COMPUTER AIDED DESIGN OF COLUMNS UNDER BIAXIAL BENDING

by

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ABSTRACT

The equations for design of rectangular reinforced concrete columns are rarely used in actual design due to their mathematical complexities. Rather, design aids in the form of tables or charts, that are sometimes inadequate to complete a design, are employed by designers.

This study covers the design of rectangular reinforced concrete tied columns under the action of axial load and uniaxial or biaxial bending moments according to the requirements of (ACI 318M-83), Building Code Requirements for Reinforced Concrete (1). A microcomputer program is developed to perform necessarry computations. The program is designed interactively and is based on iteration and internal halving techniques. It performs the design and analysis of columns with known or unknown cross sectional dimensions. When dimensions are not known, acceptable ranges for them must be fed into the computer upon which the program establishes its own cross sectional dimensions using the maximum range given for the reinforcement ratio.

ÖZET

Matematiksel karmaşıklıkları nedeniyle, diktörtgen betonarme kolonların hesabında kullanılan denklemler, gerçek hesaplamalarda nadiren kullanılmaktadır. Bunların yerine bazen çözüm için yeterli olmayan tablo ve grafikler kullanılır.

Bu çalışma eksenel yük altında tek veya iki yönde eğilmeye maruz etriyeli dikdörtgen betonarme kolonların ACI (318M-83)'e göre hesabını kapsamaktadır (1). Gerekli işlemleri yapmak üzere bir kompüter programı geliştirilmiştir. Program sorulu cevaplı olarak düzenlenmiş ve iterasyon ve yarılma tekniklerini kullanmaktadır. Programla kesit boyutları bilinen yada bilinmeyen kolonların analizi ve hesabı yapılabilir. Boyutları bilinmeyen kesitler için, programın verilen maksimum donatı oranını kullanarak kesit boyutlarını tayin edeceği makul bir boyut aralığı verilmelidir.

LIST OF FIGURES

LIST OF SYMBOLS

a_b : depth of equivalent rectangular stress block at balanced condition

A_g : gross area of section

A_s : area of tension reinforcement

A_s' : area of compression reinforcement

b : width of compression face of member

C_b : distance from extreme compression fiber to neutral axis at balanced condition.

C_m : a factor relating actual moment diagram to an equivalent moment diagram.

d' : distance from extreme compression member to centroid of compression reinforcement.

e : eccentricity

e_b : eccentricity at balanced condition

e_x : eccentricity about x axis

e_y : eccentricity about y axis

E_c : modulus of elasticity of concrete.

E_s : modulus of elasticity of reinforcement.

f'_c : specified compressive strength of concrete.

f'_s : stress of compression reinforcement.

f_y : specified yield strength of reinforcement.

h : overall thickness of member.

H : hight of building above foundations.

I_g : moment of inertia of gross concrete section about centroidal axis, neglecting reinforcement.

k : effective length factor of compression member.

ℓ_e : equivalent length of compression member.

ℓ_u : unsupported length of compression member.

M_b : value of moment at balanced condition.

M_c : factored moment to be used for design of compression member.

M_n : nominal moment capacity of section under a certain load.

M_{nx} : nominal moment capacitty of section in x direction under a certain load.

M_{ny} : nominal moment capacity of section in y direction under a certain load.

M_u : factored moment at section $\leqslant \phi M_n$.

M_{1b} : value of smaller factored end moment on a compression member due to the loads result no appreciable sidesway, calculated by conventional frame analysis, positive if member is bent in single curvature, negative bent in double curvature.

M_{2b} : value of larger factored end moment on compression member due to loads that result in no appreciable sidesway, calculated by conventional frame analysis.

M_{2s} : value of larger factored end moment on compression member due to loads that result in appreciable sidesway, calculated by conventional frame analysis.

n : number of stroies in ..e building.

N_{c1} : resultant compressive force in concrete.

N_{c2} : resultant compressive force in steel.

N_t : resultant tensile force.

P_b : column load for balanced design.

P_c : critical column load.

P_n : nominal axial load strength.

P_u : fully factored axial load.

P_o : nominal axial load strength at zero eccentricity.

P_{ox} : nominal axial load strength of section in x direction at certain eccentricity.

P_{oy} : nominal axial load strength of section in y direction at certain eccentricity.

r : radius of gyration of cross section of a compression member.

β_1 : factor used in the equivalent rectangular stress diagram for concrete at the ultimate load.

β_d : ratio of maximum factored dead load moment to maximum factored total moment.

δ : moment magnification factor.

δ_b : moment magnification factor for frames braced against sidesway.

δ_c : moment magnification factor for frames not braced against sidesway.

Δ : elastically computed lateral deflection

ε_c : compressive strain in concrete

ε_s' : strain in compression steel

ψa : average of ψ_A and ψ_B, $(\psi_A + \psi_B)/2$

ψ_A : ratio of the sum of stiffnesses of the compression members to that of the flexural members in a plane at the upper end of compression member.

ψ_B : ratio of the sum of stiffnesses of the compression members to that of the flexural members in a plane at the lower end of compression member.

ψ_{min} : smallest of ψ_A and ψ_B

ϕ : strength reduction factor

I. INTRODUCTION

The strength calculations for rectangular reinforced concrete columns under axial load and biaxial bending are tedious. Due to their mathematical complexities, the design equations are rarely used in actual design. Rather, design aids in the form of tables or charts are employed by designers.

The common design procedure is, for a given eccentricity, to locate a point on the load and moment interaction diagram and then express the relationship between bending moment and axial load capacity for a particular reinforced concrete section. Because there exist numerous combinations of section geometries and material properties for columns, the use of design aids is sometimes inadequate for a complete design. Since the interaction curves of bending moment and axial load are usually given for square columns; it is necessary to provide correction factors for rectangular columns.

In addition, the user must predetermine the ratio of the spacing between the reinforcement on opposite faces of the section and the overall section dimension before the design aids can be applied. With the increasing availability of higher strength steels and concretes, and with accurate method of analysis, it is now possible to design smaller cross sections for a given load than before. Thus more slender members have come into use, rendering slenderness effects more important in design.

Since the mini and microcomputers are increasingly available, computerized procedures for design and analysis of reinforced concrete columns without the need for design tables and charts is now possible. The design procedure presented here is based on iteration and is written in an interactive mode so that the designer and computer can respond spontaneously. The design method is applicable to the design of tied reinforced concrete columns under biaxial bending including slenderness effects, and it complies with the requirements of (ACI 318M-83) Building Code Requirements for Reinforced Concrete (1).

II. DESIGN PROCEDURE

All practical columns are members subject not only to axial load but also to moment either uniaxial or biaxial. This study covers short columns, those where lateral deflections are not significant, and long columns where deflections due to slenderness have an important effect on member strength. In design, the procedure of (ACI 318 M-83)Building Code Requirements for Reinforced Concrete (1) will be followed.

