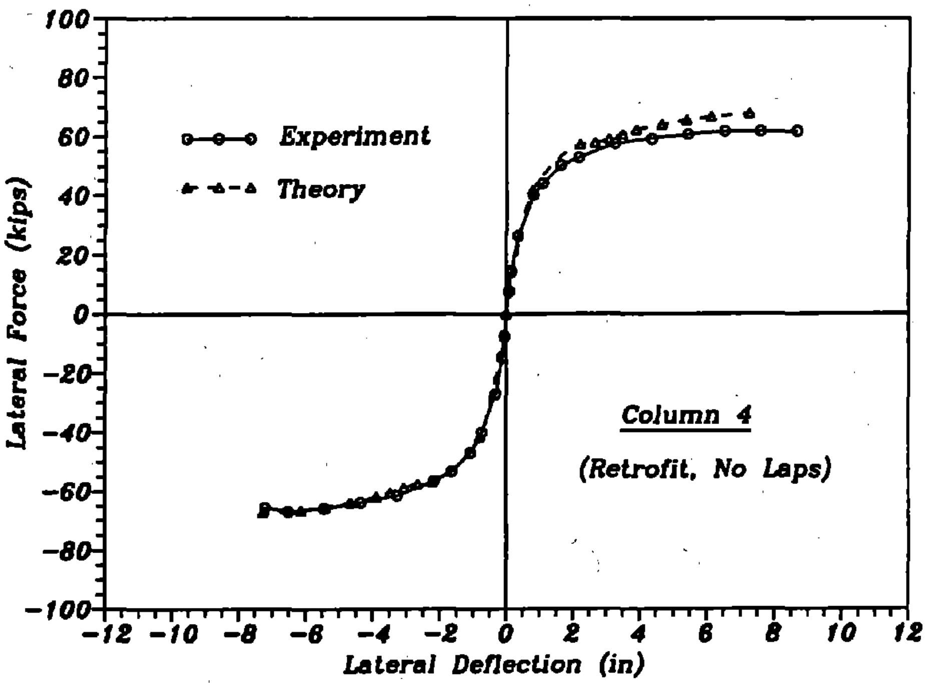

Figure 6.6: **Prediction of Envelope Curve for Column 4 (No Laps)**

steel. The ultimate compressive strain based on energy balance method (Eqn. 3.38) was $\epsilon_{cu} = 0.04$, while the extreme tension strain was $\epsilon_s = 0.1014$; $\approx 50\%$ of the actual ultimate tensile strain of the longitudinal steel. Large inelastic cyclic displacement imposed on the column during testing resulted in early fracture of the extreme longitudinal steel at a strain below the ultimate tensile strain. Even though the reduction of ductility factor is a function of the loading history, experimental testing on these columns was felt to represent well the loading conditions associated with a severe earthquake. Thus a 25% reduction of the ultimate displacement prediction by COLRET, giving a displacement of 7.323 inches, is considered adequate in providing a reasonable safety margin against this type of low-cycle fatigue fracture failure for steel jacketed columns.

The theoretical envelope curve for column 4, shown in Figure 6.6, was generated using the analytical results of Table 6.4 and 6.5. Two strategic points on the envelope curve are identified: (i) first yield of the extreme tension reinforcement, and (ii) strain-hardening of the extreme tension reinforcement. The first point corresponds to (Δ'_{yr}, V_{yr}) where Δ'_{yr} = first yield displacement, and V_{yr} = first yield column shear force. In this case, the first yield displacement was $\Delta'_{yr} = 1.046 \times 6005.6/7988.4 = 0.786$ inches. To construct the lateral strength envelope, the lateral displacements of the column in the strain-hardening range of the longitudinal reinforcement were computed using the equation:

$$\Delta = \Delta'_{yr} + (\phi - \phi'_{yr}\frac{M_p}{M_{yr}})L_{pr}(L - 0.5L_{pr}) \qquad \text{for} \quad \phi > \phi_{sh} \qquad (6.2)$$

where ϕ_{sh} = curvature at the onset of strain-hardening; other terms have previously been defined. For a strain-hardening strain of $\epsilon_{sh} = 0.022$, the corresponding curvature and moment interpolated from Figure 6.5 were $\phi_{sh} = 0.00137$ rad/in and $M_{sh} = 8367$ kip.in, respectively. Substituting $\Delta_{yr} = 1.046$ inches, $L_{pr} = 10$ inches, $L = 148.5$ inches, $\phi'_{yr} = 0.000134$ rad/in, $M_{yr} = 6005.7$ kip.in and $M_p = 7988.3$ kip.in into Eqn. 6.2 gave the lateral displacement of $\Delta_{sh} = 2.50$ inches. The corresponding lateral force was $V_{sh} = 58.0$ kips. The lateral displacements beyond Δ_{sh} were constructed using the moment-curvature results given in Table 6.5.

Figure 6.6 also shows the experimental envelope curve for test column 4 which has been corrected for the horizontal component of the axial force. The theoretical curve agrees well with the experimental envelope, especially in the pull direction. In the push direction, the prediction slightly exceeded the experimental results.

6.3.2 Column 6 - Lapped Starter Bars

The analytical predictions for column 6 reported in [1] are presented in Tables 6.6 and 6.7. It must be emphasized that the analytical prediction for the column assumes no lap failure in the column longitudinal bars. The monotonic moment-curvature curve at the base section of column 6 is shown in Figure 6.7.

The program COLRET predicted a shear force of 55.5 kips, corresponding to the plastic moment, and a shear force of 69.6 kips corresponding to an ultimate compressive strain of $\epsilon_{cu} = 0.044$ in the extreme compression fiber of the confined cover concrete. Note that the test column showed a smaller difference in the moment capacities for the two directions of loading, when compared to test column 4. The maximum experimental shear force in the pull direction was 68.4 kips, whereas the maximum shear force in the push direction was 64.2 kips; thus averaged 66.3 kips, and was less than 5% lower than the predicted ultimate shear force. Without a bond failure in the lap-splice region, column 6 behaved identical to column 4 which had continuous longitudinal reinforcement, and the prediction by COLRET was almost the same as with column 4, except for difference due to different concrete and steel jacket strengths.

The envelope curve generated by Eqn. 6.2, using the moment-curvature results in the strain-hardening range, is shown in Figure 6.8. It can be seen that a good agreement between theoretical and experimental curves is achieved for both directions of loading. The equivalent elasto-plastic yield displacement predicted by COLRET after retrofit is 1.047 inches, slightly lower than the experimental value of 1.090 inches. The ultimate displacement predicted by COLRET (based on the ultimate compressive strain of cover concrete confined by the steel jacket)

Table 6.6: **Analytical Results for Column 6 (With Laps)**

(a) Before Retrofit

```
* * * * * * * * * * * * * * * * * * * * * * * * *
*                                               *
*     UNIVERSITY OF CALIFORNIA   SAN DIEGO      *
*                                               *
*   STRENGTH AND DUCTILITY OF CIRCULAR COLUMN   *
*                                               *
*          VERSION 1.1  (MAR 1991)              *
*                                               *
* * * * * * * * * * * * * * * * * * * * * * * * *

JOB TITLE : SEISMIC RETROFIT OF BRIDGE COLUMN - COLUMN 6 (10/16/89)

                  ORIGINAL COLUMN PARAMETERS
                  **************************

                  DIAMETER OF COLUMN :     24.000 in
COLUMN HEIGHT TO PT OF CONTRAFLEXURE :     12.000 ft
                PLASTIC HINGE LENGTH :     16.020 in
                   COVER TO MAIN BAR :       .800 in

                            MAIN BAR :       # 6
                      NUMBER OF BARS :        26
        YIELD STRENGTH FOR MAIN STEEL :     45.7 ksi    (Mild Steel)
           ULTIMATE STRENGTH OF STEEL :     68.5 ksi
             YOUNGS MODULUS FOR STEEL :  29000.0 ksi
                YIELD STRAIN OF STEEL :       .00158
          STRAIN AT HARDENING OF STEEL :      .02205
             ULTIMATE STRAIN OF STEEL :       .16205

                            TIE SIZE :       # 2      (Hoops)
                         TIE SPACING :    5.000 in
                YIELD STRENGTH OF TIE :     51.0 ksi   (Mild Steel)

                   CONCRETE STRENGTH :      5.425 ksi
            CONCRETE ULTIMATE STRAIN :       .009

                   COLUMN AXIAL LOAD :    400.0 kips

                  RESULTS FOR ORIGINAL COLUMN
                  ***************************

   ORIGINAL  YIELD   MOMENT =   6024.2 kip.in
   ORIGINAL ULTIMATE MOMENT =   6946.2 kip.in

   CURVATURE AT FIRST YIELD OF EXTREME REBAR =   .00013293 Rad/in
   EQUIVALENT ELASTO-PLASTIC YIELD CURVATURE =   .00015327 Rad/in
   ULTIMATE                        CURVATURE =   .00100999 Rad/in
   CURVATURE          DUCTILITY       FACTOR =  6.6

   YIELD            DISPLACEMENT =    1.059 in
   ULTIMATE         DISPLACEMENT =    2.988 in
   DISPLACEMENT DUCTILITY FACTOR =    2.820            (FLEXURE)

               **** LIMITED DUCTILITY ****

   MAXIMUM FEASIBLE  SHEAR  FORCE =    48.2 kips ( K1 = 1.000 )
   IDEAL SHEAR STRENGTH OF COLUMN =   165.0 kips
```

Table 6.6: **Analytical Results for Column 6 (With Laps) - Cont'd**

(a) After Retrofit

```
        RETROFIT COLUMN PARAMETERS
        **************************

  OUTSIDE DIAMETER OF JACKET :      24.875 in
         THICKNESS OF JACKET :        .188 in
            LENGTH OF JACKET :      48.000 in
     JACKET TOE FROM FOOTING :       1.000 in

    YIELD STRENGTH OF JACKET :      54.000 ksi

  GROUT COMPRESSIVE STRENGTH :       1.860 ksi

 ULTIMATE COMPRESSIVE STRAIN :        .044

        PLASTIC HINGE LENGTH :      10.000 in


        RESULTS FOR RETROFIT COLUMN
        ***************************

     YIELD      MOMENT =    6004.0   kip.in
     PLASTIC    MOMENT =    7987.7   kip.in
     ULTIMATE   MOMENT =   10015.5   kip.in

CURVATURE AT FIRST YIELD OF EXTREME REBAR =   .00013415 Rad/in
EQUIVALENT ELASTO-PLASTIC YIELD CURVATURE =   .00017847 Rad/in
CURVATURE    AT    ULTIMATE     CONDITION =   .00730372 Rad/in
CURVATURE       DUCTILITY          FACTOR = 40.9

YIELD            DISPLACEMENT =    1.047 in
ULTIMATE         DISPLACEMENT =   11.011 in
DISPLACEMENT DUCTILITY FACTOR =   10.520             (FLEXURE)

PLASTIC      SHEAR       FORCE =     55.5 kips

         **** INADEQUATE JACKET LENGTH ****

**** INCREASE JACKET LENGTH TO      49.082 in ****

MAXIMUM FEASIBLE  SHEAR  FORCE =    69.6 kips ( K1 = 1.000 )
IDEAL SHEAR STRENGTH OF COLUMN =   194.8 kips
```

was $\Delta_{ur} = 11.01$ inches, and was again non-conservative. First fracture of a longitudinal bar occurred during the first push cycle to a peak displacement of 8.70 inches. A 25% reduction of the ultimate displacement predicted by COLRET gives $\Delta_u = 8.26$ inches; $\approx 5\%$ smaller than the experimental displacement when first fracture of the column longitudinal reinforcement occurred. In the absence of further experimental data, the reduction of ultimate displacement prediction by COLRET by 25% appears reasonable.

Table 6.7: Moment-Curvature Results for Column 6 (With Laps)

MOMENT CURVATURE ANALYSIS FOR RETROFIT COLUMN
****

JOB TITLE : SEISMIC RETROFIT OF BRIDGE COLUMN - COLUMN 6 (10/16/89)

MOMENT (kip.in)	CURVATURE (Rad/in)	STEEL STRAIN
309.1	.00000350	.000141
1091.3	.00001238	.000048
1807.4	.00002139	-.000047
2375.2	.00003135	-.000164
2869.3	.00004238	-.000306
3324.8	.00005368	-.000453
3762.3	.00006552	-.000613
4188.5	.00007790	-.000786
4607.7	.00009028	-.000958
5021.6	.00010266	-.001130
5430.6	.00011505	-.001302
5834.7	.00012796	-.001487
6158.1	.00014142	-.001684
6408.6	.00015596	-.001906
6619.5	.00017104	-.002139
6788.3	.00018665	-.002386
6924.7	.00020227	-.002632
7045.2	.00021789	-.002878
7144.6	.00023404	-.003136
7227.0	.00025074	-.003407
7541.0	.00033636	-.004810
7746.3	.00042631	-.006312
7894.6	.00051948	-.007887
7987.7	.00061698	-.009561
8080.6	.00071447	-.011235
8133.2	.00081627	-.013007
8183.8	.00091808	-.014779
8232.9	.00101988	-.016551
8251.8	.00112600	-.018422
8272.1	.00123212	-.020293
8319.0	.00133392	-.022065
8390.2	.00143573	-.023837
8465.2	.00153753	-.025610
8536.0	.00163934	-.027382
8619.8	.00173683	-.029056
8697.9	.00183432	-.030729
8769.5	.00193181	-.032403
8834.2	.00202930	-.034077
8909.5	.00212248	-.035652
8953.6	.00222428	-.037425
9249.6	.00276268	-.046581
9443.3	.00330107	-.055738
9587.8	.00382652	-.064599
9698.8	.00433903	-.073165
9791.9	.00483860	-.081436
9857.9	.00535112	-.090002
9912.2	.00586363	-.098568
9954.7	.00637614	-.107134
10004.8	.00687571	-.115404

*** CONCRETE STRAIN > ULTIMATE COMPRESSIVE STRAIN OF .04411 ***

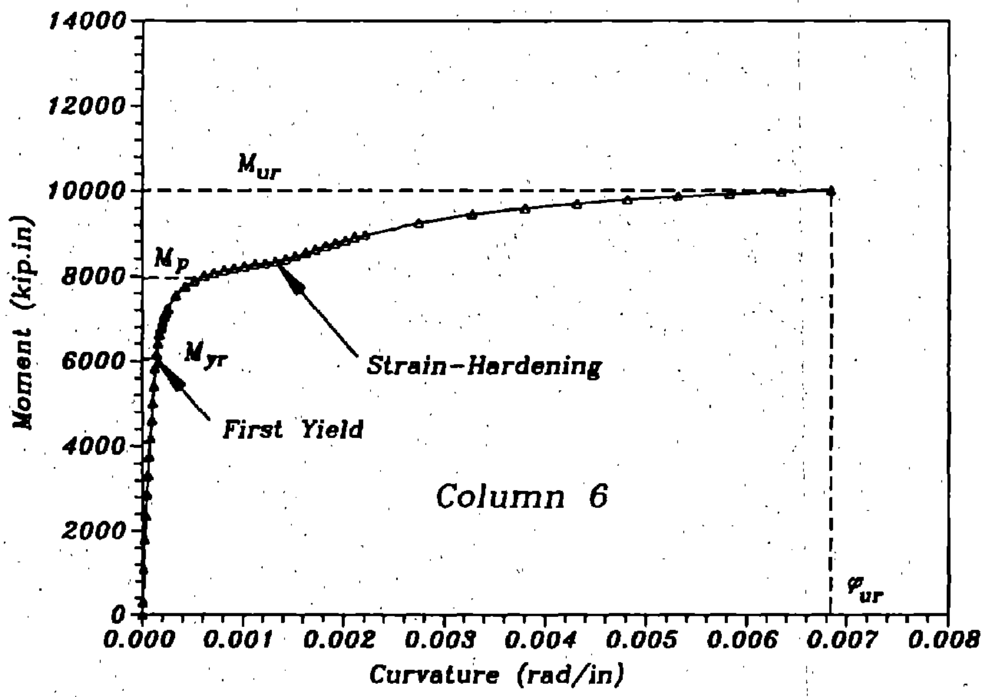

Figure 6.7: Moment-Curvature Curve for Column 6 (With Laps)

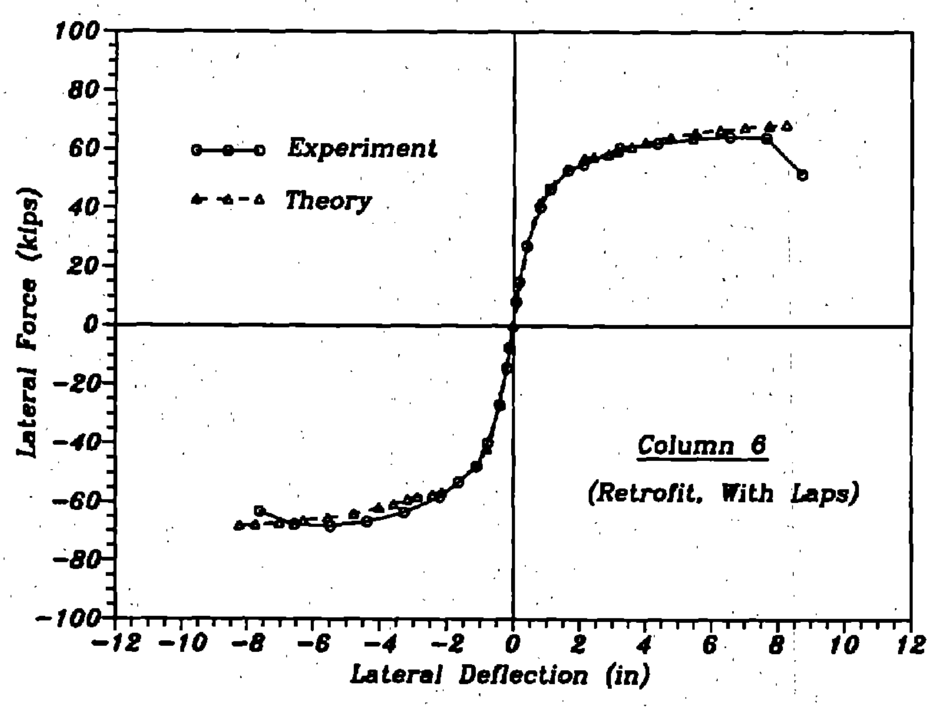

Figure 6.8: Prediction of Envelope Curve for Column 6 (With Laps)

Chapter 7

Conclusions

A simple analysis program for predicting the flexural response of circular bridge columns under seismic excitation is presented. The program provides bridge engineers a tool for assessing the seismic performance of existing bridge columns so that deficient columns can be identified for retrofit. The program is extended to steel jacketed columns so that improved performance after retrofit can be assessed.