2.1 DESIGN FOR AXIAL LOAD AND MOMENT

2.1.1 General Considerations

Design or investigation of a short compression member is based primarily on the strength of its cross-section. Strength of a cross-section under combined flexure and axial load must satisfy both stress and strain compatibility. The combined nominal axial load, P_n, and moment strength, M_n, is then multiplied by the appropriate strength reduction factor (1) ,ϕ, to obtain the design strength ($P_u = \phi P_n$, $M_u = \phi M_n$) of the section. The value of ϕ may be increased linearly from the value for compression members ($\phi = 0.70$) to the value for flexure ($\phi = 0.90$) as the design axial load strength, ϕP_n, decreases from 0.10 $f'_c A_g$ or ϕP_b, whichever is smaller, to zero (1). A "strength interaction diagram" can than be generated between the design axial load strength, ϕP_n, and design moment strength ϕM_n; this diagram defines the usable strength of a section. A typical schematic strength interaction diagram is shown in Fig. 2.1, illustrating the various strength curves for different ϕ values.

Maximum strain at the extreme concrete compression fiber is always assumed as 0.003. Tensile strength of the concrete is neglected in strength computations. The equivalent rectangular concrete stress block can be used in lieu of other complex stress-strain relationships for concrete (1). Note that the required strength (P_u,M_u) must be at least equal to the structural effects of the load groups which represent various combinations of loads and forces to which a structure may be subjected. Since all concrete columns are subject to some moment past American codes set minimum eccentrities of 0.10 h to be used for tied columns. The specified minimum eccentricities were originally intended to serve as a means of reducing the axial design load strength of a section in pure compression to account for accidental eccentricities not considered in the analysis, and to recognise that concrete strength is less under sustained high loads. The primary purpose was to limit the axial load strength for design of compression members with small or zero computed end moments. For the 1983 code this is accomplished directly by limiting the axial load strength of a section in pure compression to 80 percent of the pure axial load strength.

For tied reinforced members

$$P_n(max) = 0.80\ (P_o) \tag{2.1}$$

where the pure axial load strength is

$$P_o = 0.85\ f_c'\ (A_g - A_s) + f_y\ A_s \tag{2.2}$$

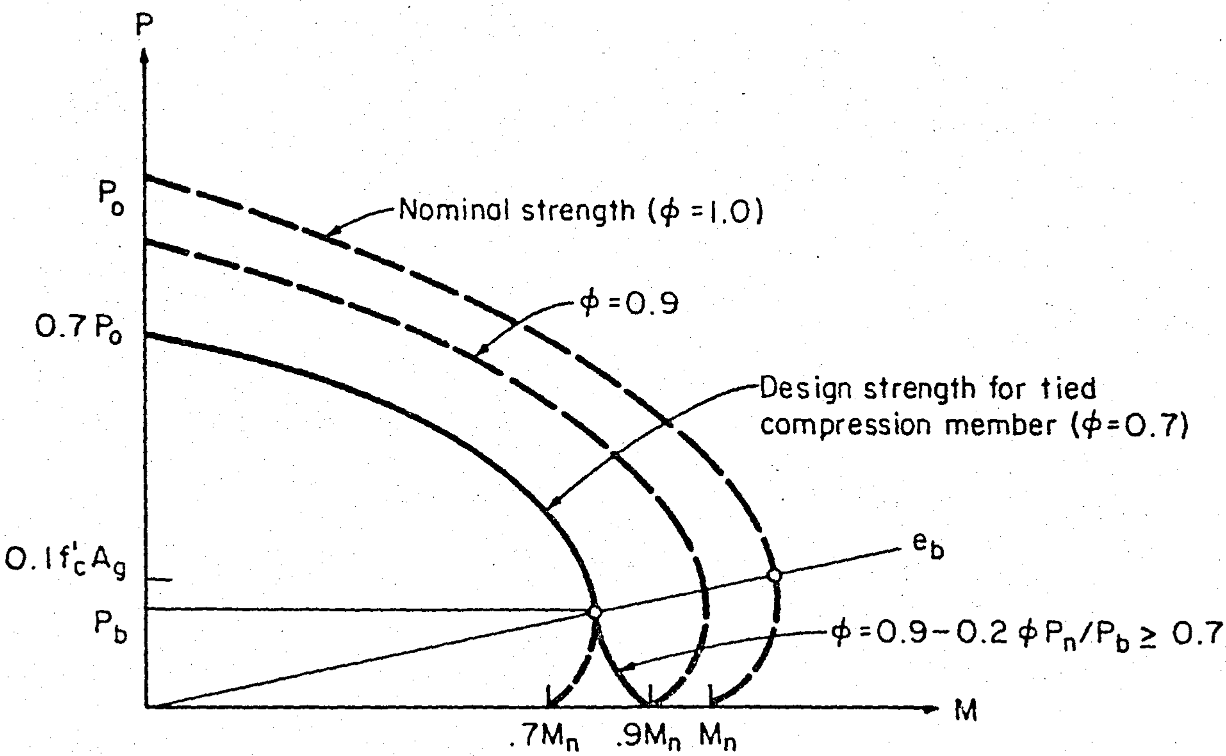


FIGURE 2.1 ϕ Factor Increase for Compression Members

where

A_g = gross area of section

f'_c = specified compressive strength of concrete

P_b = column load for balanced condition

P_o = nominal axial load strength at zero eccentricity

In the interaction diagram, the axial load capacity decreases as moment is increased. Any loading that falls within this area is a possible loading; any combination outside the area represents a failure combination. Four points along the load-moment strength interaction diagram are significant to define the behavior of members subject to combined axial load and flexure. Referring to Fig. 2.2,(1) pure compression ...P_o, (2) maximum axial load strength permitted by the Code ... $P_n(max)$, (3) balanced conditions .... P_b, M_b and (4) pure flexure ... M_n. For values of axial load strength greater than balanced conditions $P_n>P_b$, compression in the concrete controls the strength that is called compression failure and, for values of axial load strength less than balanced conditions $P_n<P_b$, tension in the reinforcement controls the strength that is called tension failure. When the axial load strength equal to balanced conditions $P_n=P_b$, and $e=e_b$ then concrete reaches a strain level of 0.003 at the same time with the yielding of steel, i.e. simultaneous crushing of concrete and yielding of steel.

2.1.2 Balanced Loading

Any column, regardless of its reinforcement, will reach its balanced ultimate load when the load is so placed as to maintain the eccentricity $e_b = M_b/P_b$. Balance in a column is a matter of loading, and it is more descriptive to speak of balanced loading rather than of a balanced column. For a given column it is very easy to establish the nominal balanced load P_b and the accompanying e_b. Fig. 2.3 shows a balanced column load condition. The maximum strain of 0.003 in compression and f_y/E_s give c_b from similar triangles (Fig. 2.3 (c)), most simply by thinking of the large dotted triangle.