Comparative studies indicate good agreement between prediction and experimental data for both full-scale and model bridge columns. Accurate assessment of the lateral strength for both 'as-built' and retrofitted columns can be achieved. The predictions of both elastic and post-yield deformation for the column are good. This will allow the increase in column stiffness and redistribution of forces in the bridge structure as a result of steel jacketing to be assessed. For 'as-built' columns, the prediction of ultimate displacement is conservative and is limited by the extreme concrete compressive strain. For retrofitted columns, the displacement capacity may be limited by low-cycle fatigue fracture of the longitudinal reinforcement which is dependent on the load-history. Without a knowledge of the hysteretic energy to be expected for bridge columns under a random seismic load, a reduction of the displacement capacity in retrofitted columns by 25% appears appropriate at this stage.

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Appendix A

Curvature Integration

A.1 General

The yield displacement at the center of seismic mass for retrofitted column is given by the first moment of the curvature distribution about the center of seismic mass. The integral expressed by Eqn. 4.22 depends on the jacket length and bond strength.

A.1.1 Case (i) - Long Jacket

Case (i) represents a jacket of sufficient length for the development of full composite action after transfer lengths l_t and l_b. The curvature distribution in retrofitted column in this case can be divided into regions as shown in case (i) in Figure A.1.

The first moment of each region about the center of seismic mass gives the contribution of that region to total yield displacement Δ_{yr} i.e.

$$\Delta_{yr} = \sum_{i=1}^{5} \Delta_i \tag{A.1}$$

where Δ_i denotes the contribution of the i^{th} region. In the constant flexural rigidity region, namely regions 1, 3 and 5, the curvature variation will be linear with height and follows the shape of the bending moment diagram. Thus the displacement contributions can be derived as:

$$\Delta_1 = \frac{\phi_1}{3}(L' - v_g - L_j)^2 \tag{A.2}$$

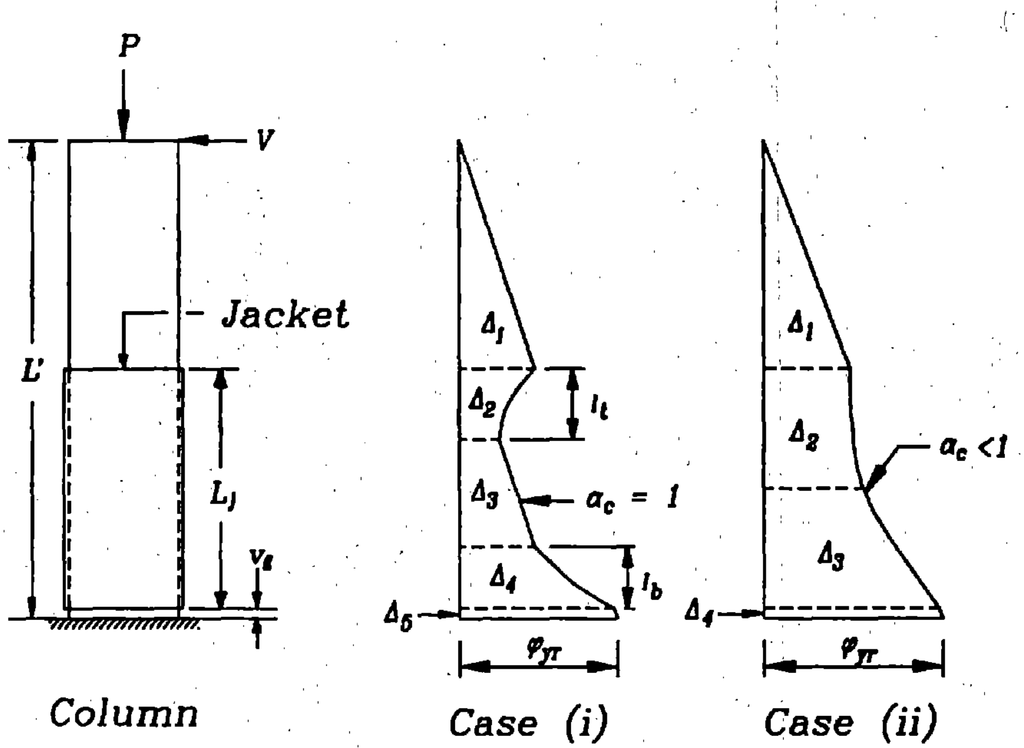

Figure A.1: **Curvature Distribution in Retrofitted Column**

$$\begin{aligned}\Delta_3 &= \frac{\phi_2}{2}(L_j - l_t - l_b)(2L' - 2v_g - L_j - l_b + l_t) + \\ &\quad \frac{(\phi_3 - \phi_2)}{6}(L_j - l_t - l_b)(3L' - 3v_g - L_j - 2l_b + l_t) \qquad (A.3)\\ \Delta_5 &= \phi_4 v_g(L' - 0.5v_g) + \frac{(\phi_5 - \phi_4)}{6}v_g(3L' - v_g) \qquad (A.4)\end{aligned}$$

The curvature variation in region 2 is however non-linear and its contribution must be integrated as follows:

$$\Delta_2 = \int_{v_g+L_j-l_t}^{v_g+L_j} \frac{M(y)}{EI(y)}(L' - y)dy \qquad (A.5)$$

By introducing $y_1 = y - v_g - L_j + l_t$ and $L_1 = L' - v_g - L_j + l_t$, Eqn. A.5 may be written as:

$$\Delta_2 = \int_0^{l_t} \phi(y_1)(L_1 - y_1)dy_1 \qquad (A.6)$$

where

$$\phi(y_1) = \frac{M(y_1)}{EI(y_1)} \tag{A.7}$$

For $0 \leq y_1 \leq l_t$,

$$M(y_1) = M_2 - V_{yr} y_1 \tag{A.8}$$

$$EI(y_1) = (EI)_c - ((EI)_c - (EI)_b)\frac{y_1}{l_t} \tag{A.9}$$

where

$$V_{yr} = \frac{M_{yr}}{L'} \tag{A.10}$$

$$M_2 = \frac{L' - v_g - L_j + l_t}{L'} M_{yr} \tag{A.11}$$

The terms V_{yr} and M_2 are the seismic shear and moment at level 2.

Eqn. A.6 may be integrated to give :

$$\begin{aligned}\frac{\Delta_2 (EI)_c}{M_2} = & -\frac{l_t L_1}{\Omega_1} ln(1 - \Omega_1) + \frac{l_t^2}{\Omega_1}(1 + \frac{V_{yr} L_1}{M_2})(1 + \frac{1}{\Omega_1} ln(1 - \Omega_1)) \\ & -\frac{V_{yr} l_t^3}{M_2 \Omega_1^3}(ln(1 - \Omega_1) + \frac{\Omega_1^2}{2} + \Omega_1)\end{aligned} \tag{A.12}$$

where

$$\Omega_1 = 1 - \frac{(EI)_b}{(EI)_c} \tag{A.13}$$

Similarly the contribution from region 4 can be written as :

$$\Delta_4 = \int_{v_g}^{v_g + l_b} \frac{M(y)}{EI(y)}(L' - y)dy \tag{A.14}$$

Again by introducing the variables $y_2 = y - v_g$ and $L_2 = L' - v_g$, Eqn. A.14 may be written as:

$$\Delta_4 = \int_0^{l_b} \phi(y_2)(L_2 - y_2)dy_2 \tag{A.15}$$

where

$$\phi(y_2) = \frac{M(y_2)}{EI(y_2)} \tag{A.16}$$

For $0 \leq y_2 \leq l_b$,

$$M(y_2) = M_4 - V_{yr}y_2 \tag{A.17}$$

$$EI(y_2) = (EI)_b + ((EI)_c - (EI)_b)\frac{y_2}{l_b} \tag{A.18}$$

where

$$M_4 = \frac{L' - v_g}{L'}M_{yr} \tag{A.19}$$

Integrating Eqn. A.15 gives:

$$\begin{aligned} \frac{\Delta_4(EI)_b}{M_4} &= \frac{l_b L_2}{\Omega_2}ln(1+\Omega_2) - \frac{l_b^2}{\Omega_2}(1+\frac{V_{yr}L_2}{M_4})(1-\frac{1}{\Omega_2}ln(1+\Omega_2)) \\ &\quad + \frac{V_{yr}l_b^3}{M_4\Omega_2^3}(ln(1+\Omega_2)+\frac{\Omega_2^2}{2}-\Omega_2) \end{aligned} \tag{A.20}$$

where

$$\Omega_2 = \frac{(EI)_c}{(EI)_b} - 1 \tag{A.21}$$

$$= \frac{\Omega_1}{\Omega_1 - 1} \tag{A.22}$$

A.1.2 Case(ii) - Short Jacket

Case (ii) represents a jacket of insufficient length for development of full composite action. Curvature distribution in this case consists of 4 regions shown in case (ii) in Figure A.1 and the integral expression in Eqn. A.1 can now be rewritten as :

$$\Delta_{yr} = \sum_{i=1}^{4} \Delta_i \tag{A.23}$$

where Δ_i again denotes the contribution of the i^{th} region. The displacement contribution from the linear regions of the curvature diagram are:

$$\Delta_1 = \frac{\phi_1}{3}(L' - v_g - L_j)^2 \quad \text{(A.24)}$$

$$\Delta_4 = \phi_3 v_g(L' - 0.5v_g) + \frac{(\phi_4 - \phi_3)}{6} v_g(3L' - v_g) \quad \text{(A.25)}$$

The contribution from region 2 of case (ii) may be written as:

$$\Delta_2 = \int_{v_g+0.5L_j}^{v_g+L_j} \frac{M(y)}{EI(y)}(L' - y)dy \quad \text{(A.26)}$$

By introducing $y_1 = y - v_g - 0.5L_j$ and $L_1 = L' - v_g - 0.5L_j$, Eqn. A.26 may be written as:

$$\Delta_2 = \int_0^{0.5L_j} \phi(y_1)(L_1 - y_1)dy_1 \quad \text{(A.27)}$$

where

$$\phi(y_1) = \frac{M(y_1)}{EI(y_1)} \quad \text{(A.28)}$$

For $0 \leq y_1 \leq 0.5L_j$,

$$M(y_1) = M_m - V_{yr}y_1 \quad \text{(A.29)}$$

$$EI(y_1) = (EI)_m - ((EI)_m - (EI)_b)\frac{2y_1}{L_j} \quad \text{(A.30)}$$

where

$$M_m = \frac{L' - v_g - 0.5L_j}{L'} M_{yr} \quad \text{(A.31)}$$

The terms M_m represents the moment at mid-height of the jacket and $(EI)_m$ is the mid-height flexural rigidity.

Carrying out the integration gives:

$$\frac{\Delta_2(EI)_m}{M_m} = -\frac{L_jL_1}{2\Omega_{m1}} ln(1 - \Omega_{m1}) + \frac{L_j^2}{4\Omega_{m1}}(1 + \frac{V_{yr}L_1}{M_m})(1 + \frac{1}{\Omega_{m1}} ln(1 - \Omega_{m1})) - \frac{V_{yr}L_j^3}{8M_m\Omega_{m1}^3}(ln(1 - \Omega_{m1}) + \frac{\Omega_{m1}^2}{2} + \Omega_{m1}) \quad \text{(A.32)}$$

where

$$\Omega_{m1} = 1 - \frac{(EI)_b}{(EI)_m} \tag{A.33}$$

Displacement contribution from region 3 is obtained from :

$$\Delta_3 = \int_{v_g}^{v_g+0.5L_j} \frac{M(y)}{EI(y)}(L' - y)dy \tag{A.34}$$

Using the variables $y_2 = y - v_g$ and $L_2 = L' - v_g$, Eqn. A.34 may be written as:

$$\Delta_3 = \int_0^{0.5L_j} \phi(y_2)(L_2 - y_2)dy_2 \tag{A.35}$$

where

$$\phi(y_2) = \frac{M(y_2)}{EI(y_2)} \tag{A.36}$$

For $0 \leq y_2 \leq 0.5L_j$,

$$M(y_2) = M_3 - V_{yr}y_2 \tag{A.37}$$

$$EI(y_2) = (EI)_b + ((EI)_m - (EI)_b)\frac{2y_2}{L_j} \tag{A.38}$$

where

$$M_3 = \frac{L' - v_g}{L'}M_{yr} \tag{A.39}$$

Integrating Eqn. A.35 gives:

$$\begin{aligned} \frac{\Delta_3(EI)_b}{M_3} &= \frac{L_jL_2}{2\Omega_{m2}}ln(1 + \Omega_{m2}) - \frac{L_j^2}{4\Omega_{m2}}(1 + \frac{V_{yr}L_2}{M_3})(1 - \frac{1}{\Omega_{m2}}ln(1 + \Omega_{m2})) \\ &+ \frac{V_{yr}L_j^3}{8M_3\Omega_{m2}^3}(ln(1 + \Omega_{m2}) + \frac{\Omega_{m2}^2}{2} - \Omega_{m2}) \end{aligned} \tag{A.40}$$

where

$$\Omega_{m2} = \frac{(EI)_m}{(EI)_b} - 1 \tag{A.41}$$

$$= \frac{\Omega_{m1}}{\Omega_{m1} - 1} \tag{A.42}$$

Appendix B

User Guides

B.1 Preliminary

The program COLRET is written in standard Fortran 77 to be executed on an IBM personal computer. Two output files by name of COLRET.OUT and CURVAT.OUT will be automatically created in the directory where the program is residing. Summaries of strength and ductility for the column will be directed to file COLRET.OUT. Results from moment-curvature analysis for the retrofit column will be contained in file CURVAT.OUT.

B.2 Input Format

Screen input is assumed by the program and data may be entered in free format. Data may also be read from a file by using the following **DOS** command: COLRET<Data_file.

The following column parameters are required by the program:

CARD 1 : Job Title

A maximum of 60 characters is allowed for the job title.

CARD 2 : Retrofit Prompt (Yes or No)

If retrofit analysis of column is required, enter **Yes**. Otherwise, enter **No**, and the strength and ductility of the 'as-built' column will be computed.

CARD 3 : Column Diameter

Nominal column diameter in inches.

CARD 4 : Column Height

Column height is defined as the distance from a fixed base to the point of contraflexure in the column. Height is to be given in feet.

CARD 5 : Concrete Cover

Clear concrete cover to longitudinal steel in inches.

CARD 6 : Longitudinal Bar Size

Only U.S. reinforcing bar sizes are permitted. Acceptable sizes are #2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 14 and 18.

CARD 7 : Number Of Longitudinal Bars

Total longitudinal steel area to be given in terms of the number of bars used. Uniform distribution of longitudinal bars are assumed.

CARD 8 : Longitudinal Steel Type

Two types of longitudinal steel may be used, Grade 40 or 60.

Grade 40 steel is characterized by a long yield plateau in its stress strain curve. The on-set of strain hardening for Grade 40 steel is assumed to be 14 times that of its yield strain. Ultimate strain is taken as 14% strain plus the strain at strain-hardening.

Grade 60 steel is assumed to experience strain-hardening at an earlier stage of 5 times the yield strain. Ultimate strain is assumed to occur at 12%.

For Grade 40 steel, enter 1.

For Grade 60 steel, enter 2.

CARD 9 : Yield Strength For Longitudinal Steel

The yield strength of the longitudinal steel in ksi.

CARD 10 : Transverse Steel Type

Two types of transverse steel may be used, spiral or circular hoops. Sufficient lap length is assumed for the circular hoop so that full yield strength can be developed.

For spiral, enter 1.

For circular hoop, enter 2.

CARD 11 : Grade of Steel for Transverse Reinforcement

Hoops or spirals may be Grade 40 or 60 steel as that described for longitudinal steel in CARD 8.

For Grade 40 steel, enter 1.

For Grade 60 steel, enter 2.

CARD 12 : Transverse Steel Size

The same set of U.S. bar sizes described in CARD 6 for longitudinal steel.

CARD 13 : Tie Spacing

Tie spacing at the critical section to be given in inches. For spiral, tie spacing is defined by the centerline pitch. For circular hoop, tie spacing is defined by the centerline distance between adjacent hoops.

CARD 14 : Transverse Steel Yield Strength

Yield strength of transverse steel in ksi. No strain-hardening is assumed for transverse steel.

CARD 15 : Concrete Strength

Uniaxial compressive strength of concrete to be given in ksi.

CARD 16 : Shear Enhancement Factor k_1

Capacity design requires a realistic estimation of the maximum feasible shear force so that the column can be designed to safeguard against brittle shear failure. The maximum shear force V_D is obtained by multiplying the shear force at the ideal flexural capacity by the shear enhancement factor. The use of this factor should reflect the increase in shear demand as a result of material overstrength, excess reinforcement and higher mode effects when the program is applied to longitudinal response.

The user is required to enter a value for the shear enhancement factor k_1. In the absence of information on the actual material strengths and reinforcement provided, a default value of 1.15 is deemed satisfactory for possible shear force enhancement.

CARD 17 : Axial Load

Column axial load to be given in kips. Compression is positive.

ADDITIONAL INPUT IF RETROFIT IS REQUIRED

CARD 18 : Jacket Diameter

Outside diameter of the steel jacket is to be given in inches. The steel jacket is assumed to be uniform in diameter.

CARD 19 : Jacket Thickness

Thickness of the steel jacket to be given in inches.

CARD 20 : Jacket Length

Jacket length represents the extent of retrofit in column. The jacket length is to be in inches.

CARD 21 : Vertical Gap of Jacket from Footing

A vertical gap is assumed to exist between the jacket toe and top of column base. Diameter of the column at the critical section thus remains unchanged.

CARD 22 : Yield Strength of Jacket

Yield strength of steel jacket to be given in ksi. The jacket is assumed to provide only confinement to the concrete at the critical section. No strain-hardening is used.

CARD 23 : Grout Strength

Uniaxial compressive strength of grout infill in ksi. A cement-based grout is assumed.