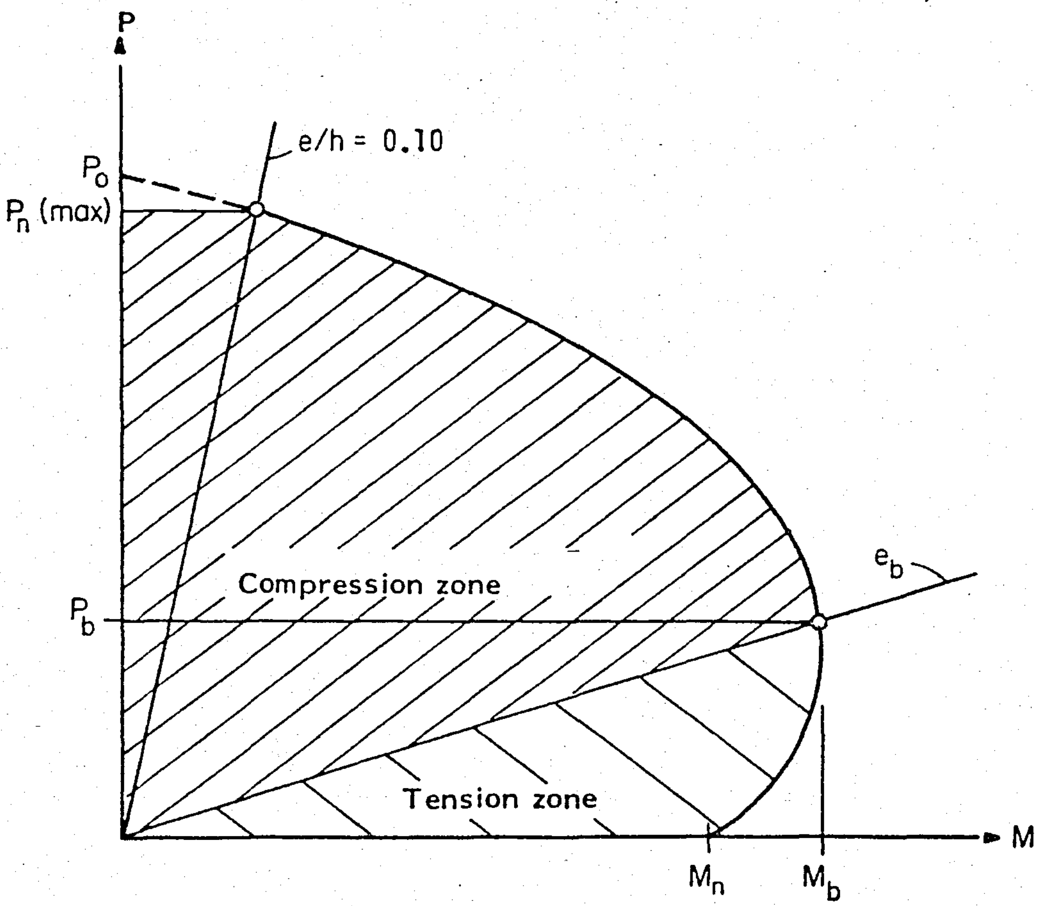


FIGURE 2.2 General Form of Load-Moment Interaction Strength

$$C_b = \frac{0.003}{0.003 + 0.002} \times d \tag{2.3}$$

or multiplying by E_s

$$C_b = \frac{600}{600 + f_y} \times d \tag{2.4}$$

For $f_c' < 30$ MPa, $a_b = 0.85\ C_b$

$$N_{c1} = 0.85.\ f_c'.b.a_b \tag{2.5}$$

$$\varepsilon_s' = \frac{C_b - d'}{C_b} \times 0.003 \tag{2.6}$$

If $\varepsilon_s' > \varepsilon_y$ than compression steel stress $f_s' = f_y$ otherwise $f_s' = \varepsilon_s' \times E_s$

Then,

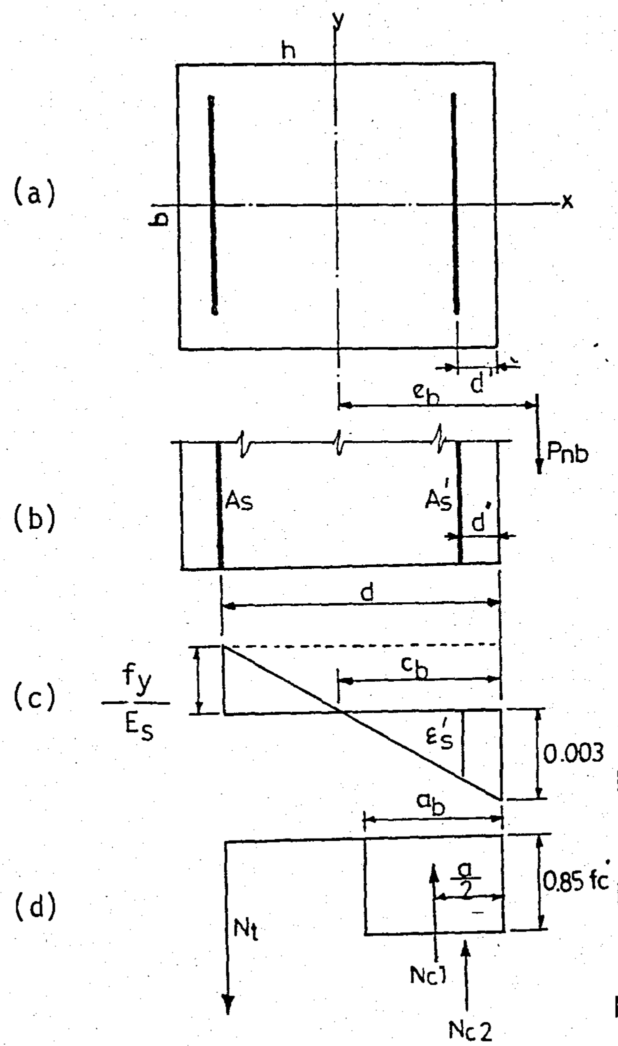
$$N_{c2} = A_s' \times (f_s' - 0.85\ f_c') \tag{2.7}$$

$$N_t = A_s \times f_y \tag{2.8}$$

These three forces are in equilibrium with P_b.