Appendix C

Program Listing - **COLRET**

```
C
C     *****************
C     *  MAIN PROGRAM *
C     *****************
C
      INTEGER NBAR,BSIZ,TSIZ,TTYPE,LTYPE,GTIE,COUNTER,FLAGR
      REAL PI,DIACOL,COVER,DLBAR,TOTAS,DHP,SHP,SSHP,FY,FYH,FCU,AOLD,
     *     OLDCORE,ASECTION,ASECORE,Y,Z,SLCORE,DEFCORE,KE,SLCOVER,
     *     DCORE,THETAOLD,ETOP,EBOTT,YSTEEL,ESPALL,X,FS,MBAL,PBAL,
     *     PTEN,PTENALL,PAXIAL,P,MOM,MYIELD,ELAST,PBALR,MULT,RDELTAP,
     *     CURYIELD,MCURVY,CURVULT,ELAST1,ELAST2,LP,MBALR,DELTAU,
     *     DELTAY,MU,TAREA,ESTART,MOMLAST,DELTAE,DELTAP,CURTEMP,
     *     PEAKMOM,PEAKCUR,HEIGHT,HEIGH1,MULTR,LJACK,CASH,FGU,REDELY,
     *     FYJOLD,ALPHA,ECMAX,EULT,CURVRU,LPR,REPU,REDELU,REPMAX,
     *     MRECURYIELD,MSU,PSU,SLOPE1,SLOPE2,ETEMP,RU,RC,MUS,MTEMP,K1,
     *     LJMIN,FACTJ
      REAL ASLICE(100),CSLICE(100),ASLCOV(100),ASLCORE(100),AS(100),
     *     THETA(100),ESTRAIN(100),FCOVER(100),FCORE(100),FSTEEL(100),
     *     BDIA(18),BAREA(18),MOMEN(50),CURVA(50),CCSLICE(100),
     *     AJACK(100),AGROUT(100),CESTRAIN(100),ACSLICE(100),
     *     ACOLSL(100),FGROUT(100),FJACK(100)
      CHARACTER TITLE*60,PROMPT*3
      DATA PI,N,NCOVER,NCORE,YSTEEL/3.141592654,100,5,90,29000./
C
C     U.S. BAR SIZES (No. 2 TO 18)
C
      DATA BDIA(2),BDIA(3),BDIA(4),BDIA(5),BDIA(6),BDIA(7),BDIA(8),
     *     BDIA(9),BDIA(10),BDIA(11),BDIA(14),BDIA(18)/0.25,0.375,
     *     0.5,0.625,0.75,0.875,1.0,1.128,1.27,1.41,1.693,2.257/
C
C     BAR AREA SQUARE INCHES
C
      DATA BAREA(2),BAREA(3),BAREA(4),BAREA(5),BAREA(6),BAREA(7),
     *     BAREA(8),BAREA(9),BAREA(10),BAREA(11),BAREA(14),BAREA(18)
     *     /0.05,0.11,0.20,0.31,0.44,0.60,0.79,1.0,1.27,1.56,2.25,4.0/
C
C     ****************************
C     *   DATA INPUT FROM SCREEN  *
C     ****************************
C
      WRITE(*,1010)
      WRITE(*,1012)
      WRITE(*,1015)
      WRITE(*,1012)
      WRITE(*,1012)
      WRITE(*,1011)
      WRITE(*,1012)
      WRITE(*,1013)
      WRITE(*,1012)
      WRITE(*,1010)
      WRITE(*,1990)
1990  FORMAT(/,1X,'Job Title ( Less Than 60 Characters )')
      READ (*,1091) TITLE
1091  FORMAT(A60)
1894  WRITE(*,*)'Retrofit of Column (Yes or No)'
      READ(*,1792)PROMPT
1792  FORMAT(A3)
      IF(PROMPT.EQ.'Y  '.OR.PROMPT.EQ.'YES'.OR.PROMPT.EQ.'y  '.
     *   OR.PROMPT.EQ.'yes'.OR.PROMPT.EQ.'Yes')GO TO 1892
      IF(PROMPT.EQ.'N  '.OR.PROMPT.EQ.'NO '.OR.PROMPT.EQ.'n  '.
     *   OR.PROMPT.EQ.'no '.OR.PROMPT.EQ.'No ')GO TO 1893
      WRITE(*,*)'Unacceptable Answer'
      GO TO 1894
C
C     FLAGR = 2 FOR RETROFIT ANALYSIS
C
1892  FLAGR = 1
      GO TO 1895
1893  FLAGR = 2
1895  WRITE(*,1016)
1016  FORMAT(/,1X,'Input Column Data',/,1X,17('*'),/)
      WRITE(*,*) 'Column Diameter (in)'
      READ (*,*) DIACOL
      WRITE(*,*) 'Column Height to Point of Contraflexure (ft)'
      READ (*,*) HEIGH1
      WRITE(*,*) 'Cover to Main Bar (in)'
      READ (*,*) COVER
1345  WRITE(*,*) 'Long Bar Size'
      READ (*,*) BSIZ
      IF (BSIZ.LT.2.OR.BSIZ.GT.18.OR.BSIZ.EQ.12.OR.BSIZ.EQ.13
     *    .OR.BSIZ.EQ.15.OR.BSIZ.EQ.16.OR.BSIZ.EQ.17) THEN
         WRITE(*,*)'Size Unacceptable, Enter Again'
         WRITE(*,*)
         GO TO 1345
      ELSE
         DLBAR = BDIA(BSIZ)
         WRITE(*,*) 'No. of Bars'
         READ (*,*) NBAR
C
C     TOTAS = TOTAL MAIN STEEL AREA
C
         TOTAS = NBAR*BAREA(BSIZ)
      END IF
C
C     EFFECTIVE COLUMN HEIGHT = ACTUAL HEIGHT + YIELD PENETRATION
C     ASSUME YIELD PENETRATION = 6 * LONG BAR DIAMETER
C
      HEIGHT = HEIGH1 + 6.0*DLBAR/12.0
801   WRITE(*,*) 'Main Steel Type (Grade 40 = 1, Grade 60 = 2)'
      READ (*,*) LTYPE
      IF(LTYPE.NE.1.AND.LTYPE.NE.2) GO TO 801
      WRITE(*,*) 'Yield Strength for Main Steel (ksi)'
      READ (*,*) FY
802   WRITE(*,*) 'Transverse Steel Type (Spiral = 1, Hoop = 2)'
      READ (*,*) TTYPE
      IF(TTYPE.NE.1.AND.TTYPE.NE.2) GO TO 802
      IF(TTYPE.EQ.1) THEN
8113  WRITE(*,*)'Grade of Steel for Spiral (Grade 40 = 1, Grade 60',
     *           ' = 2)'
      READ (*,*)GTIE
```

```
C
C     GTIE = 1 FOR GRADE 40
C     GTIE = 2 FOR GRADE 60
C
      IF(GTIE.NE.1.AND.GTIE.NE.2) GO TO 8113
      ELSE
8114  WRITE(*,*)'Grade of Steel for Hoop (Grade 40 = 1, Grade 60',
     *          ' = 2)'
      READ (*,*)GTIE
      IF(GTIE.NE.1.AND.GTIE.NE.2) GO TO 8114
      END IF
1346  WRITE(*,*) 'Tie Size'
      READ (*,*) TSIZ
      IF(TSIZ.LT.2.OR.TSIZ.GT.18.OR.TSIZ.EQ.12.OR.TSIZ.EQ.13.OR.
     *   TSIZ.EQ.15.OR.TSIZ.EQ.16.OR.TSIZ.EQ.17) THEN
         WRITE(*,*)'Tie Size Unacceptable, Enter Again'
         WRITE(*,*)
         GO TO 1346
      ELSE
C
C     DHP   = CROSS-SECTION DIAMETER OF HOOP OR SPIRAL
C     TAREA = CROSS-SECTION AREA OF HOOP OR SPIRAL
C
           DHP   = BDIA(TSIZ)
           TAREA = BAREA(TSIZ)
      END IF
      WRITE(*,*) 'Tie Spacing (in)'
      READ (*,*) SHP
      WRITE(*,*) 'Yield Strength for Transverse Steel (ksi)'
      READ (*,*) FYH
      WRITE(*,*) 'Concrete Compressive Strength (ksi)'
      READ (*,*) FCU
      WRITE(*,*)'Shear Enhancement Factor (Suggested Value = 1.15)'
      READ (*,*) K1
      WRITE(*,*) 'Column Axial Load in kips (Compression = +ve)'
      READ (*,*) PCOL
      IF(FLAGR.EQ.2) GO TO 1042
      WRITE(*,*) 'Outside Diameter of Jacket (in)'
      READ (*,*) DJACK
      WRITE(*,*) 'Thickness of Jacket (in)'
      READ (*,*) TJACK
      WRITE(*,*) 'Length of Jacket (in)'
      READ (*,*) LJACK
      WRITE(*,*) 'Height of Jacket Toe from Footing (in)'
      READ (*,*) CASH
      WRITE(*,*) 'Yield Strength of Jacket (ksi)'
      READ (*,*) FYJ
      WRITE(*,*) 'Grout Compressive Strength (ksi)'
      READ (*,*) FGU
C
C     CONVERT GROUT STRENGTH TO PSI
C
      FGU = 1000.0*FGU
C
C     OPENING OUTPUT FILES: COLRET.OUT AND CURVAT.OUT
C
C     DIRECTING  SUMMARY OF RESULTS TO FILE COLRET.OUT
C     DIRECTING  MOMENT CURVATURE RESULTS TO CURVAT.OUT
C
1042  OPEN (UNIT=11,FILE='COLRET.OUT',STATUS='UNKNOWN')
      OPEN (UNIT=12,FILE='CURVAT.OUT',STATUS='UNKNOWN')
      WRITE(*,321)
321   FORMAT(//,21X,'PROGRAM RUNNING',/,21X,15('*'))
C
C     INITIALISES VECTORS
C
      DO 50 I=1,N,1
C
C     THETA(I)  = SUBTENDED ANGLE ASSOCIATED WITH I-SLICE
C     AS(I)     = AREA OF I-SLICE
C     FCOVER(I) = STRESS IN COVER CONCRETE FOR I-SLICE
C     FCORE(I)  = STRESS IN CORE CONCRETE FOR I-SLICE
C     FSTEEL(I) = STEEL STRESS ASSOCIATED WITH I-SLICE
C
              THETA(I)   = 0.0
              AS(I)      = 0.0
              FCOVER(I)  = 0.0
              FCORE(I)   = 0.0
              FSTEEL(I)  = 0.0
50    CONTINUE
C
C     ***************************************************************
C     *     ASSUMED STRAIN-HARDENING PROPERTY FOR LONG STEEL        *
C     ***************************************************************
C
C     TENSILE STRENGTH FSU = 1.5*FY FOR BOTH GRADES OF STEEL
C     STRAIN-HARDENING ESH = 14*EY  FOR GRADE 40 STEEL
C                      ESH = 5.0*EY FOR GRADE 60 STEEL
C     ULTIMATE STRAIN  ESU = 0.14 + ESH FOR GRADE 40 STEEL
C                      ESU = 0.12   FOR GRADE 60 STEEL
C     YOUNG'S MODULUS  YSTEEL = 29000. KSI
C
C     EYIELD =  YIELD STRAIN
C
      EYIELD = -FY/YSTEEL
      EY     = -EYIELD
      FSU    = 1.5*FY
      IF (LTYPE.EQ.1) THEN
              ESH = 14.0*EY
              ESU = 0.14 + ESH
      ELSE
              ESH = 5.0*EY
              ESU = 0.12
      END IF
C
C     **************************************
C     *     CRITICAL SECTION PARAMETERS    *
C     **************************************
C
C     SSHP    = CLEAR SPACING OF BETWEEN HOOPS
```

```
C     DCORE    = CORE OF COLUMN MEASURED TO CENTER OF HOOP
C     N        = NO OF SLICES = 2*NCOVER + NCORE
C     NCOVER   = NO OF SLICES IN COVER CONCRETE
C     NCORE    = NO OF SLICES IN COLUMN CORE
C     SLCOVER  = SLICE THICKNESS IN COVER CONCRETE
C     CSLICE   = CENTER OF SLICE
C     ASLICE   = AREA OF SLICE
C
      DCORE    = DIACOL-2*COVER+DHP
      SSHP     = SHP-DHP
      SLCOVER  = (DIACOL-DCORE)/NCOVER/2.0
      SLCORE   = DCORE/NCORE
C
C     SLICING COLUMN SECTION INTO SEGMENTS
C
      CALL SLICE(NCOVER,CSLICE,SLCOVER,ASLICE,ASLCORE,ASLCOV,NCORE,
     *           SLCORE,DIACOL,DCORE,N)
C
C     ASSIGNING STEEL AREA TO EACH SEGMENT
C
      CALL ASSIGN(N,NCOVER,NCORE,THETA,AS,SLCORE,DIACOL,DCORE,
     *            COVER,DLBAR,TOTAS,PI)
C
C     SLICING OF STEEL JACKET AND GROUT RING FOR COMPOSITE ANALYSIS
C
      IF(FLAGR.EQ.2)GO TO 5836
      CALL COMSLICE(TJACK,CCSLICE,AJACK,AGROUT,DJACK,DIACOL,N,
     *            ACSLICE,ACOLSL)
C
C     *********************
C     *  AS-BUILT COLUMN  *
C     *********************
C
C     PARAMETERS FOR MANDER'S MODEL FOR AS-BUILT COLUMN
C
C     RHOCC    = STEEL TO CORE AREA RATIO
C     RHOS     = TRANSVERSE STEEL VOLUMETRIC RATIO
C     KE       = CONFINEMENT EFFECTIVENESS COEFFICIENT
C     FLU      = CONFINING PRESSURE FOR AS-BUILT COLUMN
C     ECU      = STRAIN AT UNCONFINED PEAK COMPRESSIVE STRESS
C     FCCU     = AS-BUILT CONFINED COMPRESSIVE STRENGTH
C     ECCU     = STRAIN AT CONFINED PEAK COMPRESSIVE STRESS
C     REXPT    = R FACTOR FOR DETERMINING ECCU IN MANDER'S MODEL
C     YSEC     = SECANT MODULUS FOR CONFINED CONCRETE
C     YCONC    = TANGENT MODULUS OF UNCONFINED CONCRETE
C     RC       = RATIO OF TANGENT TO DIFFERENCE OF TANGENT AND SECANT
C                MODULI FOR CONFINED CONCRETE
C     RU       = RATIO OF TANGENT TO DIFFERENCE OF TANGENT AND SECANT
C                MODULI FOR UNCONFINED CONCRETE
5836  RHOCC    = 4.0*TOTAS/DCORE**2.0/PI
      RHOS     = 4.0*TAREA/DCORE/SHP
      KE       = (1.0-0.5*SSHP/DCORE)**ITYPE/(1.0-RHOCC)
      FLU      = 0.5*KE*RHOS*FYH
      ECU      = 0.002
      FCCU     = FCU*(2.254*SQRT(1.0+7.94*FLU/FCU)-2.0*FLU/FCU-1.254)
      REXPT    = 5.0
      ECCU     = (REXPT*(FCCU/FCU-1.0)+1.0)*ECU
      YSEC     = FCCU/ECCU
      YCONC    = 1904.158*SQRT(FCU)
      RC       = YCONC/(YCONC-YSEC)
      RU       = YCONC/(YCONC-FCU/ECU)
C
C     ULTIMATE CONCRETE STRAIN BY PROPOSED EQUATIONS USING
C     ENERGY BALANCE METHOD
C
C     ESPALL   = ASSUMED SPALLING STRAIN IN UNCONFINED CONCRETE
C
      ESPALL   = 0.005
C
C
      IF(GTIE.EQ.1) THEN
C
C     GRADE 40 TIE
C
      ALPHA1   = 2000.0*RHOS/((1.0+(1428.0*RHOS)**4.0)**0.25)
C
C     ESMAX    = STEEL STRAIN AT ULTIMATE STRESS FSU
C
      ESMAX    = 0.14+14.0*FY/YSTEEL
      ELSE
C
C     GRADE 60 TIE
C
      ALPHA1   = 2000.0*RHOS/((1.0+(1480.0*RHOS)**2.5)**0.4)
C
C     ESMAX    = STEEL STRAIN AT ULTIMATE STRESS FSU
C
      ESMAX    = 0.12
      END IF
C
C     ESPAL1 = ULTIMATE COMPRESSIVE STRAIN FOR CONFINED CONCRETE
C
      ESPAL1 = ESPALL + ALPHA1*RHOS*FYH*ESMAX/FCCU
C
C     PARAMETERS FOR STRENGTH AND DUCTILITY
C
C     LP = PLASTIC HINGE LENGTH IN INCHES
C     LP = 0.08*COLUMN HEIGHT + 6.0*LONG. COL. BAR DIAMETER
C
      LP = 0.08*HEIGHT*12.0 + 6.0*DLBAR
C
C     COMPUTE TENSION CAPACITY OF COLUMN AT FIRST YIELD OF MAIN STEEL
C
C     PTEN     = TENSION CAPACITY OF COLUMN AT YIELD OF MAIN STEEL
C     PTENALL  = ALLOWABLE TENSION CAPACITY (ASSUMED 75%)
C
      PTEN     = -TOTAS*FY
      PTENALL  = 0.75*PTEN
C
C     CHECK IF MAGNITUDE OF COLUMN TENSION IS LESS THAN
```

```
C     75% OF COLUMN TENSION CAPACITY
C
      IF(PCOL.GT.PTENALL) GO TO 101
      DO 18 I=1,4,1
      WRITE(*,*)
18    CONTINUE
      GO TO 1008
C
C     COMPUTE AXIAL COMPRESSIVE STRENGTH OF AS-BUILT COLUMN
C
C     X = NORMALIZED STRAIN
C
101   X       = ESPALL/ECU
      ES      = ESPALL
      CALL COVERFC(X,FCU,RU,ESPALL,ECU,FCOV)
      CALL COREFC(X,FCCU,RC,FCC)
      CALL REBAR(ES,YSTEEL,FY,ESH,ESU,FSU,FS,EY)
      PAXIAL  = TOTAS*FS+PI*(DIACOL**2.0-DCORE**2.0)/4.0*FCOV
     *          +(PI*DCORE**2.0/4.0-TOTAS)*FCC
C
C     CHECK IF COLUMN AXIAL LOAD IS GREATER THAN COMPRESSION CAPACITY
C
      IF(PCOL.LT.PAXIAL) GO TO 212
      DO 19 I=1,4,1
      WRITE(*,*)
19    CONTINUE
      GO TO 1008
C
C     AT BALANCED STATE - EXTREME STEEL REACHING YIELD AND
C     COVER CONCRETE REACHING ULTIMATE STRAIN SIMULTANEOUSLY
C
C     ETOP  = EXTREME COMPRESSIVE FIBER STRAIN
C     EBOTT = EXTREME TENSION STEEL STRAIN
C
212   ETOP    = ESPAL1
      EBOTT   = EYIELD
      CALL STRAIN(N,CSLICE,ETOP,EBOTT,DIACOL,COVER,DLBAR,DHP,ESTRAIN)
      CALL STRESS(N,ESTRAIN,ECU,ECCU,FCU,FCCU,RU,RC,ESPALL,FCOVER,
     *            FCORE,YSTEEL,FY,ESH,ESU,FSU,EY,FSTEEL)
      CALL FORCE(P,FCOVER,FCORE,FSTEEL,ASLCOV,ASLCORE,AS,N)
      CALL MOMENT(MOM,FCOVER,FCORE,FSTEEL,ASLCOV,ASLCORE,AS,CSLICE,N,
     *            DIACOL)
C
C     PBAL = COLUMN AXIAL FORCE AT BALANCE CONDITION
C     MBAL = MOMENT AT BALANCE CONDITION
C
      PBAL    = P
      MBAL    = MOM
C
C     FIRST YIELD STATE (EXTREME STEEL REACHES YIELD)
C
      IF(PCOL.GT.PBAL) GO TO 777
      WRITE(*,322)
322   FORMAT(/,10X,'- First Yield Moment For Original Column')
C
C     ASSUME CONVERGENCE OF SOLUTION IF DELTA P < 0.05% OF BALANCED
C     AXIAL LOAD
C
C     SOLUTION FAILS TO CONVERGE IF NUMBER OF ITEATIONS EQUAL TO 100
C
      COUNTER = 0
C
C     DELTAP  = TOLERANCE LIMIT ON COLUMN AXIAL FORCE FOR CONVERGENCE
C
      DELTAP  = 0.0005*PBAL
      EBOTT   = EYIELD
C
C     ELAST1, ELAST2 = TEMPORARY STRAIN VARIABLES
C
      ELAST1  = EYIELD
      ELAST2  = ESPAL1
111   ETOP    = (ELAST1+ELAST2)/2.0
      CALL STRAIN(N,CSLICE,ETOP,EBOTT,DIACOL,COVER,DLBAR,DHP,ESTRAIN)
      CALL STRESS(N,ESTRAIN,ECU,ECCU,FCU,FCCU,RU,RC,ESPALL,FCOVER,
     *            FCORE,YSTEEL,FY,ESH,ESU,FSU,EY,FSTEEL)
      CALL FORCE(P,FCOVER,FCORE,FSTEEL,ASLCOV,ASLCORE,AS,N)
      COUNTER = COUNTER + 1
      IF(COUNTER.GT.100) GO TO 998
      IF(ABS(P-PCOL).GT.DELTAP) THEN
            IF(P.GE.PCOL) THEN
                  ELAST2= ETOP
            ELSE
                  ELAST1 = ETOP
            END IF
            GO TO 111
      ELSE
            CALL MOMENT(MOM,FCOVER,FCORE,FSTEEL,ASLCOV,ASLCORE,AS,
     *                  CSLICE,N,DIACOL)
      END IF
      MYIELD        = MOM
C
C     FIRST YIELD CURVATURE
C
      CURYIELD      = (ETOP-EBOTT)/(DIACOL-COVER-DLBAR/2.0)
C
C     ULTIMATE CONDITION FOR COLUMN LOAD < BALANCED AXIAL LOAD
C
      WRITE(*,323)
323   FORMAT(/,10X,'- Ultimate Moment For Original Column')
C
C     SET INITIAL VALUES OF STRAINS
C
      ETOP    = ESPAL1
      EBOTT   = 2.0*EYIELD
2004  CALL STRAIN(N,CSLICE,ETOP,EBOTT,DIACOL,COVER,DLBAR,DHP,ESTRAIN)
      CALL STRESS(N,ESTRAIN,ECU,ECCU,FCU,FCCU,RU,RC,ESPALL,FCOVER,
     *            FCORE,YSTEEL,FY,ESH,ESU,FSU,EY,FSTEEL)
      CALL FORCE(P,FCOVER,FCORE,FSTEEL,ASLCOV,ASLCORE,AS,N)
      IF(P.LT.PCOL) GO TO 2005
      EBOTT   = 2.0*EBOTT
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      GO TO 2004
2005  COUNTER = 0
      DELTAP  = 0.0005*PBAL
      ELAST1  = EYIELD
      ELAST2  = EBOTT
222   EBOTT   = (ELAST1+ELAST2)/2.0
      CALL STRAIN(N,CSLICE,ETOP,EBOTT,DIACOL,COVER,DLBAR,DHP,ESTRAIN)
      CALL STRESS(N,ESTRAIN,ECU,ECCU,FCU,FCCU,RU,RC,ESPALL,FCOVER,
     *            FCORE,YSTEEL,FY,ESH,ESU,FSU,EY,FSTEEL)
      CALL FORCE(P,FCOVER,FCORE,FSTEEL,ASLCOV,ASLCORE,AS,N)
      COUNTER = COUNTER + 1
      IF(COUNTER.GT.100) GO TO 998
      IF(ABS(P-PCOL).GT.DELTAP) THEN
             IF(P.GT.PCOL) THEN
                   ELAST1 = EBOTT
             ELSE
                   ELAST2 = EBOTT
             END IF
      GO TO 222
      ELSE
      CALL MOMENT(MOM,FCOVER,FCORE,FSTEEL,ASLCOV,ASLCORE,AS,CSLICE,N,
     *            DIACOL)
      END IF
      MULT    = MOM
      CURVULT = (ETOP-EBOTT)/(DIACOL-COVER-DLBAR/2.0)
      IF (FLAGR.EQ.1) THEN
         GO TO 1128
      ELSE
         GO TO 1008
      END IF
C
C     ULTIMATE CONDITION FOR COLUMN LOAD > BALANCED AXIAL LOAD
C
777   COUNTER = 0
      WRITE(*,323)
      DELTAP  = 0.0005*PBAL
      ETOP    = ESPAL1
      ELAST1  = ESPAL1
      ELAST2  = EYIELD
778   EBOTT   = (ELAST1+ELAST2)/2.0
      CALL STRAIN(N,CSLICE,ETOP,EBOTT,DIACOL,COVER,DLBAR,DHP,ESTRAIN)
      CALL STRESS(N,ESTRAIN,ECU,ECCU,FCU,FCCU,RU,RC,ESPALL,FCOVER,
     *            FCORE,YSTEEL,FY,ESH,ESU,FSU,EY,FSTEEL)
      CALL FORCE(P,FCOVER,FCORE,FSTEEL,ASLCOV,ASLCORE,AS,N)
      COUNTER = COUNTER + 1
      IF(COUNTER.GT.100) GO TO 998
      IF(ABS(P-PCOL).GT.DELTAP) THEN
             IF(P.GT.PCOL) THEN
                   ELAST1 = EBOTT
             ELSE
                   ELAST2 = EBOTT
             END IF
      GO TO 778
      ELSE
      CALL MOMENT(MOM,FCOVER,FCORE,FSTEEL,ASLCOV,ASLCORE,AS,CSLICE,N,
     *            DIACOL)
      END IF
      MULT    = MOM
      CURVULT = (ETOP-EBOTT)/(DIACOL-COVER-DLBAR/2.0)
      IF(FLAGR.EQ.2) GO TO 1008
C
C     ************************
C     *  RETROFITTED COLUMN  *
C     ************************
C
C     PARAMETERS FOR MANDER'S MODEL ADAPTED FOR STEEL JACKET
C
C     RHOSJ  = CONFINING RATIO FOR STEEL JACKET
C
1128  RHOSJ  = 4.0*TJACK/(DJACK-TJACK)
C
C     IF LATERAL PRESSURE IMPLIED BY FULL YIELD STRENGTH OF STEEL
C     JACKET EXCEED THE GROUT UNIAXIAL COMPRESSIVE STRENGTH,
C     LATERAL PRESSURE IS SET TO GROUT STRENGTH
C
      FYJOLD = FYJ
      IF((0.5*RHOSJ*FYJ).LE.(FGU/1000.)) GO TO 1126
      FYJ    = FGU/1000./0.5/RHOSJ
C     RHOCC   = STEEL TO CORE AREA RATIO
C     KE      = CONFINEMENT EFFECTIVENESS COEFFICIENT DUE TO
C               INTERNAL HOOPS OR SPIRAL
C     FLH     = CONFINING PRESSURE FOR INTERNAL HOOP OR SPIRAL
C     FLUCOV  = CONFINING PRESSURE FOR COVER CONCRETE
C     FLUCOR  = CONFINING PRESSURE FOR CORE CONCRETE
C     ECU     = STRAIN AT UNCONFINED PEAK COMPRESSIVE STRESS
C     FCCCOV  = CONFINED COMPRESSIVE STRENGTH OF COVER CONCRETE
C     FCCCOR  = CONFINED COMPRESSIVE STRENGTH OF CORE CONCRETE
C     ECCCOV  = STRAIN AT PEAK COMPRESSIVE STRESS FOR COVER CONCRETE
C     ECCCOR  = STRAIN AT PEAK COMPRESSIVE STRESS FOR CORE CONCRETE
C     REXPT   = R FACTOR FOR ECCU IN MANDER'S MODEL, AS BEFORE
C     YSECOV  = SECANT MODULUS FOR CONFINED COVER CONCRETE
C     YSECOR  = SECANT MODULUS FOR CONFINED CORE CONCRETE
C     YCONC   = TANGENT MODULUS OF UNCONFINED CONCRETE, AS BEFORE
C     RCOV    = RATIO OF TANGENT TO DIFFERENCE OF TANGENT AND SECANT
C               MODULI FOR CONFINED COVER CONCRETE
C     RCOR    = RATIO OF TANGENT TO DIFFERENCE OF TANGENT AND SECANT
C               MODULI FOR CONFINED CORE CONCRETE
C
1126  RHOCC  = 4.0*TOTAS/DCORE**2.0/PI
      KE     = (1.0-0.5*SSHP/DCORE)**TTYPE/(1.0-RHOCC)
      FLH    = 2.0*KE*TAREA*FYH/DCORE/SHP
      FLUCOV = 0.5*RHOSJ*FYJ
      FLUCOR = FLUCOV + FLH
      ECU    = 0.002
      FCCCOV = FCU*(2.254*SQRT(1.0+7.94*FLUCOV/FCU)-2.0*FLUCOV/FCU
     *         -1.254)
      FCCCOR = FCU*(2.254*SQRT(1.0+7.94*FLUCOR/FCU)-2.0*FLUCOR/FCU
     *         -1.254)
      REXPT  = 5.0
      ECCCOV = (REXPT*(FCCCOV/FCU-1.0)+1.0)*ECU
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      ECCCOR = (REXPT*(FCCCOR/FCU-1.0)+1.0)*ECU
      YSECOV = FCCCOV/ECCCOV
      YSECOR = FCCCOR/ECCCOR
      YCONC  = 1904.0*SQRT(FCU)
      RCOV   = YCONC/(YCONC-YSECOV)
      RCOR   = YCONC/(YCONC-YSECOR)
C
C     PARAMETERS FOR COMPOSITE ANALYSIS
C     ASSUMED MANDER MODEL FOR GROUT INFILLL
C
C     KEGRO   = CONFINEMENT EFFEECTIVENESS COEFFICIENT FOR GROUT INFILL
C     FLUGRO  = CONINING PRESSURE FOR GROUT INFILL
C     EGU     = STRAIN AT PEAK UNCONFINED GROUT
C     FCCGRO  = CONFINED COMPRESSIVE STRENGTH OF GROUT
C     ECCGRO  = STRAIN AT FCCGRO
C     YSEGRO  = SECANT MODULUS FOR CONFINED GROUT
C     YGROUT  = TANGENT MODULUS FOR UNCONFINED GROUT
C     RGRO    = RATIO OF TANGENT TO THE DIFFERENCE OF TANGENT AND
C               SECANT MODULI FOR GROUT
C
      KEGRO        = 1.0
      FLUGRO       = 0.5*KEGRO*RHOSJ*FYJ
      EGU          = 0.002
      FCCGRO = FGU/1000.*(2.254*SQRT(1.0+7940.*FLUGRO/FGU)
     *         -FLUGRO*2000./FGU-1.254)
      ECCGRO = (REXPT*(1000.*FCCGRO/FGU-1.0)+1.0)*EGU
      YSEGRO = FCCGRO/ECCGRO
      YGROUT = 1904.*SQRT(FGU/1000.)
      RGRO   = YGROUT/(YGROUT-YSEGRO)
C
C     ESTIMATION OF ULTIMATE COMPRESSION STRAIN (EULT) FOR CONCRETE
C     CONFINED BY STEEL JACKET - ENERGY BALANCE METHOD
C
      ALPHAJ = 2000.0*RHOS/((1.0+(1428.0*RHOS)**4.0)**0.25)
      ECMAX = 0.14+14.0*FYJ/YSTEEL
C
C     EULT = ULTIMATE STRAIN OF COVER CONCRETE CONFINED BY STEEL
C            JACKET
C
      EULT   = ESPALL + ALPHAJ*RHOSJ*FYJ*ECMAX/FCCCOV
C
C     BALANCED CONDITION FOR RETROFIT COLUMN (EXTREME STEEL
C     REACHES YIELD AND COVER CONCRETE REACHES ULTIMATE STRAIN
C     SIMULTANEOUSLY)
C
      ETOP     = EULT
      EBOTT    = EYIELD
      CALL STRAIN(N,CSLICE,ETOP,EBOTT,DIACOL,COVER,DLBAR,DHP,ESTRAIN)
      CALL RESTRESS(N,ESTRAIN,FCCCOV,RCOV,FCOVER,FCCCOR,ECCCOV,
     *          ECCCOR,RCOR,FCORE,YSTEEL,FY,ESH,EY,ESU,FSU,FSTEEL)
      CALL FORCE(P,FCOVER,FCORE,FSTEEL,ASLCOV,ASLCORE,AS,N)
      CALL MOMENT(MOM,FCOVER,FCORE,FSTEEL,ASLCOV,ASLCORE,AS,
     *               CSLICE,N,DIACOL)
      PBALR    = P
      MBALR    = MOM
C
C     BALANCED CONDITION OF EXTREME STEEL AT FRACTURE STRAIN AND
C     COVER CONCRETE AT ULTIMATE STRAIN SIMULTANEOUSLY
C
      ETOP     = EULT
      EBOTT    = -ESU
      CALL STRAIN(N,CSLICE,ETOP,EBOTT,DIACOL,COVER,DLBAR,DHP,ESTRAIN)
      CALL RESTRESS(N,ESTRAIN,FCCCOV,RCOV,FCOVER,FCCCOR,ECCCOV,
     *          ECCCOR,RCOR,FCORE,YSTEEL,FY,ESH,EY,ESU,FSU,FSTEEL)
      CALL FORCE(P,FCOVER,FCORE,FSTEEL,ASLCOV,ASLCORE,AS,N)
      CALL MOMENT(MOM,FCOVER,FCORE,FSTEEL,ASLCOV,ASLCORE,AS,
     *               CSLICE,N,DIACOL)
      PSU      = P
      MSU      = MOM
C
C     FIRST YIELD OF EXTREME STEEL UNDER CONFINED CONDITION
C
1787  IF(PCOL.GT.PBALR)GO TO 768
      WRITE(*,324)
324   FORMAT(/,10X,'- First Yield Moment For Retrofit',
     *       ' Column')
C
C     ASSUME CONVERGENCE IF DELTA P < 0.05% OF BALANCED AXIAL
C     LOAD FOR CONFINED CONDITIONS
C
      COUNTER = 0
      ESTART  = EULT
C
C     RDELTAP = AXIAL FORCE TOLERANCE LIMIT FOR CONVERGENCE
C
      RDELTAP = 0.0005*PBALR
      EBOTT   = EYIELD
C
C     ELAST1, ELAST2 = TEMPORARY STRAIN VARIABLES
C
      ELAST1   = EYIELD
      ELAST2   = ESTART
666   ETOP     = (ELAST1+ELAST2)/2.0
      CALL STRAIN(N,CSLICE,ETOP,EBOTT,DIACOL,COVER,DLBAR,DHP,ESTRAIN)
      CALL RESTRESS(N,ESTRAIN,FCCCOV,RCOV,FCOVER,FCCCOR,ECCCOV,
     *          ECCCOR,RCOR,FCORE,YSTEEL,FY,ESH,EY,ESU,FSU,FSTEEL)
      CALL FORCE(P,FCOVER,FCORE,FSTEEL,ASLCOV,ASLCORE,AS,N)
      COUNTER = COUNTER+1
      IF(COUNTER.GT.100) GO TO 998
      IF(ABS(P-PCOL).GT.RDELTAP) THEN
            IF(P.GE.PCOL) THEN
                 ELAST2 = ETOP
            ELSE
                 ELAST1 = ETOP
            END IF
         GO TO 666
      ELSE
         CALL MOMENT(MOM,FCOVER,FCORE,FSTEEL,ASLCOV,ASLCORE,AS,
     *               CSLICE,N,DIACOL)
      END IF
```

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C
C     REMYIELD    = RETROFITTED YIELD MOMENT
C     RECURYIELD = RETROFITTED YIELD CURATURE
C
      REMYIELD    = MOM
      RECURYIELD = (ETOP-EBOTT)/(DIACOL-COVER-DLBAR/2.0)
C
C     ULTIMATE CONDITION FOR RETROFIT COLUMN WHEN PCOL < PBALR
C
C     TWO LIMIT STATES EXIST; EITHER EXTREME COMPRESSION FIBER
C     REACHES ULTIMATE COMPRESSIVE STRAIN OR EXTREME TENSION STEEL
C     REACHES FRACTURE STRAIN FIRST
C
      WRITE(*,4323)
4323  FORMAT(/10X,'- Ultimate Moment For Retrofit Column')
      COUNTER = 0
      RDELTAP  = 0.0005*PBALR
      IF(PCOL.GE.PSU) THEN
         ETOP     = EULT
         ELAST1   = EULT
         ELAST2   = -ESU
5222     EBOTT    = (ELAST1+ELAST2)/2.0
         CALL STRAIN(N,CSLICE,ETOP,EBOTT,DIACOL,COVER,DLBAR,DHP,
     *               ESTRAIN)
         CALL RESTRESS(N,ESTRAIN,FCCCOV,RCOV,FCOVER,FCCCOR,ECCCOV,
     *            ECCCOR,RCOR,FCORE,YSTEEL,FY,ESH,EY,ESU,FSU,FSTEEL)
         CALL FORCE(P,FCOVER,FCORE,FSTEEL,ASLCOV,ASLCORE,AS,N)
         COUNTER  = COUNTER + 1
         IF(COUNTER.GT.100) GO TO 998
         IF(ABS(P-PCOL).GT.RDELTAP)THEN
               IF(P.GT.PCOL) THEN
                     ELAST1 = EBOTT
               ELSE
                     ELAST2 = EBOTT
               END IF
         GO TO 5222
         ELSE
         CALL MOMENT(MOM,FCOVER,FCORE,FSTEEL,ASLCOV,ASLCORE,AS,
     *               CSLICE,N,DIACOL)
         END IF
         MULTR = MOM
         CURVRU= (ETOP-EBOTT)/(DIACOL-COVER-DLBAR/2.0)
      ELSE
         EBOTT = -ESU
         ELAST1= -ESU
         ELAST2= EULT
5242     ETOP = (ELAST1+ELAST2)/2.0
         CALL STRAIN(N,CSLICE,ETOP,EBOTT,DIACOL,COVER,DLBAR,DHP,
     *               ESTRAIN)
         CALL RESTRESS(N,ESTRAIN,FCCCOV,RCOV,FCOVER,FCCCOR,ECCCOV,
     *            ECCCOR,RCOR,FCORE,YSTEEL,FY,ESH,EY,ESU,FSU,FSTEEL)
         CALL FORCE(P,FCOVER,FCORE,FSTEEL,ASLCOV,ASLCORE,AS,N)
         COUNTER  = COUNTER + 1
         IF(COUNTER.GT.100) GO TO 998
         IF(ABS(P-PCOL).GT.RDELTAP)THEN
               IF(P.GT.PCOL) THEN
                     ELAST2 = ETOP
               ELSE
                     ELAST1 = ETOP
               END IF
         GO TO 5242
         ELSE
         CALL MOMENT(MOM,FCOVER,FCORE,FSTEEL,ASLCOV,ASLCORE,AS,
     *               CSLICE,N,DIACOL)
         END IF
C
C    MULTR = ULTIMATE RETROFITTED MOMENT
C    CURVRU = ULTIMATE RETROFITTED CURVATURE
C
         MULTR = MOM
         CURVRU= (ETOP-EBOTT)/(DIACOL-COVER-DLBAR/2.0)
      END IF
      GO TO 5224
C
C     ULTIMATE CONDITION FOR RETROFIT COLUMN WHEN PCOL > PBALR
C
768   COUNTER = 0
      WRITE(*,4323)
      RDELTAP = 0.0005*PBALR
      ETOP    = EULT
      ELAST1  = EULT
      ELAST2  = EYIELD
5778  EBOTT   = (ELAST1+ELAST2)/2.0
      CALL STRAIN(N,CSLICE,ETOP,EBOTT,DIACOL,COVER,DLBAR,DHP,ESTRAIN)
      CALL RESTRESS(N,ESTRAIN,FCCCOV,RCOV,FCOVER,FCCCOR,ECCCOV,
     *         ECCCOR,RCOR,FCORE,YSTEEL,FY,ESH,EY,ESU,FSU,FSTEEL)
      CALL FORCE(P,FCOVER,FCORE,FSTEEL,ASLCOV,ASLCORE,AS,N)
      COUNTER = COUNTER + 1
      IF(COUNTER.GT.100) GO TO 998
      IF(ABS(P-PCOL).GT.RDELTAP) THEN
            IF(P.GT.PCOL)THEN
                  ELAST1 = EBOTT
            ELSE
                  ELAST2 = EBOTT
            END IF
      GO TO 5778
      ELSE
      CALL MOMENT(MOM,FCOVER,FCORE,FSTEEL,ASLCOV,ASLCORE,AS,
     *            CSLICE,N,DIACOL)
      END IF
      MULTR = MOM
      CURVRU= (ETOP-EBOTT)/(DIACOL-COVER-DLBAR/2.0)
C
C     **************************************************
C     *  MOMENT CURVATURE ANALYSIS FOR RETROFIT COLUMN *
C     **************************************************
C
5224  WRITE(*,2254)
2254  FORMAT(/,10X,'- Moment-Curvature, It will take a while')
      WRITE(12,2784)
```