$$\Sigma F_y = 0 = P_b + N_t - N_{c1} - N_{c2}$$

$$P_b = N_{c1} + N_{c2} - N_t \tag{2.9}$$



a_b = depth of equivalent stress block at balanced condition

A_s = Area of tension reinforcement

A_s' = Area of compression reinforcement

b = width of section

C_b = distance from extreme compression to neutral axis at balanced condition

d = effective depth

d' = distance from extreme compression member to centroid of compression reinforcement

e_b = eccentricity at balanced condition

h = section depth

N_{c1} = resultant compressive force in concrete

N_{c2} = resultant compressive force in steel

N_t = resultant tensile force

FIGURE 2.3 Balanced Column Load

(a) Column Cross Section (b) Side View of Column (c) Strain Distribution (d) Resulting Forces

ΣM about plastic centroid of column = 0

$$N_t \times (h/2-d') + N_{c1} \times (h/2-a_b/2) + N_{c2} \times (h/2-d') - P_b \times e_b = 0$$

$$M_b = P_b \times e_b = N_t \times (h/2-d') + N_{c1} \times (h/2-a_b/2) + N_{c2} (h/2-d') \qquad (2.10)$$

If column steel is distributed along four column faces, the numbers used in finding P_b would be increased by an additional term for each group of bars falling at different distances from the neutral axis. Bars very near the neutral axis will not be effective in carrying stress, for that combination of M and P any bars near the axis will have stresses lower than the yield stress. For any given neutral axis one should sketch the strain distribution to establish the status of nearby bars. The deformation sketch is quite simple to use whenever c is known or assumed. It also facilitates the inclusion in an analysis of bars with f_s or f_s' values less than f_y.

2.1.3 Other Loading Conditions

Points below the balanced loading representing primary failures in tension steel past the yield strain and ε_c still 0.003, are easily found with assumed c values smaller than c_b.

Curve points above P_b can be established by using c values greater than c_b with $\varepsilon_c = 0.003$. As c increases the tensile steel stress must drop and the failure is in primary compression.

All columns are required (1) to contain longidudinal bars sufficient to make the steel ratio, $\rho = A_s/A_g$, at least 0.01, because of the shrinkage and creep stresses on smaller, areas, and ρ must not exceed 0.08. At 0.08 crowding in the member is very severe. A family of curves for a section with given dimensions and locations but different amounts of longitudinal steel can be plotted on the same chart as shown in Fig. 2.4. For a given steel ratio, there will be one and only one continuous interaction curve corresponding to that steel ratio. As the longitudinal steel ratio is increased, the curve moves farther from the origin of the interaction diagram.

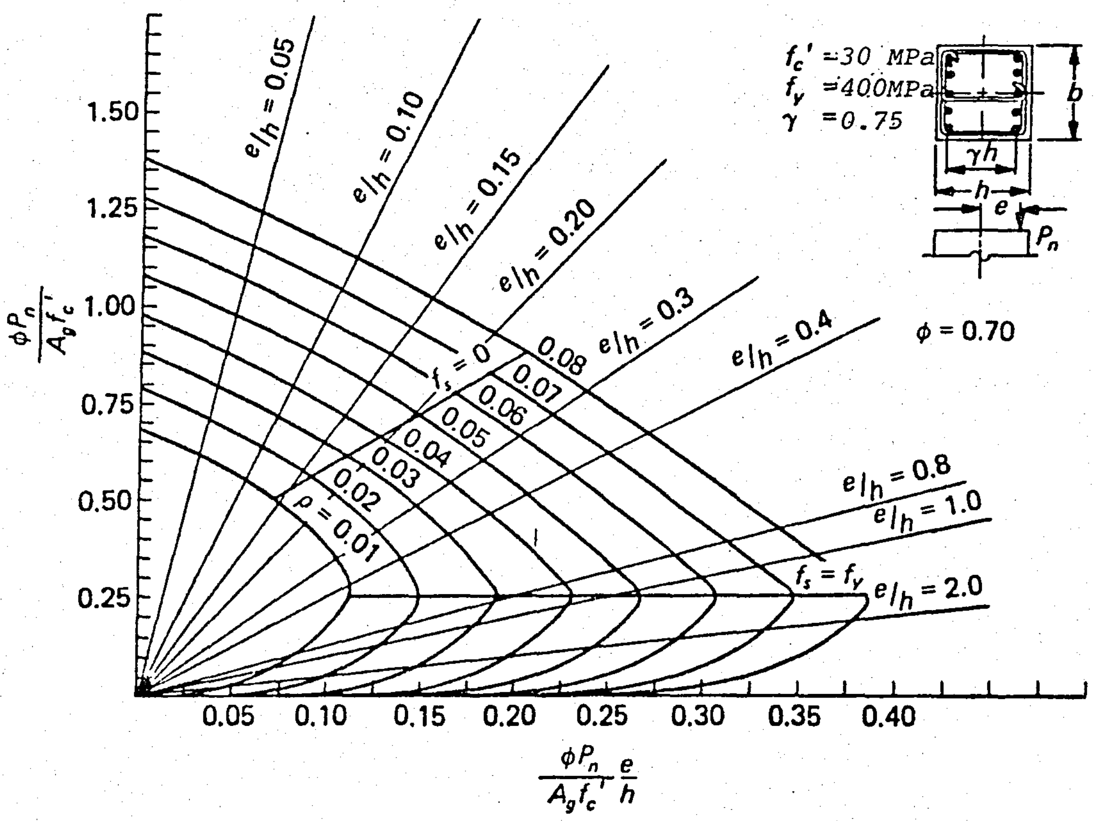


FIGURE 2.4 Interaction Diagram of a Section with Different Steel Ratios ρ.

2.2 BIAXIAL LOADING

2.2.1 General Considerations

A uniaxial interaction diagram defines the load moment strength in a single plane of a section under an axial load ,P, and a uniaxial moment, M. Many columns are subject simultaneously to moments about both major axes, especially corner columns. The biaxial bending resistance of an axially loaded column can be represented schematicly (see Fig. 2.5) as a surface formed by a series of uniaxial interaction curves drawn radially from the P axis. Data for these intermediate curves are obtained by varying the angle of the neutral axis with respect to the major axis (see Fig. 2.6).

When the position of the neutral axis is known or assumed, the magnitude of the load P_u and the components of bending moments M_x and M_y which result in the prescribed limit strain, can be determined using equations of equilibrium. When the position of the neutral axis is not known, the equations of equilibrium can be solved only by the method of successive approximations. All such procedures involve more or less tedious cycles of numerical calculations. The extensive calculations are compounded when minimization of the reinforcement or cross section is sought.

For uniaxial bending, it is customary to utilize design aids in the form of interaction curves or tables. However, for biaxial bending, because of the voluminous nature of the data and the difficulty in multiple interpolations, the development of interaction curves or tables for the various ratios of bending moments about each axis is impractical.