```
2784  FORMAT(//,13X,'MOMENT CURVATURE ANALYSIS FOR RETROFIT COLUMN'
     *     ,/,13X,45('*'))
      WRITE(12,1092) TITLE
      WRITE(12,1592)
1592  FORMAT(/,15X,'MOMENT (kip.in)',6X,'CURVATURE (Rad/in)',6X,
     *             'STEEL STRAIN',/)
      I = 1
      MOM = 0.0
      CURTEMP = 0.0
      DELTAE  = EULT/400.0
C
C     DETERMINE STRAIN LEVEL FOR AXIAL FORCE
C
      IF(PCOL.LE.0.0) THEN
      ESTART  = 0.0
      GO TO 5562
      ELSE
      END IF
      EAXIAL  = 0.0
3871  EAXIAL  = EAXIAL + DELTAE
      XCOV    = EAXIAL/ECCCOV
      XCOR    = EAXIAL/ECCCOR
      ES      = EAXIAL
      CALL COREFC(XCOV,FCCCOV,RCOV,FCCOV)
      CALL COREFC(XCOR,FCCCOR,RCOR,FCCOR)
      CALL REBAR(ES,YSTEEL,FY,ESH,ESU,FSU,FS,EY)
      PTEMP  = TOTAS*FS+PI*(DIACOL**2.0-DCORE**2.0)/4.0*FCCOV
     *          +(PI*DCORE**2.0/4.0-TOTAS)*FCCOR
      IF(PTEMP.LT.PCOL) GO TO 3871
C
C     START OF MOMENT CURVATURE ANALYSIS
C
      ESTART  = EAXIAL
5562  ETOP    = ESTART
      EBOTT   = ETOP
3004  CALL STRAIN(N,CSLICE,ETOP,EBOTT,DIACOL,COVER,DLBAR,DHP,ESTRAIN)
      CALL RESTRESS(N,ESTRAIN,FCCCOV,RCOV,FCOVER,FCCCOR,ECCCOV,
     *           ECCCOR,RCOR,FCORE,YSTEEL,FY,ESH,EY,ESU,FSU,FSTEEL)
      CALL FORCE(P,FCOVER,FCORE,FSTEEL,ASLCOV,ASLCORE,AS,N)
      IF(P.LT.PCOL) GO TO 3005
      EBOTT   = EBOTT + EYIELD
      GO TO 3004
3005  COUNTER = 0
      IF(I.EQ.1) THEN
         ELAST1 = EBOTT - EYIELD
      ELSE
         ELAST1 = ETEMP
      END IF
      ELAST2  = EBOTT
3222  EBOTT   = (ELAST1+ELAST2)/2.0
      CALL STRAIN(N,CSLICE,ETOP,EBOTT,DIACOL,COVER,DLBAR,DHP,ESTRAIN)
      CALL RESTRESS(N,ESTRAIN,FCCCOV,RCOV,FCOVER,FCCCOR,ECCCOV,
     *           ECCCOR,RCOR,FCORE,YSTEEL,FY,ESH,EY,ESU,FSU,FSTEEL)
      CALL FORCE(P,FCOVER,FCORE,FSTEEL,ASLCOV,ASLCORE,AS,N)
      COUNTER = COUNTER + 1
      IF (COUNTER.GT.100) GO TO 998
      IF (ABS(P-PCOL).GT.RDELTAP) THEN
           IF(P.GT.PCOL) THEN
                ELAST1 = EBOTT
           ELSE
                ELAST2 = EBOTT
           END IF
      GO TO 3222
      ELSE
      IF(EBOTT.GE.-ESU)GO TO 1178
         WRITE(12,2273)ESU
2273     FORMAT(/,5X,'*** EXTREME REBAR > FRACTURE STRAIN OF ',F6.5,
     *                 ' ***')
      GO TO 2999
1178  CALL MOMENT(MOM,FCOVER,FCORE,FSTEEL,ASLCOV,ASLCORE,AS,CSLICE,N,
     *           DIACOL)
      END IF
      CURTEMP = (ETOP-EBOTT)/(DIACOL-COVER-DLBAR/2.0)
C
C     PLASTIC MOMENT DEFINED BY ETOP = 0.005
C
      IF(ETOP.LE.0.005) THEN
         PEAKMOM = MOM
         PEAKCUR = CURTEMP
      ELSE
      END IF
C
      ETEMP   = EBOTT
      MOMEN(I) = MOM
      CURVA(I) = CURTEMP
      WRITE(12,3425)MOM,CURTEMP,EBOTT
3425  FORMAT(12X,F10.1,11X,F12.8,15X,F10.6)
      IF(ETOP.GE.0.05*EULT) DELTAE = EULT/80.0
      IF(ETOP.GE.0.3*EULT) DELTAE = EULT*0.071
      ETOP     = ETOP + DELTAE
      I = I+1
      IF(ETOP.LT.EULT) GO TO 3004
      WRITE(12,2703)EULT
2703  FORMAT(/,9X,'*** CONCRETE STRAIN > ULTIMATE COMPRESSIVE ',
     *           'STRAIN OF ',F6.5,' ***')
C
C     AXIAL STRAIN LEVEL FOR COMPOSITE ANALYSIS
C
2999  I            = 1
      MOM          = 0.0
      CURTEMP      = 0.0
      DELTAE       = EULT/400.0
      NJACK        = 3
      IF(PCOL.LE.0.0)THEN
         ESTART    = 0.0
         GO TO 15562
      ELSE
      END IF
      EAXIAL       = 0.0
13871 EAXIAL       = EAXIAL+DELTAE
```

```
      XCOV         = EAXIAL/ECCCOV
      XCOR         = EAXIAL/ECCCOR
      XGRO         = EAXIAL/ECCGRO
      ES           = EAXIAL
      CALL COREFC(XCOV,FCCCOV,RCOV,FCCOV)
      CALL COREFC(XCOR,FCCCOR,RCOR,FCCOR)
      CALL COREFC(XGRO,FCCGRO,RGRO,FCGRO)
      CALL REBAR(ES,YSTEEL,FY,ESH,ESU,FSU,FS,EY)
      CALL JACK(ES,YSTEEL,FYJOLD,FSJ)
      PTEMP  = TOTAS*FS+PI*(DIACOL**2.0-DCORE**2.0)/4.0*FCCOV
     *         +(PI*DCORE**2.0/4.0-TOTAS)*FCCOR
      COMPTEMP = PTEMP+(DJACK**2.0-(DJACK-2.0*TJACK)**2.0)*PI*FSJ/4.0
     *         +((DJACK-2.0*TJACK)**2.0-DIACOL**2.0)*FCGRO*PI/4.0
      IF(COMPTEMP.LT.PCOL) GO TO 13871
C
C     MOMENT CURVATURE ANALYSIS FOR COMPOSITE SECTION
C
      ESTART       = EAXIAL
15562 ETOP         = ESTART
      EBOTT        = ETOP
13004 CALL STRAIN(N,CSLICE,ETOP,EBOTT,DIACOL,COVER,DLBAR,DHP,ESTRAIN)
      CALL CSTRAIN(CESTRAIN,N,ETOP,EBOTT,DIACOL,COVER,DJACK,
     *             DHP,DLBAR,CCSLICE)
      CALL RESTRESS(N,ESTRAIN,FCCCOV,RCOV,FCOVER,FCCCOR,ECCCOV,
     *             ECCCOR,RCOR,FCORE,YSTEEL,FY,ESH,EY,ESU,FSU,FSTEEL)
      CALL COMSTRESS(N,CESTRAIN,ECCGRO,RGRO,FCCGRO,FGROUT,
     *             FJACK,YSTEEL,FYJOLD)
      CALL FORCE(P,FCOVER,FCORE,FSTEEL,ASLCOV,ASLCORE,AS,N)
      CALL COMFORCE(PCOM,FGROUT,FJACK,AGROUT,AJACK,N)
      IF((P+PCOM).LT.PCOL)GO TO 13005
      EBOTT        = EBOTT+EYIELD
      GO TO 13004
13005 COUNTER      = 0
      IF(I.EQ.1)THEN
            ELAST1  = EBOTT-EYIELD
      ELSE
            ELAST1  = ETEMP
      END IF
      ELAST2       = EBOTT
13222 EBOTT        = (ELAST1+ELAST2)/2.0
      CALL STRAIN(N,CSLICE,ETOP,EBOTT,DIACOL,COVER,DLBAR,DHP,ESTRAIN)
      CALL CSTRAIN(CESTRAIN,N,ETOP,EBOTT,DIACOL,COVER,DJACK,
     *             DHP,DLBAR,CCSLICE)
      CALL RESTRESS(N,ESTRAIN,FCCCOV,RCOV,FCOVER,FCCCOR,ECCCOV,
     *             ECCCOR,RCOR,FCORE,YSTEEL,FY,ESH,EY,ESU,FSU,FSTEEL)
      CALL COMSTRESS(N,CESTRAIN,ECCGRO,RGRO,FCCGRO,FGROUT,
     *             FJACK,YSTEEL,FYJOLD)
      CALL FORCE(P,FCOVER,FCORE,FSTEEL,ASLCOV,ASLCORE,AS,N)
      CALL COMFORCE(PCOM,FGROUT,FJACK,AGROUT,AJACK,N)
      COUNTER      = COUNTER+1
      IF(COUNTER.GT.100)GO TO 998
      IF(ABS(P+PCOM-PCOL).GT.RDELTAP)THEN
         IF((P+PCOM).GT.PCOL)THEN
            ELAST1 = EBOTT
         ELSE
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```
            ELAST2 = EBOTT
         END IF
      GO TO 13222
      ELSE
         CALL MOMENT(MOM,FCOVER,FCORE,FSTEEL,ASLCOV,ASLCORE,AS,
     *               CSLICE,N,DIACOL)
         CALL CMOMENT(CMOM,FJACK,FGROUT,AJACK,AGROUT,
     *               CCSLICE,N,DJACK)
      END IF
      IF((CMOM+MOM).GE.REMYIELD)THEN
      CRECURYIELD  = (CESTRAIN(1)-CESTRAIN(N))/
     *               (DJACK-2.0*TJACK/NJACK)
      CREMYIELD    = CMOM+MOM
      CRECURYIELD  = CCURLAST+(CRECURYIELD-CCURLAST)*(REMYIELD-CMLAST)
     *               /(CREMYIELD-CMLAST)
      FAVE         = (ABS(FJACK(1))+ABS(FJACK(N)))/2.0
      GO TO 10012
      ELSE
      ETEMP        = EBOTT
      ETOP         = ETOP + DELTAE
      I            = I+1
      CMLAST       = CMOM+MOM
      CCURLAST     = (CESTRAIN(1)-CESTRAIN(N))/
     *               (DJACK-2.0*TJACK/NJACK)
      GO TO 13004
      END IF
C
C     YIELD DISPLACEMENT OF RETROFIT COLUMN
C
10012 CALL RDISPLACE(REDELY,REMYIELD,RECURYIELD,HEIGHT,
     *       HEIGHT,DJACK,TJACK,LJACK,CASH,PEAKMOM,PI,
     *       MRECURYIELD,FAVE,CRECURYIELD)
C
C     ULTIMATE DISPLACEMENT OF RETROFIT COLUMN
C
C     LPR    = RETROFIT PLASTIC HINGE LENGTH
C            = 12*BAR DIA + VERT GAP BETWEEN JACKET AND FOOTING
C
      LPR    = 12.0*DLBAR + CASH
      CALL RUDISPLACE(REDELU,REPU,RECURYIELD,HEIGHT,CURVRU,
     *            MULTR,LPR,PEAKMOM,REMYIELD,REPMAX,REDELY)
C
C     *************
C     * DATA ECHO *
C     *************
C
1008  WRITE(11,1007)
1007  FORMAT(/)
1009  WRITE (11,1010)
1010  FORMAT(17X,24('* '),'*')
      WRITE(11,1012)
      WRITE(11,1015)
1015  FORMAT(17X,'*     UNIVERSITY OF CALIFORNIA   SAN DIEGO      *')
      WRITE(11,1012)
      WRITE(11,1011)
```

```
1011  FORMAT(17X,'*   STRENGTH AND DUCTILITY OF CIRCULAR COLUMN   *')
      WRITE(11,1012)
1012  FORMAT(17X,'*                                               *')
      WRITE(11,1013)
1013  FORMAT(17X,'*          VERSION 1.1  (MAR 1991)              *')
      WRITE(11,1012)
      WRITE(11,1010)
      WRITE(11,1092) TITLE
1092  FORMAT(/,6X,' JOB TITLE : ',A60)
      WRITE (11,1020)
1020  FORMAT(/,25X,'ORIGINAL COLUMN PARAMETERS',
     *     /,25X,26('*'),/)
      WRITE(*,1631) DIACOL
1631  FORMAT(/,14X,'            DIAMETER OF COLUMN :',F11.3,1X,'in')
      WRITE(11,1030)DIACOL
1030  FORMAT(14X,'            DIAMETER OF COLUMN :',F11.3,1X,'in')
      WRITE(*,1200)  HEIGHT
      WRITE(11,1200) HEIGHT
1200  FORMAT(7X,'COLUMN HEIGHT TO PT OF CONTRAFLEXURE :',F11.3,
     *              1X,'ft')
      WRITE(11,1210) LP
1210  FORMAT(14X,'          PLASTIC HINGE LENGTH :',F11.3,1X,'in')
      WRITE(*,2040) COVER
2040  FORMAT(14X,'             COVER TO MAIN BAR :',F11.3,1X,'in')
      WRITE(11,1040)COVER
1040  FORMAT(14X,'             COVER TO MAIN BAR :',F11.3,1X,'in',/)
      WRITE(*,1050) BSIZ
      WRITE(11,1050)BSIZ
1050  FORMAT(14X,'                      MAIN BAR :        #',I2)
      WRITE(*,1060) NBAR
      WRITE(11,1060)NBAR
1060  FORMAT(14X,'                NUMBER OF BARS :        ',I3)
      IF(LTYPE.EQ.1) THEN
              WRITE(*,1070)  FY
              WRITE(11,1070) FY
      ELSE
              WRITE(*,1080)  FY
              WRITE(11,1080) FY
      END IF
1070  FORMAT(14X,'YIELD STRENGTH FOR MAIN STEEL :',F9.1,1X,
     *            'ksi   (Mild Steel)')
1080  FORMAT(14X,'YIELD STRENGTH FOR MAIN STEEL :',F9.1,1X,
     *            'ksi   (H.S.)')
      WRITE(11,1160)FSU
1160  FORMAT(14X,'  ULTIMATE STRENGTH OF STEEL :',F9.1,1X,
     *            'ksi')
      WRITE(11,1090)YSTEEL
1090  FORMAT(14X,'     YOUNGS MODULUS FOR STEEL :',F9.1,1X,
     *            'ksi')
      WRITE(11,1180) EY
1180  FORMAT(14X,'        YIELD STRAIN OF STEEL :',5X,F8.5)
      WRITE(11,1150) ESH
1150  FORMAT(14X,' STRAIN AT HARDENING OF STEEL :',5X,F8.5)
      WRITE(11,1170) ESU
1170  FORMAT(14X,'     ULTIMATE STRAIN OF STEEL :',5X,F8.5,/)
      IF(TTYPE.EQ.1) THEN
              WRITE(*,1100) TSIZ
              WRITE(11,1100)TSIZ
              WRITE(*,1110) SHP
              WRITE(11,1110)SHP
      ELSE
              WRITE(*,1120) TSIZ
              WRITE(11,1120)TSIZ
              WRITE(*,1130) SHP
              WRITE(11,1130)SHP
      END IF
      IF(GTIE.EQ.1)THEN
        WRITE (*,2131) FYH
        WRITE(11,1131) FYH
      ELSE
        WRITE (*,12131) FYH
        WRITE(11,11131) FYH
      END IF
2131  FORMAT(14X,'        YIELD STRENGTH OF TIE :',F9.1,1X,
     *            'ksi   (Mild Steel)')
1131  FORMAT(14X,'        YIELD STRENGTH OF TIE :',F9.1,1X,
     *            'ksi   (Mild Steel)',/)
12131 FORMAT(14X,'        YIELD STRENGTH OF TIE :',F9.1,1X,
     *            'ksi   (H.S.)')
11131 FORMAT(14X,'        YIELD STRENGTH OF TIE :',F9.1,1X,
     *            'ksi   (H.S.)',/)
1100  FORMAT(14X,'                     TIE SIZE :        #',I2,
     *          4X,' (Spiral)')
1110  FORMAT(14X,'                        PITCH :',F9.3,1X,'in')
1120  FORMAT(14X,'                     TIE SIZE :        #',I2,
     *          4X,' (Hoops)')
1130  FORMAT(14X,'                  TIE SPACING :',F9.3,1X,'in')
      WRITE(*,2140) FCU
2140  FORMAT(14X,'          CONCRETE STRENGTH :',F11.3,1X,'ksi',/)
      WRITE(11,1140)FCU
1140  FORMAT(14X,'          CONCRETE STRENGTH :',F11.3,1X,'ksi')
      WRITE(11,1142)ESPAL1
1142  FORMAT(19X,'CONCRETE ULTIMATE STRAIN :',6X,F5.3,/)
      WRITE(*,1248) PCOL
1248  FORMAT(14X,'          COLUMN AXIAL LOAD :',1X,F8.1,' kips',/)
      WRITE(11,1248)PCOL
C
C     *******************
C     *  RESULTS  OUTPUT *
C     *******************
C
      WRITE(*,1141)
      WRITE(11,1141)
1141  FORMAT(24X,'RESULTS FOR ORIGINAL COLUMN',/,24X,27('*'),/)
      IF(PCOL.LT.PAXIAL) GO TO 1146
            WRITE(*,1332)
            WRITE(11,1332)
1332        FORMAT(5X,'CHECK COLUMN AXIAL LOAD ( TOO LARGE )')
            WRITE(*,1331) PCOL,PAXIAL
            WRITE(11,1331)PCOL,PAXIAL
```