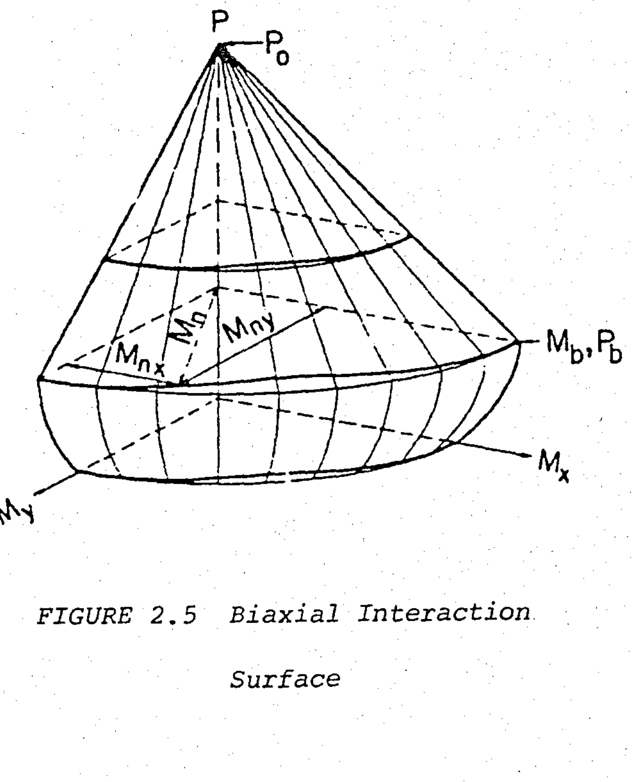


FIGURE 2.5 Biaxial Interaction Surface

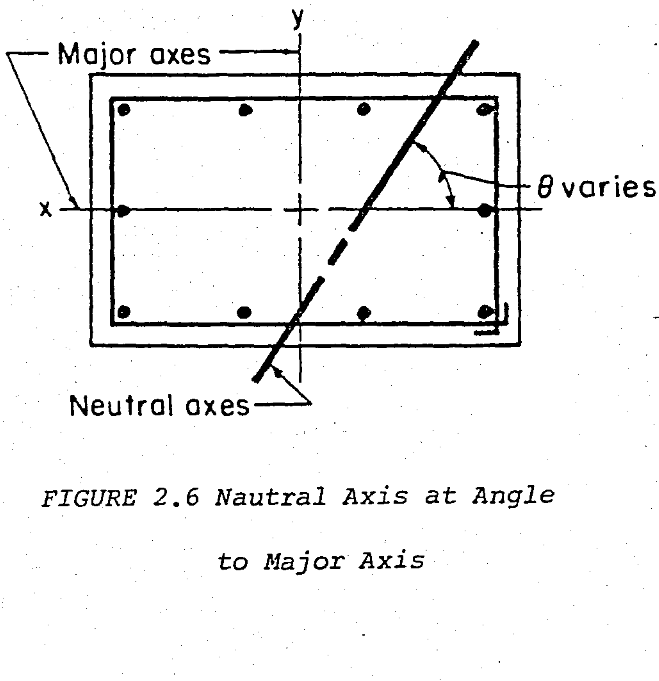


FIGURE 2.6 Nautral Axis at Angle to Major Axis

Instead, several approaches (based on acceptable approximations) have been developed that relate the response of a column in biaxial bending to its uniaxial bending resistance about each major axis.

The nominal strength of a section under biaxial bending and compression is a function of three variables P_n, M_{nx} and M_{ny} which may be expressed in terms of an axial load acting at eccentricities $e_x = \frac{M_{ny}}{P_n}$ and $e_y = \frac{M_{nx}}{P_n}$ as shown in Fig. 2.7. A failure surface may be described as a surface produced by plotting the failure load p_n as a function of its eccentricities e_x and e_y or of its associated bending moments M_{ny} and M_{nx}.

Three types of failure surfaces have been defined. The basic surface S_1 is defined by a function which is dependent upon the variables P_n,e_x and e_y as shown in Fig. 2.8. A reciprocal surface can be derived from S_1 in which the reciprocal of the nominal axial load P_n is employed to produce surface S_2 $(1/P_n, e_x, e_y)$ as illustrated in Fig. 2.9. The third type of failure surface, shown in Fig. 2.10, is obtained by relating the nominal axial load P_n to moments M_{nx} and M_{ny} to produce surface S_3 (P_n, M_{nx}, M_{ny}). Failure surface S_3 is the three dimensional extension of the uniaxial interaction diagram previously described.

2.2.2 Bresler Reciprocal Load Method

A number of investigators have made approximations for both S_2 and S_3 failure surfaces for use in design and analysis. The simplest and the general one of these approximations is the Bresler Reciprocal Load Method.

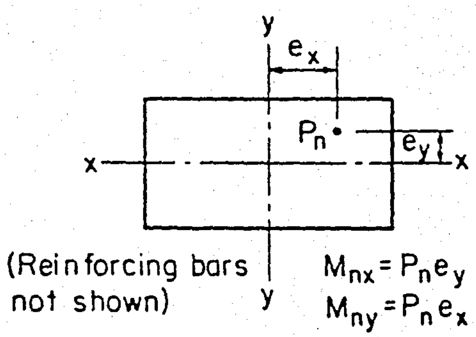


FIGURE 2.7 Notation for Biaxial Loading

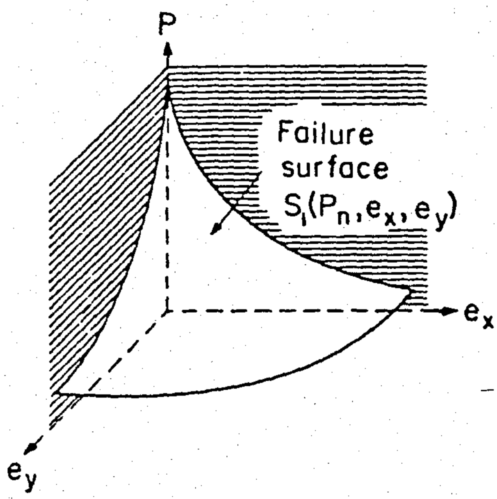


FIGURE 2.8 Failure Surface S_1

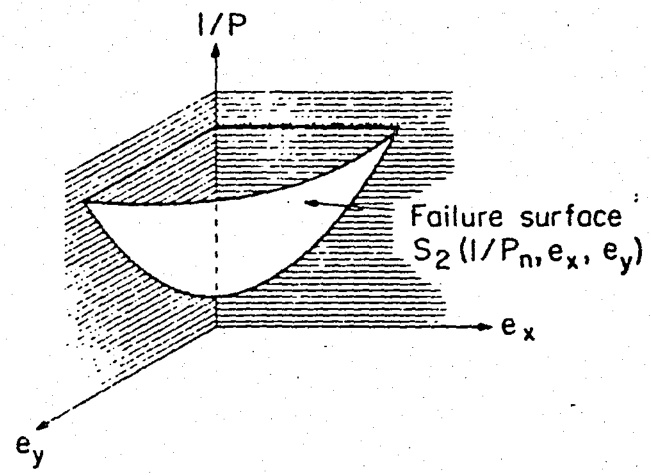


FIGURE 2.9 Reciprocal Failure Surface S_2

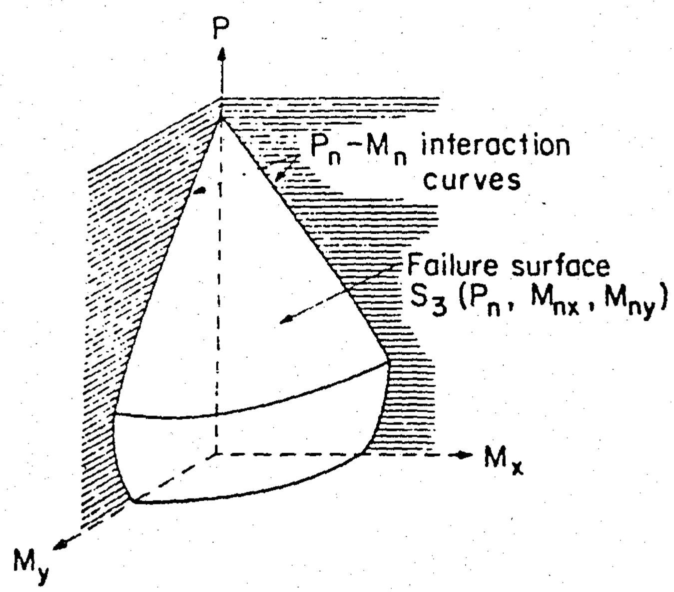


FIGURE 2.10 Failures Surface S_3

This method approximates the ordinate $1/P_n$, on the surface S_2 $(1/P_n, e_x, e_y)$ by a corresponding ordinate $1/P_n'$ on the plane S_2' $(1/P_n', e_x, e_y)$, which is defined by the characteristic points A,B and C as indicated in Fig. 2.11. For any particular cross section, the value P_o (corresponding to point C) is the load strength under pure axial compression; P_{ox} (corresponding to point B) and P_{oy} (corresponding to point A) are the load strengths under uniaxial eccentricities e_y and e_x, respectively. For every point on the surface S_2 $(1/P_n, e_x, e_y)$, there is a corresponding plane S_2' $(1/P_n', e_x, e_y)$. The approximation of S_2 involves an infinite number of planes, each one applicable only for particular values of eccentricities e_x and e_y, and thus each plane defines only one point $1/P_n'$ which serves as an approximation to $1/P_n$. Each point on the true surface is approximated by a different plane; therefore, the entire surface is approximated using an infinite number of planes.

The general expression for any values of e_x and e_y when derived yields the following equation :

$$\frac{1}{P_n} \sim \frac{1}{P_n'} = \frac{1}{P_{ox}} + \frac{1}{P_{oy}} - \frac{1}{P_o} \tag{2.11}$$

Rearranging the variables yields:

$$P_n = \frac{1}{(1/P_{ox}) + (1/P_{oy}) - (1/P_o)} \tag{2.12}$$

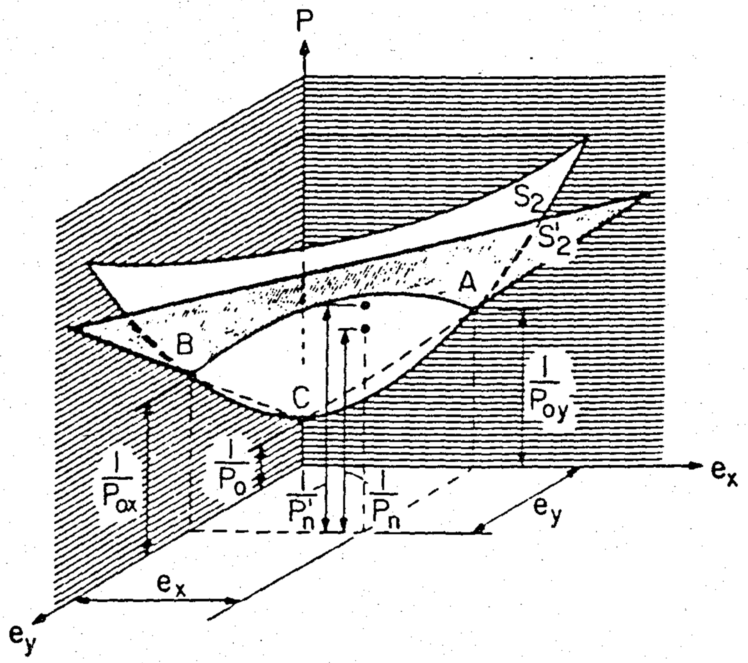


FIGURE 2.11 Reciprocal Load Method

This equation is simple in form and the variables are easily determined. Axial load strengths P_o,P_{ox} and P_{oy} are shown in Fig. 2.12 . As an approximate method it is one of the best when resulting P_n is greater than $0.1\ f'_c\ A_g$. If P_n is lower than the balanced design level ($0.10\ f'_c\ A_g$ level) the errors by this method can increase. In typical cases it is then on the safe side to design for biaxial moment alone, since tension failure then controls.

2.3 SLENDERNESS EFFECTS

2.3.1 General Considerations

Design of compression members shall be based on forces and moments determined from analysis of the structure. Such analysis take into account influence of axial loads and variable moment of inertia,member stiffness and fixed-end moments, effect of deflection on moments and forces, and the effects of duration of loads.

In lieu of the procedure prescribed above, slenderness effects in compression members may be evaluated in accordance with the approximate procedure presented below.

2.3.2 Approximate Evaluation of Slenderness Effects

The approximate moment magnification procedure is similar to the method used for structural steel design. The moment magnifier δ. is a function of the ratio of the axial load to the critical or buckling load of the column., the ratio of the moments at the ends