```
1331       FORMAT(/,5X,'COLUMN AXIAL LOAD =',F8.1,' kips >',
     *         ' PURE AXIAL CAPACITY =',F8.1,' kips')
      CLOSE(11)
      CLOSE(12)
      GO TO 9998
1146  IF(PCOL.GT.PBAL) THEN
         WRITE(*,1220)
         WRITE(11,1220)
1220     FORMAT(10X,'TENSION STEEL DOES NOT YIELD BEFORE SPALLING',
     *          ' OF COVER CONCRETE',/)
         WRITE(*,1230) PCOL,PBAL
         WRITE(11,1230)PCOL,PBAL
1230     FORMAT(10X,'COLUMN AXIAL LOAD =',1X,F6.1,' kips > ',
     *          'BALANCED AXIAL LOAD =',1X,F6.1,' kips',/)
         WRITE(*,1231) MULT
         WRITE(11,1231)MULT
1231     FORMAT(10X,'ULTIMATE MOMENT CAPACITY =',1X,
     *          F8.1,' kip.in',/)
         WRITE(*,1232) CURVULT
         WRITE(11,1232)CURVULT
1232     FORMAT(10X,'ULTIMATE                    CURVATURE =',1X,
     *          F10.8,' Rad/in')
      ELSE
         IF(PCOL.GT.PTENALL) GO TO 1234
           WRITE(*,1333)
           WRITE(11,1333)
1333       FORMAT(5X,'CHECK COLUMN TENSION ( TOO SMALL )')
           WRITE(*,1330) PCOL,PTENALL
           WRITE(11,1330)PCOL,PTENALL
1330       FORMAT(/,5X,'COLUMN TENSION =',F8.1,' kips <',
     *          ' 75% STEEL CAPACITY =',F8.1,' kips')
           CLOSE(11)
           CLOSE(12)
         GO TO 9998
1234  CALL DISPLACE(DELTAU,DELTAY,CURYIELD,CURVULT,LP,HEIGHT,
     *     HEIGH1,MYIELD,MULT,MU,MCURVY)
      WRITE(11,1250)MYIELD
1250  FORMAT(10X,'ORIGINAL  YIELD   MOMENT =',1X,F8.1,' kip.in')
      WRITE(*,1260) MULT
      WRITE(11,1260)MULT
1260  FORMAT(10X,'ORIGINAL ULTIMATE MOMENT =',1X,F8.1,' kip.in',/)
      WRITE(11,1270)CURYIELD
1270  FORMAT(10X,'CURVATURE AT FIRST YIELD OF EXTREME REBAR = ',
     *     1X,F10.8,' Rad/in')
      WRITE(11,1280)MCURVY
1280  FORMAT(10X,'EQUIVALENT ELASTO-PLASTIC YIELD CURVATURE = ',
     *     1X,F10.8,' Rad/in')
      WRITE(11,1290)CURVULT
1290  FORMAT(10X,'ULTIMATE                    CURVATURE = ',
     *     1X,F10.8,' Rad/in')
      WRITE(11,1291)CURVULT/MCURVY
1291  FORMAT(10X,'CURVATURE      DUCTILITY        FACTOR =',
     *          F5.1,/)
      WRITE(*,1310) DELTAY
      WRITE(11,1310)DELTAY
1310  FORMAT(10X,'YIELD            DISPLACEMENT =',1X,F8.3,' in')
      WRITE(*,1320) DELTAU
      WRITE(11,1320)DELTAU
1320  FORMAT(10X,'ULTIMATE         DISPLACEMENT =',1X,F8.3,' in')
      WRITE(*,1300) MU
      WRITE(11,1300)MU
1300  FORMAT(10X,'DISPLACEMENT DUCTILITY FACTOR =',1X,F8.3,
     *                       '      (FLEXURE)')
      CALL SHEAR(HEIGH1,DIACOL,DCORE,SHP,TAREA,FYH,FCU,PCOL,MU,
     *           K1,PI,MULT,VD,VCOL,MUS)
      WRITE(11,4108)VD,K1
4108  FORMAT(/,10X,'MAXIMUM FEASIBLE  SHEAR  FORCE =',F8.1,' kips',
     *' ( K1 = ',F5.3,' )')
      WRITE(11,4107)VCOL
4107  FORMAT(10X,'IDEAL SHEAR STRENGTH OF COLUMN =',F8.1,' kips')
      IF(MUS.GT.0.)WRITE(11,4109)MUS
4109  FORMAT(/,10X,'REDUCED DISPLACEMENT DUCTILITY FACTOR =',1X,F8.3
     *          ,'      (SHEAR)')
      END IF
      IF(FLAGR.EQ.2) GO TO 1999
      WRITE(*,3201)
      WRITE(11,3218)
3218  FORMAT(/////)
      WRITE(11,3201)
3201  FORMAT(///,23X,'RETROFIT COLUMN PARAMETERS',/,23X,26('*'),/)
      WRITE(*,3202) DJACK
      WRITE(11,3202) DJACK
3202  FORMAT(14X,'    OUTSIDE DIAMETER OF JACKET :',F11.3,1X,'in')
      WRITE(*,3203) TJACK
      WRITE(11,3203) TJACK
3203  FORMAT(14X,'            THICKNESS OF JACKET :',F11.3,1X,'in')
      WRITE(*,3471) LJACK
      WRITE(11,3471)LJACK
3471  FORMAT(14X,'               LENGTH OF JACKET :',F11.3,1X,'in')
      WRITE(*,3472) CASH
      WRITE(11,3472) CASH
3472  FORMAT(6X,'                 JACKET TOE FROM FOOTING :', F11.3,1X,
     *          'in',/)
      WRITE(*,3204) FYJOLD
      WRITE(11,3204) FYJOLD
3204  FORMAT(14X,'      YIELD STRENGTH OF JACKET :',F11.3,1X,'ksi',/)
      WRITE(*,3473) FGU/1000.0
      WRITE(11,3473)FGU/1000.0
3473  FORMAT(14X,'   GROUT COMPRESSIVE STRENGTH :',F11.3,1X,'ksi',/)
      WRITE(*,1711) EULT
      WRITE(11,1711)EULT
1711  FORMAT(14X,'  ULTIMATE COMPRESSIVE STRAIN :',6X,F5.3,/)
      WRITE(*,1712) LPR
      WRITE(11,1712)LPR
1712  FORMAT(14X,'          PLASTIC HINGE LENGTH :',F11.3,1X,'in',/)
      WRITE(*,3141)
      WRITE(11,3141)
3141  FORMAT(//,23X,'RESULTS FOR RETROFIT COLUMN',/,23X,27('*'),//)
      IF(PCOL.LE.PBALR)GO TO 6772
      WRITE(*,7220)
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      WRITE(11,7220)
7220  FORMAT(10X,'TENSION STEEL DOES NOT YIELD BEFORE CONCRETE',
     *           ' REACHES ULTIMATE STRAIN')
      WRITE(*,1230)PCOL,PBALR
      WRITE(11,1230)PCOL,PBALR
      WRITE(*,3261)MULTR
      WRITE(11,3261)MULTR
      WRITE(*,1284)CURVRU
      WRITE(11,1284)CURVRU
      WRITE(*,8239)PEAKMOM/HEIGH1/12.0
      WRITE(11,8239)PEAKMOM/HEIGH1/12.0
8239  FORMAT(/,10X,'PLASTIC      SHEAR      FORCE =',
     *                  1X,F8.1,' kips')
      GO TO 1999
6772  WRITE(*,3142) REMYIELD
      WRITE(11,3142)REMYIELD
3142  FORMAT(15X,'    YIELD      MOMENT =',1X,F8.1,'   kip.in')
      WRITE(*,3144) PEAKMOM
      WRITE(11,3144)PEAKMOM
3144  FORMAT(15X,'    PLASTIC    MOMENT =',1X,F8.1,'   kip.in')
      WRITE(*,3261) MULTR
      WRITE(11,3261)MULTR
3261  FORMAT(15X,'    ULTIMATE   MOMENT =',1X,F8.1,'   kip.in',/)
      WRITE(*,1270) RECURYIELD
      WRITE(11,1270)RECURYIELD
      WRITE(*,1280) MRECURYIELD
      WRITE(11,1280)MRECURYIELD
      WRITE(*,1284) CURVRU
      WRITE(11,1284)CURVRU
1284  FORMAT(10X,'CURVATURE    AT    ULTIMATE     CONDITION =',2X,F10.8
     *       ,' Rad/in')
      WRITE(*,1291) CURVRU/MRECURYIELD
      WRITE(11,1291)CURVRU/MRECURYIELD
      WRITE(*,3146) REDELY
      WRITE(11,3146)REDELY
3146  FORMAT(10X,'YIELD            DISPLACEMENT =',1X,F8.3,' in')
      WRITE(*,1320) REDELU
      WRITE(11,1320)REDELU
      WRITE(*,1300) REDELU/REDELY
      WRITE(11,1300)REDELU/REDELY
      WRITE(*,8239) PEAKMOM/HEIGH1/12.0
      WRITE(11,8239)PEAKMOM/HEIGH1/12.0
      IF(PEAKMOM.GT.MULTR)THEN
         MTEMP = PEAKMOM
      ELSE
         MTEMP = MULTR
      END IF
C
C     CHECK FLEXURE JACKET LENGTH
C
      IF((HEIGH1*12.-CASH-LJACK).LT.0.2*DIACOL) GO TO 7612
C
C     BEND MOMENT IMMEDIATELY ABOVE JACKET SHOULD BE LESS THAN OR
C     EQUAL TO FACTJ*(UNRETROFITTED FLEXURAL CAPACITY)
C
      FACTJ = 0.75
      LJMIN = HEIGH1*12.0 -CASH -FACTJ*HEIGH1*12.0*MULT/PEAKMOM
      IF(LJMIN.GE.LJACK) THEN
         WRITE(*,63241)
         WRITE(11,63241)
         WRITE(*,63242)LJMIN
         WRITE(11,63242)LJMIN
      ELSE
         WRITE(11,63243)LJMIN
      END IF
63241 FORMAT(/,10X,'         **** INADEQUATE JACKET LENGTH ****')
63242 FORMAT(/,10X,'**** INCREASE JACKET LENGTH TO ',F11.3,' in ****')
63243 FORMAT(/,10X,'MINIMUM     JACKET     LENGTH = ',F11.3,' in')
7612  CALL RSHEAR(HEIGH1,DIACOL,DCORE,SHP,TAREA,FYH,FCU,PCOL,MU,
     *     K1,PI,MTEMP,DJACK,TJACK,LJACK,CASH,FYJ,VD,VCOL)
1999  CLOSE(11)
      CLOSE(12)
9998  WRITE(*,*)
      IF(FLAGR.EQ.1) THEN
        WRITE(*,*)'For Hardcopy, "Print Colret.out"',
     *         ' or "Print Curvat.out"'
      ELSE
        WRITE(*,*)'For Hardcopy, "Print Colret.out"'
      END IF
      GO TO 9999
998   WRITE(*,*)
      WRITE(*,997)
997   FORMAT(' Solution fails to converge ( No. of iterations > 100)')
9999  END
C
C     *********************************************
C     * SUBROUTINE FOR SLICING OF COLUMN SECTION *
C     *********************************************
C
      SUBROUTINE SLICE(NCOVER,CSLICE,SLCOVER,ASLICE,ASLCORE,ASLCOV,
     *                 NCORE,SLCORE,DIACOL,DCORE,N)
         REAL SLCOVER,SLCORE,DIACOL,DCORE
         REAL CSLICE(N),ASLICE(N),ASLCORE(N),ASLCOV(N)
C
C     SLICING TOP COVER CONCRETE IN NCOVER SLICES
C
      AOLD = 0.0
      DO 10 I = 1,NCOVER,1
            Y           = I*SLCOVER
            CSLICE(I)   = Y-SLCOVER/2.0
            ASECTION    = DIACOL**2.0*ACOS(1.0-2.0*Y/DIACOL)/4.0-
     *                    (DIACOL/2.0-Y)*SQRT(DIACOL*Y-Y**2.0)
            ASLICE(I)   = ASECTION-AOLD
            ASLCORE(I)  = 0.0
            ASLCOV(I)   = ASLICE(I)
            AOLD        = ASECTION
10    CONTINUE
C
C     SLICING CORE CONCRETE INTO NCORE SLICES
C
```

```
      OLDCORE = 0.0
      DO 20 I = NCOVER+1,NCORE+NCOVER,1
              J          = I-NCOVER
              Z          = J*SLCORE
              Y          = (DIACOL-DCORE)/2.0+Z
              ASECTION   = DIACOL**2.0*ACOS(1.0-2.0*Y/DIACOL)/4.0-
     *                     (DIACOL/2.0-Y)*SQRT(DIACOL*Y-Y**2.0)
              ASECORE    = DCORE**2.0*ACOS(1.0-2.0*Z/DCORE)/4.0-
     *                     (DCORE/2.0-Z)*SQRT(DCORE*Z-Z**2.0)
              ASLICE(I)  = ASECTION-AOLD
              ASLCORE(I) = ASECORE-OLDCORE
              ASLCOV(I)  = ASLICE(I)-ASLCORE(I)
              CSLICE(I)  = Y-SLCORE/2.0
              AOLD       = ASECTION
              OLDCORE    = ASECORE
20    CONTINUE
C
C     SLICING BOTTOM COVER CONCRETE INTO NCOVER SLICES
C
      DO 30 I = NCORE+NCOVER+1,N-1,1
              J          = I-NCORE-NCOVER
              Y          = J*SLCOVER+DCORE+(DIACOL-DCORE)/2.0
              CSLICE(I)  = Y-SLCOVER/2.0
              ASECTION   = DIACOL**2.0*ACOS(1.0-2.0*Y/DIACOL)/4.0-
     *                     (DIACOL/2.0-Y)*SQRT(DIACOL*Y-Y**2.0)
              ASLICE(I)  = ASECTION-AOLD
              ASLCORE(I) = 0.0
              ASLCOV(I)  = ASLICE(I)
              AOLD       = ASECTION
30    CONTINUE
      YLAST     = DIACOL - SLCOVER
      CSLICE(N) = DIACOL - SLCOVER/2.0
      ASLICE(N) = DIACOL**2.0*PI/4.0 -
     *            (DIACOL**2.0*ACOS(1.0-2.0*YLAST/DIACOL)/4.0-
     *            (DIACOL/2.0-YLAST)*SQRT(DIACOL*YLAST-YLAST**2.0))
      ASLCORE(N)= 0.0
      ASLCOV(N) = ASLCOV(1)
      RETURN
      END
C
C     ***********************************************
C     * SUBROUTINE TO ASSIGN STEEL AREA TO SEGMENTS *
C     ***********************************************
C
      SUBROUTINE ASSIGN(N,NCOVER,NCORE,THETA,AS,SLCORE,DIACOL,
     *                  DCORE,COVER,DLBAR,TOTAS,PI)
         REAL AS(N),THETA(N)
         THETAOLD = THETA(NCOVER)
      DO 60 I=NCOVER+1,NCORE+NCOVER,1
              FLAG       = 0.0
              J          = I-NCOVER
              Z          = J*SLCORE
              Y          = (DIACOL-DCORE)/2.0+Z
              IF((DIACOL/2.0-Y).GT.0.0) GO TO 90
              IF((Y-DIACOL/2.0).LE.(DIACOL/2.0-COVER-DLBAR/2.0))
     *        GO TO 80
              IF(FLAG.EQ.1.0) GO TO 60
              SUM        = 0.0
              DO 100 K=1,I-1,1
                   SUM = AS(K) + SUM
100           CONTINUE
              AS(I)        = TOTAS-SUM
              FLAG         = 1.0
              GO TO 60
90            IF((DIACOL/2.0-Y).GT.(DIACOL/2.0-COVER-DLBAR/2.0))
     *        GO TO 60
80            THETA(I)    = 2.0*ACOS((DIACOL-2.0*Y)/
     *                      (DIACOL-2.0*COVER-DLBAR))
              AS(I)       = TOTAS*(THETA(I)-THETAOLD)/2.0/PI
              THETAOLD    = THETA(I)
60    CONTINUE
      RETURN
      END
C
C     *******************************************************
C     *  SUBROUTINE TO SLICE STEEL JACKET AND GROUT RING FOR *
C     *                COMPOSITE ANALYSIS                     *
C     *******************************************************
C
      SUBROUTINE COMSLICE(TJACK,CCSLICE,AJACK,AGROUT,DJACK,DIACOL,N,
     *                    ACSLICE,ACOLSL)
      INTEGER NJACK,NGROUT,N
      REAL CCSLICE(N),AJACK(N),AGROUT(N),ACSLICE(N),ACOLSL(N)
      REAL DJACK,DIACOL,Y,TJACK,DGROUT,OLDCORE,AOLD,SLJACK,SLGROUT,
     *     SLCOL,OLDCOL,ASECTION,ASECORE
C
C     TOP PORTION OF STEEL JACKET INTO NJACK SLICES
C
      AOLD          = 0.0
      NJACK         = 3
      NGROUT        = 3
      SLJACK        = TJACK/NJACK
      SLGROUT       = (DJACK-2.0*TJACK-DIACOL)/2.0/NGROUT
      SLCOL         = DIACOL/(N-2.0*NJACK-2.0*NGROUT)
      DO 10 I = 1,NJACK,1
      Y             = I*SLJACK
      CCSLICE(I)    = Y-SLJACK/2.0
      ASECTION      = DJACK**2.0*ACOS(1.0-2.0*Y/DJACK)/4.0-
     *                (DJACK/2.0-Y)*SQRT(DJACK*Y-Y**2.0)
      ACSLICE(I)    = ASECTION-AOLD
      AJACK(I)      = ACSLICE(I)
      AGROUT(I)     = 0.0
      AOLD          = ASECTION
10    CONTINUE
C
C     SLICING OUTER GROUT RING
C
      DGROUT        = DJACK-2.0*TJACK
      OLDCORE       = 0.0
      DO 20 I = NJACK+1,NJACK+NGROUT,1
```

```
      J            = I-NJACK
      Z            = J*SLGROUT
      Y            = TJACK+Z
      ASECTION     = DJACK**2.0*ACOS(1.0-2.0*Y/DJACK)/4.0-
     *                  (DJACK/2.0-Y)*SQRT(DJACK*Y-Y**2.0)
      ASECORE      = DGROUT**2.0*ACOS(1.0-2.0*Z/DGROUT)/4.0-
     *                  (DGROUT/2.0-Z)*SQRT(DGROUT*Z-Z**2.0)
      ACSLICE(I)   = ASECTION-AOLD
      AGROUT(I)    = ASECORE-OLDCORE
      AJACK(I)     = ACSLICE(I)-AGROUT(I)
      CCSLICE(I)   = Y-SLGROUT/2.0
      AOLD         = ASECTION
      OLDCORE      = ASECORE
20    CONTINUE
C
C     SLICING MIDDLE PORTION
C
      OLDCOL       = 0.0
      DO 30 I = NJACK+NGROUT+1,N-NJACK-NGROUT,1
      J            = I-NJACK-NGROUT
      X            = J*SLCOL
      Z            = X+(DJACK-2.0*TJACK-DIACOL)/2.0
      Y            = TJACK+Z
      ASECTION     = DJACK**2.0*ACOS(1.0-2.0*Y/DJACK)/4.0-
     *                  (DJACK/2.0-Y)*SQRT(DJACK*Y-Y**2.0)
      ASECORE      = DGROUT**2.0*ACOS(1.0-2.0*Z/DGROUT)/4.0-
     *                  (DGROUT/2.0-Z)*SQRT(DGROUT*Z-Z**2.0)
      ACOL         = DIACOL**2.0*ACOS(1.0-2.0*X/DIACOL)/4.0-
     *                  (DIACOL/2.0-X)*SQRT(DIACOL*X-X**2.0)
      ACSLICE(I)   = ASECTION-AOLD
      ACOLSL(I)    = ACOL-OLDCOL
      AGROUT(I)    = ASECORE-OLDCORE-ACOLSL(I)
      AJACK(I)     = ACSLICE(I)-ACOLSL(I)-AGROUT(I)
      CCSLICE(I)   = Y-SLCOL/2.0
      OLDCOL       = ACOL
      OLDCORE      = ASECORE
      AOLD         = ASECTION
30    CONTINUE
      DO 40 I = N-NJACK-NGROUT+1,N
      AGROUT(I)    = AGROUT(N+1-I)
      AJACK(I)     = AJACK(N+1-I)
      CCSLICE(I)   = DJACK-CCSLICE(N+1-I)
40    CONTINUE
      RETURN
      END
C     ***********************************************************
C     * SUBROUTINE TO COMPUTE STRAIN PROFILE ACROSS SECTION *
C     ***********************************************************
C
      SUBROUTINE STRAIN(N,CSLICE,ETOP,EBOTT,DIACOL,COVER,DLBAR,
     *                  DHP,ESTRAIN)
      REAL ESTRAIN(N),CSLICE(N)
      DO 1000 I=1,N,1
         ESTRAIN(I) = ETOP-(ETOP-EBOTT)/(DIACOL-2.0*COVER-DLBAR/2.0
     *                 +DHP/2.0)*CSLICE(I)
```

```
1000  CONTINUE
      RETURN
      END
C     *****************************************************************
C     *      SUBROUTINE TO COMPUTE STRAIN PROFILE ACROSS STEEL        *
C     *          JACKET AND GROUT RING FOR COMPOSITE ANALYSIS         *
C     *****************************************************************
C
      SUBROUTINE CSTRAIN(CESTRAIN,N,ETOP,EBOTT,DIACOL,COVER,DJACK,
     *           DHP,DLBAR,CCSLICE)
      REAL CESTRAIN(N),CCSLICE(N)
      REAL ETOP1,DHP
      ETOP1          = (ETOP-EBOTT)*((DJACK+DIACOL)/2.0-COVER-DLBAR/2.0)
     *                 /(DIACOL-2.*COVER-DLBAR/2.+DHP/2.)+EBOTT
      DO 1000 I=1,N,1
      CESTRAIN(I) = ETOP1-(ETOP1-EBOTT)/(DJACK/2.0+DIACOL/2.0
     *                 -COVER-DLBAR/2.0)*CCSLICE(I)
1000  CONTINUE
      RETURN
      END
C
C     ***************************************
C     * COMPUTE STRESSES IN ORIGINAL COLUMN *
C     ***************************************
C
      SUBROUTINE STRESS(N,ESTRAIN,ECU,ECCU,FCU,FCCU,RU,RC,ESPALL,
     *           FCOVER,FCORE,YSTEEL,FY,ESH,ESU,FSU,EY,FSTEEL)
      INTEGER N
      REAL X,ES,ECU,FCU,FCCU,RU,RC,ESPALL,YSTEEL,FY,ESH,EY,
     *     FCOV,FCC,FS
      REAL ESTRAIN(N),FCOVER(N),FCORE(N),FSTEEL(N)
      DO 120 I=1,N,1
              IF(ESTRAIN(I).GT.0.0) THEN
              X  = ESTRAIN(I)/ECU
              CALL COVERFC(X,FCU,RU,ESPALL,ECU,FCOV)
              FCOVER(I) = FCOV
              X  = ESTRAIN(I)/ECCU
              CALL COREFC(X,FCCU,RC,FCC)
              FCORE(I)  = FCC
      ELSE
              FCOVER(I) = 0.0
              FCORE(I)  = 0.0
      END IF
              ES        = ESTRAIN(I)
              IF(ES.GE.0.0) THEN
                     CALL REBAR(ES,YSTEEL,FY,ESH,ESU,FSU,FS,EY)
                     FSTEEL(I) = FS
              ELSE
                     ES        = -ES
                     CALL REBAR(ES,YSTEEL,FY,ESH,ESU,FSU,FS,EY)
                     FSTEEL(I) = -FS
              END IF
120   CONTINUE
      RETURN
      END
```

```
C
C     ***************************************
C     * COMPUTE STRESSES IN RETROFIT COLUMN *
C     ***************************************
C
      SUBROUTINE RESTRESS(N,ESTRAIN,FCCCOV,RCOV,FCOVER,FCCCOR,ECCCOV,
     *           ECCCOR,RCOR,FCORE,YSTEEL,FY,ESH,EY,ESU,FSU,FSTEEL)
      INTEGER N
      REAL X,ES,ESU,FCU,FCCCOV,RCOV,FCCCOR,RCOR,FCCOV,FCCOR,YSTEEL,
     *     FY,ESH,EY,FS,FSU,ECCCOV,ECCCOR
      REAL ESTRAIN(N),FCOVER(N),FCORE(N),FSTEEL(N)
      DO 205 I=1,N,1
             IF(ESTRAIN(I).GT.0.0) THEN
                X = ESTRAIN(I)/ECCCOV
                CALL COREFC(X,FCCCOV,RCOV,FCCOV)
                FCOVER(I) = FCCOV
                X = ESTRAIN(I)/ECCCOR
                CALL COREFC(X,FCCCOR,RCOR,FCCOR)
                FCORE(I)  = FCCOR
             ELSE
                FCOVER(I) = 0.0
                FCORE(I)  = 0.0
             END IF
                ES        = ESTRAIN(I)
             IF(ES.GE.0.0) THEN
                CALL REBAR(ES,YSTEEL,FY,ESH,ESU,FSU,FS,EY)
                FSTEEL(I) = FS
             ELSE
                ES        = -ES
                CALL REBAR(ES,YSTEEL,FY,ESH,ESU,FSU,FS,EY)
                FSTEEL(I) = -FS
             END IF
205   CONTINUE
      RETURN
      END
C
C     ********************************************************
C     * SUBROUTINE FOR STRESSES IN STEEL JACKET AND GROUT RING *
C     *              IN COMPOSITE ANALYSIS                     *
C     ********************************************************
C
      SUBROUTINE COMSTRESS(N,CESTRAIN,ECCGRO,RGRO,FCCGRO,FGROUT,
     *           FJACK,YSTEEL,FYJOLD)
      INTEGER N
      REAL CESTRAIN(N),FGROUT(N),FJACK(N)
      REAL FS,FGECCGRO,RGRO,YSTEEL,FYJOLD,EY,FG,ES
      DO 205 I = 1,N,1
         IF(CESTRAIN(I).GT.0.0) THEN
            X           = CESTRAIN(I)/ECCGRO
            CALL COREFC(X,FCCGRO,RGRO,FG)
            FGROUT(I)  = FG
         ELSE
            FGROUT(I)  = 0.0
         END IF
C
C     JACKET STRESS
C
            ES            = CESTRAIN(I)
         IF(ES.GE.0.0)THEN
            CALL JACK(ES,YSTEEL,FYJOLD,FS)
            FJACK(I)    = FS
         ELSE
            ES          = -ES
            CALL JACK(ES,YSTEEL,FYJOLD,FS)
            FJACK(I)    = -FS
         END IF
205   CONTINUE
      RETURN
      END
C     *********************************************************
C     * UNCONFINED STRESS-STRAIN RELATIONS FOR COVER CONCRETE *
C     *********************************************************
C
      SUBROUTINE COVERFC(X,FCU,RU,ESPALL,ECU,FCOV)
      IF (X.GT.2.0) THEN
           IF (X.LE.ESPALL/ECU) THEN
                 FCOV = FCU*(2.0*RU/(RU-1.0+2.0**RU))*
     *                  (1.0-(X-2.0)/(ESPALL/ECU-2.0))
           ELSE
                 FCOV = 0.0
           END IF
      ELSE
           FCOV = FCU*RU*X/(RU-1.0+X**RU)
      END IF
      RETURN
      END
C
C     ******************************************************
C     * CONFINED STRESS-STRAIN RELATIONS FOR CORE CONCRETE *
C     ******************************************************
C
      SUBROUTINE COREFC(X,FCCU,RC,FCC)
           FCC = FCCU*X*RC/(RC-1.0+X**RC)
      RETURN
      END
C
C     ******************************************
C     * STRESS-STRAIN RELATIONS FOR LONG STEEL *
C     ******************************************
C
      SUBROUTINE REBAR(ES,YSTEEL,FY,ESH,ESU,FSU,FS,EY)
           RE = ESU-ESH
           M  = (FSU/FY*(30.0*RE+1.0)**2.0-60.0*RE-1.0)/15.0/RE**2
         IF(ES.LE.EY) THEN
           FS = YSTEEL*ES
         ELSE
           IF(ES.LE.ESH) THEN
              FS = FY
           ELSE
              IF(ES.LE.ESU) THEN
```