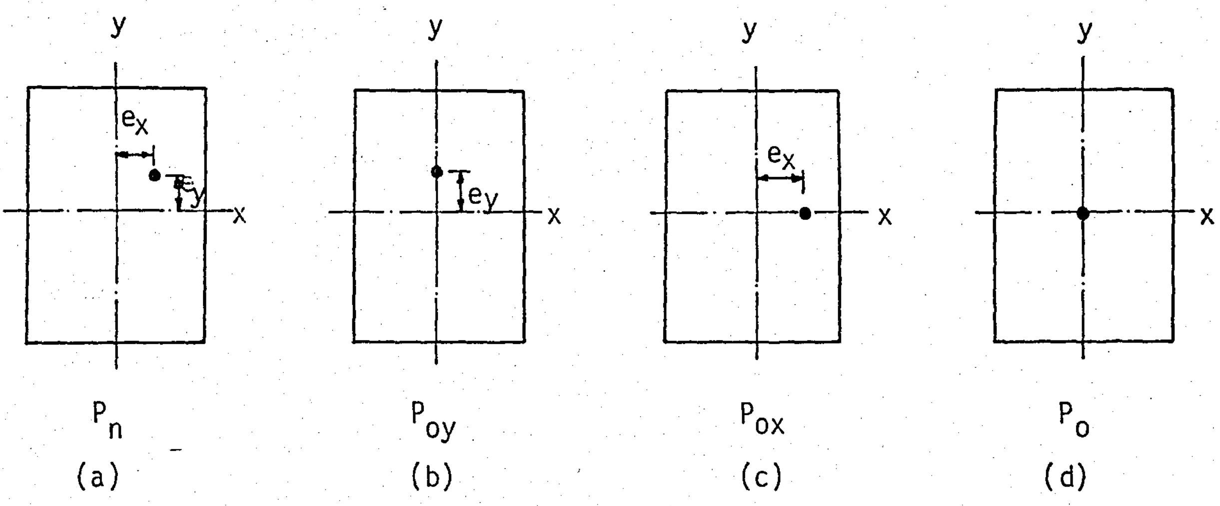


FIGURE 2.12 Bresler's biaxial loading (a) Biaxial moments (b) Eccentricity about X axis (c) Eccentricity about Y axis (d) Axial load alone

of the column, and the deflected shape of the column.

The objective of column design is the selection of a cross section with reinforcement for a specified combination of factored axial load ,P_u, and factored moment, M_u. A column is said to be slender if its cross-section dimensions are small in comparison with its length. The degree of slenderness is expressed in terms of the slenderness ratio,$k\ell_u/r$, where k is the effective length factor which is dependent on end conditions of the compression member and bracing aginst sideway, and r is the radius of gyration of the cross-section of the member. Concepts of three ranges of slenderness ratios are given along with column design methods proposed for each range.

More than 90 percent of the columns in braced frames and 40 percent in unbraced frames fall into the classification in which secondary moments can be disregarded and only the axial load and primary moment used to select the cross section.

Within moderate slenderness limits, the approximate analysis based on a moment magnifier is suggested. Whenever the slenderness of a column or member exceeds moderate slenderness a more rational second-order analysis is required. No upper limit for slenderness are given. When high slenderness ratios are encountered. The analysis shall take into account the design procedure prescribed above. Slenderness effects are considered for both braced and unbraced frames.

a. Decision For Type of Frame

Secondary moments due to deflection of the member , greatly depend on bracing against sidesway. A structure may be assumed braced if it is supported by bracing elements (shearwalls, shear trusses, or other types of lateral bracing) and the following expression is realized (7).

For $n > 4$, where n is number of stories in building

$$H \frac{\Sigma P_u}{\Sigma E_c I_g} < 0.6 \tag{2.13a}$$

For $1 < n < 4$

$$H \frac{\Sigma P_u}{\Sigma E_c I_g} < 0.2 + 0.1 n \tag{2.13b}$$

b. Unsupported and Effective Lengths

The unsupported length , ℓ_u , of a compression member is to be taken as the clear distance between lateral supports as shown in Fig. 2.13. It is also to be understood that the length ℓ_u may be different in each of the principal axes of the compression members. The radius of gyration may be taken as 0.3 of the overall dimension of a rectangular section (1) (Fig. 2.14).

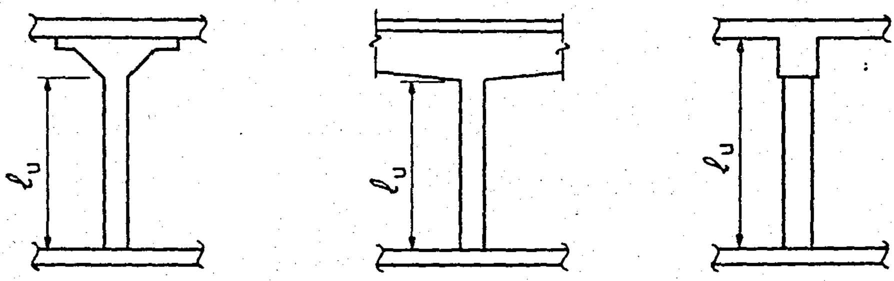
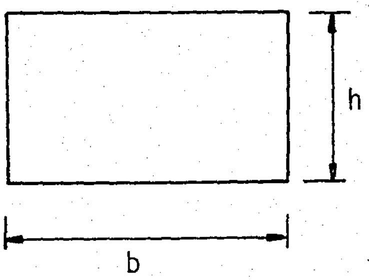


FIGURE 2.13 Unsupported Length (ℓ_u).



$$r = 0.3\ h \qquad (2.14)$$

$$r = 0.3\ b$$

FIGURE 2.14 Radius of Gyration

A short column may fail due to a combination of moment and axial load which may exceed the strength of the cross-section. This type of failure is known as a "material failure". As an illustration, consider the column shown in Fig. 2.15. The column has a deflection Δ which will cause an additional moment in the column. In the free body diagram, it can be seen that the maximum moment in the column occurs at section A-A and this is equal to the applied moment plus the moment due to the deflection, that is $M = P(e + \Delta)$. In the interaction curve, the failure of a short column occurs at any point along the curve depending on the combination of moment and axial load applied. As mentioned above, same deflections would occur and a "material failure" would result when the load P and $M = P(e + \Delta)$ combination intersects the particular cross-section interaction curve. If the column is very slender, it may reach a deflection due to the axial force P, and moment, $P.e$, such that deflections can increase indefinitely with small increases in load, P. The change in moment occurs without any increase in load. This type of failure is known as a "stability failure" and may occur in a slender column.

The basic information on the behavior of straight, concentrically loaded slender columns was developed by Euler more than 200 years ago. It states that a member will fail by buckling at the critical load $P_c = \pi^2 EI/(\ell_e)^2$, where ℓ_e is the effective length $k\ell_u$. For a very stocky column, the value of buckling load calculated from this equation exceeds the direct crushing strength. For more slender members, that is for larger $k\ell_u/r$ values, the failure occurs by buckling, with buckling load decreasing for greater slenderness (Fig. 2.16). Hence a family of slender column interaction diagrams for members of varying