```
                  FS = FY*((M*(ES-ESH)+2.0)/(60.0*(ES-ESH)+2.0)+
     *                 (ES-ESH)*(60.0-M)/2.0/(30.0*RE+1.0)**2.0)
              ELSE
                  FS = 0.0
              END IF
          END IF
      END IF
      RETURN
      END
C
C     ***************************************************
C     *      STRESS-STRAIN RELATION FOR STEEL JACKET     *
C     ***************************************************
C
      SUBROUTINE JACK(ES,YSTEEL,FY,FS)
      REAL ES,YSTEEL,FY,FS
      EY           = FY/YSTEEL
      IF(ES.LE.EY)THEN
         FS       = YSTEEL*ES
      ELSE
         FS       = FY
      END IF
      RETURN
      END
C     *********************************
C     * SUMMING UP ALL INTERNAL FORCES *
C     *********************************
C
      SUBROUTINE FORCE(P,FCOVER,FCORE,FSTEEL,ASLCOV,ASLCORE,AS,N)
              REAL P,AS(N),ASLCORE(N)
              REAL FCOVER(N),FCORE(N),FSTEEL(N),ASLCOV(N)
              P = 0.0
              DO 2000 I = 1,N,1
                   P = P+FCOVER(I)*ASLCOV(I)+FCORE(I)*(ASLCORE(I)
     *                 -AS(I))+FSTEEL(I)*AS(I)
2000  CONTINUE
      RETURN
      END
C
C     ****************************************************************
C     * SUMMING UP OF INTERNAL FORCES FOR STEEL JACKET AND GROUT RING *
C     ****************************************************************
C
      SUBROUTINE COMFORCE(PCOM,FGROUT,FJACK,AGROUT,AJACK,N)
      REAL AGROUT(N),AJACK(N),FGROUT(N),FJACK(N)
      REAL PCOM
      INTEGER N
      PCOM          = 0.0
      DO 2000 I =1,N,1
      PCOM             = PCOM+FJACK(I)*AJACK(I)+FGROUT(I)*AGROUT(I)
2000  CONTINUE
      RETURN
      END
C
C     ******************************
C     * SUMMING UP INTERNAL MOMENTS *
C     ******************************
C
      SUBROUTINE MOMENT(MOM,FCOVER,FCORE,FSTEEL,ASLCOV,ASLCORE,AS,
     *                  CSLICE,N,DIACOL)
         REAL MOM,AS(N),CSLICE(N),ASLCORE(N)
         REAL FCOVER(N),FCORE(N),FSTEEL(N),ASLCOV(N)
         MOM = 0.0
         DO 2100 I = 1,N,1
              MOM = MOM+(FCOVER(I)*ASLCOV(I)+FCORE(I)*(ASLCORE(I)
     *              -AS(I))+FSTEEL(I)*AS(I))*(DIACOL/2.0-CSLICE(I))
2100  CONTINUE
      RETURN
      END
C
C     ***********************************************************
C     * SUMMING UP OF MOMENT DUE TO STEEL JACKET AND GROUT RING *
C     ***********************************************************
C
      SUBROUTINE CMOMENT(CMOM,FJACK,FGROUT,AJACK,AGROUT,
     *                   CCSLICE,N,DJACK)
      REAL FJACK(N),FGROUT(N),AJACK(N),AGROUT(N),CCSLICE(N)
      REAL CMOM,DJACK
      INTEGER N
      CMOM          = 0.0
      DO 2100 I=1,N,1
      CMOM = CMOM+(FJACK(I)*AJACK(I)+FGROUT(I)*AGROUT(I))*
     *       (DJACK/2.0-CCSLICE(I))
2100  CONTINUE
      RETURN
      END
C     ********************************************
C     * TO COMPUTE DISPLACEMENT OF ORIGINAL COLUMN *
C     ********************************************
C
      SUBROUTINE DISPLACE(DELTAU,DELTAY,CURYIELD,CURVULT,LP,HEIGHT,
     *           HEIGH1,MYIELD,MULT,MU,MCURVY)
      REAL DELTAU,DELTAY,CURVYIELD,CURVULT,LP,HEIGHT,MYIELD,MULT,MU
      REAL MCURVY,HEIGH1
      MCURVY = CURYIELD*MULT/MYIELD
      DELTAY = MCURVY*(HEIGH1*12.0)**2.0/3.0
      MU       = 1.0+3.0*(CURVULT/MCURVY-1.0)*LP/12.0/HEIGH1*
     *           (HEIGHT/HEIGH1-LP/HEIGH1/24.0)
      DELTAU = MU*DELTAY
      RETURN
      END
C
C     **********************************************
C     * COMPUTE YIELD DISPLACEMENT OF RETROFIT COLUMN *
C     **********************************************
C
      SUBROUTINE RDISPLACE(REDELY,REMYIELD,RECURYIELD,HEIGHT,
     *       HEIGH1,DJACK,TJACK,LJACK,CASH,PEAKMOM,PI,
     *       MRECURYIELD,FAVE,CRECURYIELD)
      REAL REDELY,REMYIELD,RECURYIELD,HEIGHT,TJACK,LJACK,VYR,
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```
     *     CASH,PEAKMOM,DIN,OLDEI,COMEI,DJACK,PI,OMEGA1,OMEGA2,
     *     HEIGH1,H,H1,MRECURYIELD,MMID,EIMID,ALPHAC,AG,ASJ,
     *     BOND,PERI,LB,LT,M2,M3,M4,OMEG1M,OMEG2M
      REAL DEL(5),PHI(5),Y(5)
      MRECURYIELD = RECURYIELD*PEAKMOM/REMYIELD
      H      = HEIGHT*12.0
      H1     = HEIGH1*12.0
      DIN    = DJACK - TJACK*2.0
      ASJ    = (DJACK**2.0-DIN**2.0)*PI/4.0
      PERI   = PI*DIN
      OLDEI  = REMYIELD/RECURYIELD
      COMEI  = REMYIELD/CRECURYIELD
      VYR    = REMYIELD/H1
C
C     AVERAGE BOND STRENGTH BETWEEN STEEL JACKET AND GROUT IN KSI
C
      BOND   = 0.110
      LT     = ASJ*FAVE*(H1-CASH-LJACK)/(BOND*PERI*H1-ASJ*FAVE)
      LB     = ASJ*FAVE*(H1-CASH)/(ASJ*FAVE+BOND*PERI*H1)
      IF(LT.LE.0.0)GO TO 5369
      IF((LT+LB).GT.LJACK)GO TO 5369
      Y(1)   = H1-CASH-LJACK
      PHI(1) = Y(1)*RECURYIELD/H1
      Y(2)   = Y(1)+LT
      PHI(2) = Y(2)*REMYIELD/H1/COMEI
      Y(3)   = H1-CASH-LB
      PHI(3) = Y(3)*REMYIELD/H1/COMEI
      Y(4)   = Y(3)+LB
      PHI(4) = Y(4)*RECURYIELD/H1
      Y(5)   = H1
      PHI(5) = RECURYIELD
C
      DEL(1) = PHI(1)*Y(1)**2.0/3.0
C
      OMEGA1 = COMEI/OLDEI-1.0
      OMEGA2 = OMEGA1/(OMEGA1+1.0)
      M4     = Y(4)/H1*REMYIELD
      M2     = Y(2)/H1*REMYIELD
      IF(M2.EQ.0.) THEN
         DEL(2) = 0.
      ELSE
         DEL(2) = (-LT*Y(2)*LOG(1.0-OMEGA2)/OMEGA2+
     1     LT**2.0/OMEGA2*(1.0+Y(2)/(H1-CASH-LJACK+LT))
     2     *(1.0+LOG(1.0-OMEGA2)/OMEGA2)
     3     -LT**3.0/OMEGA2**3.0*(LOG(1.0-OMEGA2)
     4     +OMEGA2**2.0/2.0+OMEGA2)/(H1-CASH-LJACK+LT))*M2/COMEI
      END IF
C
      DEL(3) = PHI(2)*(Y(3)-Y(2))*(Y(3)+Y(2))/2.0+
     1         (PHI(3)-PHI(2))*(Y(3)-Y(2))*(Y(2)+2.0*Y(3))/6.0
C
      DEL(4) = (LB*Y(4)*LOG(1.0+OMEGA1)/OMEGA1-
     1         LB**2.0/OMEGA1*(1.0-LOG(1.0+OMEGA1)/OMEGA1)*
     2         (1.0+VYR*Y(4)/M4)+VYR*LB**3.0/M4/OMEGA1**3.0*
     3         (LOG(1.0+OMEGA1)+OMEGA1**2.0/2.0-OMEGA1))*M4/OLDEI
C
      DEL(5)  = PHI(4)*(Y(5)-Y(4))*(Y(5)+Y(4))/2.0+
     1          (PHI(5)-PHI(4))*(Y(5)-Y(4))*(Y(4)+2.0*Y(5))/6.0
      GO TO 5634
C
C     ALPHAC = COEFFICIENT FOR DEGREE OF COMPOSITE ACTION
C
C     SUMMING UP FOR YIELD DISPLACEMENT
C
5369  ALPHAC = BOND*PERI*LJACK*H1/2./ASJ/FAVE/(H1-CASH-LJACK/2.)
      EIMID  = OLDEI+ALPHAC*(COMEI-OLDEI)
      Y(1)   = H1-CASH-LJACK
      PHI(1) = Y(1)*RECURYIELD/H1
      Y(2)   = H1-CASH-LJACK/2.0
      PHI(2) = Y(2)/H1*REMYIELD/EIMID
      Y(3)   = H1-CASH
      PHI(3) = Y(3)/H1*RECURYIELD
      Y(4)   = H1
      PHI(4) = RECURYIELD
      Y(5)   = 0.0
      PHI(5) = 0.0
      OMEG1M = EIMID/OLDEI-1.0
      OMEG2M = 1.0-OLDEI/EIMID
C
      DEL(1) = PHI(1)*Y(1)**2.0/3.0
C
      M3     = Y(3)/H1*REMYIELD
      M2     = Y(2)/H1*REMYIELD
      DEL(2) = (-LJACK*Y(2)*LOG(1.0-OMEG2M)/OMEG2M/2.0+
     1         LJACK**2.0/OMEG2M/4.0*(1.0+VYR/M2*Y(2))*
     2         (1.0+LOG(1.0-OMEG2M)/OMEG2M)
     3         -VYR*LJACK**3.0/OMEG2M**3.0/M2/8.0*
     4         (LOG(1.0-OMEG2M)+OMEG2M**2.0/2.0+OMEG2M))*M2/EIMID
      DEL(3) = (LJACK*Y(3)*LOG(1.0+OMEG1M)/OMEG1M/2.0-
     1         LJACK**2.0/OMEG1M/4.0*(1.0-LOG(1.0+OMEG1M)/OMEG1M)*
     2         (1.0+VYR*Y(3)/M3)+VYR*LJACK**3.0/M3/OMEG1M**3.0/8.0*
     3         (LOG(1.0+OMEG1M)+OMEG1M**2.0/2.0-OMEG1M))*M3/OLDEI
C
      DEL(4)  = PHI(3)*(Y(4)-Y(3))*(Y(4)+Y(3))/2.0+
     1          (PHI(4)-PHI(3))*(Y(4)-Y(3))*(Y(3)+2.0*Y(4))/6.0
C
      DEL(5)  = 0.0
C
C     SUMMING OF DISPLACEMENT CONTRIBUTIONS
C
5634  REDELY= 0.0
      DO 1329 I=1,5,1
        REDELY = REDELY + DEL(I)
1329  CONTINUE
C
C     SCALING OF FIRST YIELD DISPLACEMENT TO GIVE EQUIVALENT
C     ELASTO-PLASTIC YIELD DISPLACEMENT
C
      REDELY = REDELY*PEAKMOM/REMYIELD
      RETURN
```

```
      END
C
C     ****************************************************
C     * COMPUTE ULTIMATE DISPLACEMENT OF RETROFIT COLUMN *
C     ****************************************************
C
      SUBROUTINE RUDISPLACE(REDELU,REPU,RECURYIELD,HEIGHT,
     *    CURVRU,MULTR,LPR,PEAKMOM,REMYIELD,REPMAX,REDELY)
      REAL REDELU,RECURYIELD,HEIGHT,REPU,H,LPR,MULTR,
     *     CURVRU,PEAKMOM,REMYIELD,REPMAX
      H      = HEIGHT*12.0
      REPU   = MULTR/H
      REPMAX = PEAKMOM/H
      REDEL1 = REDELY*REMYIELD/PEAKMOM
      REDELU = REDEL1+(CURVRU-RECURYIELD*PEAKMOM/REMYIELD)*LPR*
     *         (H-0.5*LPR)
      RETURN
      END
C
C     *******************************************************
C     *      SHEAR CONSIDERATIONS FOR EXISTING COLUMN       *
C     *******************************************************
C
      SUBROUTINE SHEAR(HEIGH1,DIACOL,DCORE,SHP,TAREA,FYH,FCU,PCOL,
     *              MUF,K1,PI,PEAKMOM,VD,VCOL,MUS)
      REAL NU,CHI,VCF,VSF,VF,VD,VI,AE,AG,ALPHA,PI,RHOS,TAREA,DCORE,
     *     SHP,HEIGH1,FYH,FCU,MUS,MUF,VCOL,K1
      MUS   = -1.0
      ALPHA = 1.0
      IF(HEIGH1*12.0/DIACOL.LT.2.0) ALPHA = 2.0*DIACOL/HEIGH1/12.0
      AG    = PI*DIACOL**2.0/4.0
      AE    = 0.8*AG
      RHOS  = 4.0*TAREA/DCORE/SHP
      NU    = 0.2
      CHI   = RHOS*FYH/NU/FCU
C
C     SHEAR DEMAND FROM CAPACITY DESIGN
C
C     INCREASE SHEAR DEMAND BY 15% FOR MATERIAL OVERSTRENGTH
C
      VD = PEAKMOM/HEIGH1/12.0*K1
C
C     TRANSVERSE STEEL'S CONTRIBUTION TO FINAL SHEAR STRENGTH
C
      IF((((1.0-CHI)/CHI).LT.2.15**2.0).AND.(CHI.LT.1.0)) THEN
         VSF = PI*TAREA*FYH*DCORE*SQRT((1.0-CHI)/CHI)/SHP/2.
      ELSE
         VSF = 2.15*PI*TAREA*FYH*DCORE/SHP/2.
      END IF
C
C     CONCRETE CONTRIBUTION TO FINAL SHEAR STRENGTH
C
      IF(PCOL.GE.-0.01*FCU*AG) THEN
            IF(RHOS.GE.0.01) THEN
              VCF   = 2.227*SQRT(FCU*1000.0)*AE/1000.0
            ELSE
              VCF   = 222.7*RHOS*SQRT(FCU*1000.0)*AE/1000.0
            END IF
      ELSE
         VCF = 0.0
      END IF
C
C     FINAL SHEAR STRENGTH
C
      VF = VCF + VSF
C
C     INITIAL SHEAR STRENGTH
C
      IF(PCOL.GE.0.)THEN
       VI = 0.00445*ALPHA*(1.0+3.0*PCOL/FCU/AG)*SQRT(1000.0*FCU)*AE
     *      + PI*TAREA*FYH*DCORE/SHP/2.
      ELSE
         VI = PI*TAREA*FYH*DCORE/SHP/2.
      END IF
C
C     COMPARE COLUMN SHEAR STRENGTH WITH SHEAR DEMAND
C
      IF(VD.GE.VI) THEN
            WRITE(*,2562)
            WRITE(11,2562)
2562  FORMAT(/,23X,'**** BRITTLE SHEAR FAILURE ****')
            WRITE(*,2563)
            WRITE(11,2563)
2563  FORMAT(18X,'**** RECOMMEND RETROFIT FOR SHEAR STRENGTH ****')
            IF(MUF.LE.2.0) THEN
               VCOL = VI
            ELSE
               IF(MUF.GE.6.0) THEN
                 VCOL = VF
               ELSE
                 VCOL = VI-(VI-VF)*(MUF-2.0)/4.0
               END IF
             END IF
            WRITE(*,32141)VD,VCOL
      ELSE
            IF(MUF.GE.6.0) THEN
               VCOL = VF
               IF(VD.GE.VCOL) THEN
                  MUS= 4.0*(VI-VD)/(VI-VF)+2.0
                  WRITE(*,2564)
                  WRITE(11,2564)
2564  FORMAT(/,23X,'**** LIMITED DUCTILITY ****')
                  WRITE(*,32143)MUS
               ELSE
                  WRITE(*,2565)
                  WRITE(11,2565)
2565  FORMAT(/,23X,'**** DUCTILE FAILURE ****')
                  WRITE(*,32142)VD,VCOL
               END IF
            ELSE
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               IF(MUF.LE.2.0)THEN
                  VCOL = VI
                  WRITE(*,2564)
                  WRITE(11,2564)
                  WRITE(*,32142)VD,VCOL
               ELSE
                  VCOL = VI-(VI-VF)*(MUF-2.0)/4.0
                  IF(VD.GT.VCOL) THEN
                    MUS= 4.0*(VI-VD)/(VI-VF)+2.0
                    WRITE(*,2564)
                    WRITE(11,2564)
                    WRITE(*,32143)MUS
                  ELSE
                    WRITE(*,2564)
                    WRITE(11,2564)
                    WRITE(*,32142)VD,VCOL
                  END IF
               END IF
            END IF
      END IF
32141 FORMAT(/,5X,'MAX SHEAR FORCE =',F8.1,' kips >',
     *       ' SHEAR STRENGTH =',F8.1,' kips')
32142 FORMAT(/,5X,'MAX SHEAR FORCE =',F8.1,' kips <',
     *       ' SHEAR STRENGTH =',F8.1,' kips')
32143 FORMAT(/,10X,'REDUCED DISPLACEMENT DUCTILITY FACTOR =',F8.3,
     *       '          (SHEAR)')
      RETURN
      END
C
C     **********************************************************
C     *       SHEAR CONSIDERATIONS FOR RETROFITTED COLUMN      *
C     **********************************************************
C
      SUBROUTINE RSHEAR(HEIGH1,DIACOL,DCORE,SHP,TAREA,FYH,FCU,PCOL,
     *       MUF,K1,PI,PEAKMOM,DJACK,TJACK,LJACK,CASH,FYJ,VD,VCOL)
      REAL NU,CHI,VCF,VSF,VF,VD,VI,AE,AG,ALPHA,PI,RHOS,TAREA,DCORE,
     *     SHP,HEIGH1,FYH,FCU,VCOL,DJACK,TJACK,LJACK,CASH,VCAS,K1,
     *     TEMPV,MUF
      VCOL = 0.0
      MUS   = -1.0
      ALPHA = 1.0
      IF(HEIGH1*12.0/DIACOL.LT.2.0) ALPHA = 2.0*DIACOL/HEIGH1/12.0
      AG    = PI*DIACOL**2.0/4.0
      AE    = 0.8*AG
      RHOS  = 4.0*TAREA/DCORE/SHP
      NU    = 0.2
      CHI   = RHOS*FYH/NU/FCU
C
C     SHEAR DEMAND FROM CAPACITY DESIGN
C
C     INCREASE SHEAR DEMAND BY K1 FACTOR FOR MATERIAL OVERSTRENGTH
C
      VD = PEAKMOM/HEIGH1/12.0*K1
C
C     TRANSVERSE STEEL'S CONTRIBUTION TO FINAL SHEAR STRENGTH
C
      IF((((1.0-CHI)/CHI).LT.2.15**2.0).AND.(CHI.LT.1.0)) THEN
         VSF = PI*TAREA*FYH*DCORE*SQRT((1.0-CHI)/CHI)/SHP/2.
      ELSE
         VSF = 2.15*PI*TAREA*FYH*DCORE/SHP/2.
      END IF
C
C     CONCRETE CONTRIBUTION TO FINAL SHEAR STRENGTH
C
      IF(PCOL.GE.-0.01*FCU*AG) THEN
            IF(RHOS.GE.0.01) THEN
              VCF   = 2.227*SQRT(FCU*1000.0)*AE/1000.0
            ELSE
              VCF   = 222.7*RHOS*SQRT(FCU*1000.0)*AE/1000.0
            END IF
      ELSE
         VCF = 0.0
      END IF
C
C     FINAL SHEAR STRENGTH
C
      VF = VCF + VSF
C
C     INITIAL SHEAR STRENGTH
C
      IF(PCOL.GE.0.)THEN
       VI = 0.00445*ALPHA*(1.0+3.0*PCOL/FCU/AG)*SQRT(1000.0*FCU)*AE
     *      + PI*TAREA*FYH*DCORE/SHP/2.
      ELSE
          VI = PI*TAREA*FYH*DCORE/SHP/2.
      END IF
C
C     SHEAR STRENGTH OF STEEL JACKET
C
      VCAS   = PI*TJACK*(DJACK-TJACK)*FYJ/2.0
C
C     CHECK IF UNCASED LENGTH EXCEEDS 0.2*COLUMN DIAMETER
C
      IF((HEIGH1*12.-LJACK-CASH).GT.0.2*DIACOL)THEN
         IF(VI.GT.(VF+VCAS)) THEN
          IF(MUF.LE.2.0) THEN
              TEMPV = VI
            ELSE
              IF(MUF.GE.6.0) THEN
                 TEMPV = VF
              ELSE
                 TEMPV = VI-(VI-VF)*(MUF-2.0)/4.0
              END IF
            END IF
            VCOL = TEMPV+VCAS
            IF(VD.GT.VCOL) GO TO 8999
              WRITE(11,42141)VD,K1,VCOL
              WRITE(*,42142)VD,VCOL
             RETURN
           ELSE
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              VCOL = VI
                IF(VD.GT.VCOL) THEN
                  WRITE(*,6300)
                  WRITE(11,6300)
                  WRITE(11,42141)VD,K1,VCOL
                  WRITE(*,6301)
                  WRITE(11,6301)
                  RETURN
                ELSE
                  WRITE(11,42141)VD,K1,VCOL
                  WRITE(*,42142)VD,VCOL
                  RETURN
                END IF
            END IF
      ELSE
            IF(MUF.LE.2.0) THEN
               TEMPV = VI
            ELSE
               IF(MUF.GE.6.0) THEN
                 TEMPV = VF
               ELSE
                 TEMPV = VI-(VI-VF)*(MUF-2.0)/4.0
               END IF
             END IF
          VCOL = TEMPV+VCAS
          IF(VD.GT.VCOL) GO TO 8999
             WRITE(11,42141)VD,K1,VCOL
             WRITE(*,42142)VD,VCOL
      END IF
      RETURN
8999           TADD = (DJACK-SQRT(DJACK**2.-8.*(VD-VCOL)/PI/FYJ))/2.
               WRITE(*,6254)
               WRITE(11,6254)
               WRITE(11,42141)VD,K1,VCOL
               WRITE(*,42143)VD,VCOL
               IF((HEIGHT*12.-LJACK-CASH).GT.0.2*DIACOL)THEN
                  WRITE(*,6301)
                  WRITE(11,6301)
               ELSE
                  WRITE(*,6253)(TJACK+TADD)
                  WRITE(11,6253)(TJACK+TADD)
               END IF
               RETURN
6254  FORMAT(/,20X,'**** INADEQUATE SHEAR STRENGTH ****')
6253  FORMAT(/,10X,'INCREASE JACKET THICKNESS TO ',F6.3,' in')
6300  FORMAT(/,10X,'**** INADEQUATE  SHEAR  STRENGTH IN UNCASED REGION'
     *          ,' ****')
6301  FORMAT(/,10X,'**** EXTEND JACKET LENGTH TO FULL HEIGHT OF COLUMN'
     *          ,' ****')
42141 FORMAT(/,10X,'MAXIMUM FEASIBLE  SHEAR  FORCE =',F8.1,' kips',
     *' ( K1 = ',F5.3,' )',/,10X,
     *'IDEAL SHEAR STRENGTH OF COLUMN =',F8.1,' kips')
42142 FORMAT(/,10X,'MAX SHEAR FORCE =',F8.1,' kips <',
     *          ' SHEAR STRENGTH =',F8.1,' kips')
42143 FORMAT(/,10X,'MAX SHEAR FORCE =',F8.1,' kips >',
     *          ' SHEAR STRENGTH =',F8.1,' kips')
      RETURN
      END
```