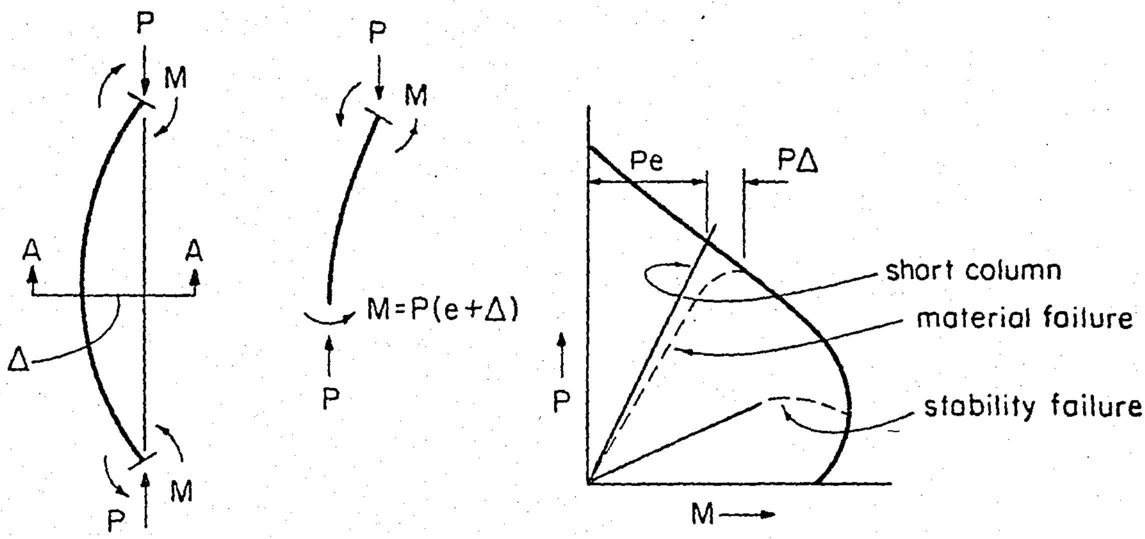


FIGURE 2.15 Interaction in Slender Columns

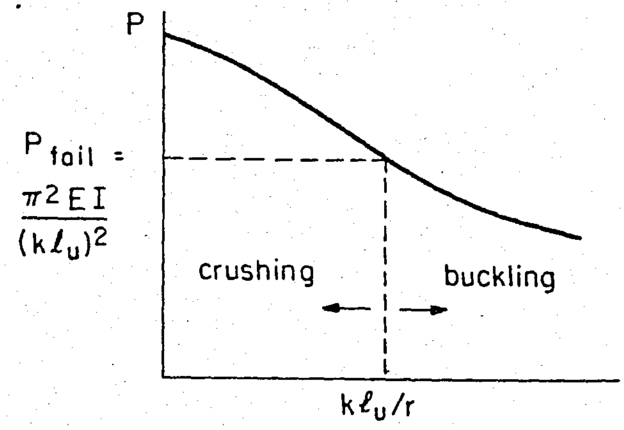


FIGURE 2.16 Column Curve

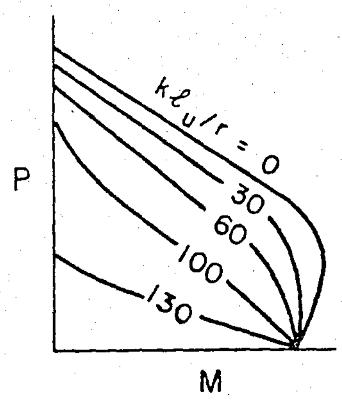


FIGURE 2.17 Slender Column Interaction Diagrams

slenderness ratios can be developed as shown in Fig. 2.17. The interaction for $k\ell_u/r = 0$ is that which corresponds to the combination of moment and loads for a particular section with reinforcement as in a short column. The shape of the interaction curves for higher $k\ell_u/r$ values is dependent on the moments applied to the column.

In the critical load given by the Eular equation an originally straight member buckles into half sine wave as shown in Fig. 2.18(a). In this configuration, bending moment $P.\Delta$ acts at any section where Δ is the deflection at that point. This deflection continues to increase until the bending stress caused by the increasing moment, together with the original compression stress, exceeds the compressive strength and the member fails. The effective length ℓ_e ($k\ell_u$) is between pinned ends, zero moments or inflection points, and in this case is equal to the unsupported length ℓ_u. If the member is fixed against rotation at both ends, as shown in Fig. 2.18 (b), it will buckle in the shape shown. Inflection points will occur as shown and the effective length ℓ_e ($k\ell_u$) will be one half of the unsupported length. When Euler's equation is applied to this column, the column will carry four times as much load as when ends are hinged. Rarely are columns in real structures either hinged or fixed, rather they are partially restrained against rotation by abutting members and thus the effective length will occur between $\ell_u/2$ and ℓ_u as shown in Fig. 2.18(c). The precise value will depend on the rigidity of the members abutting the column.

A compression member that is fixed at one end and entirely free at the other end would buckle as shown in Fig. 2.19(a). The upper end would move laterally in respect to the lower. This is known as sidesway. The inflection points would occur at the upper end of the member and thus

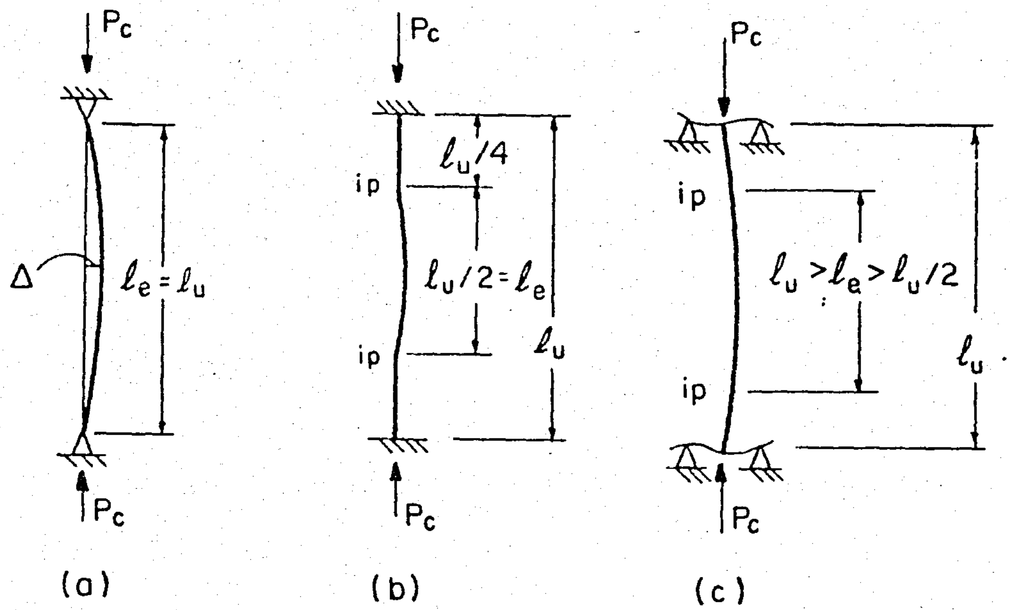


FIGURE 2.18 Effective Length ℓ_e (sidesway Prevented)

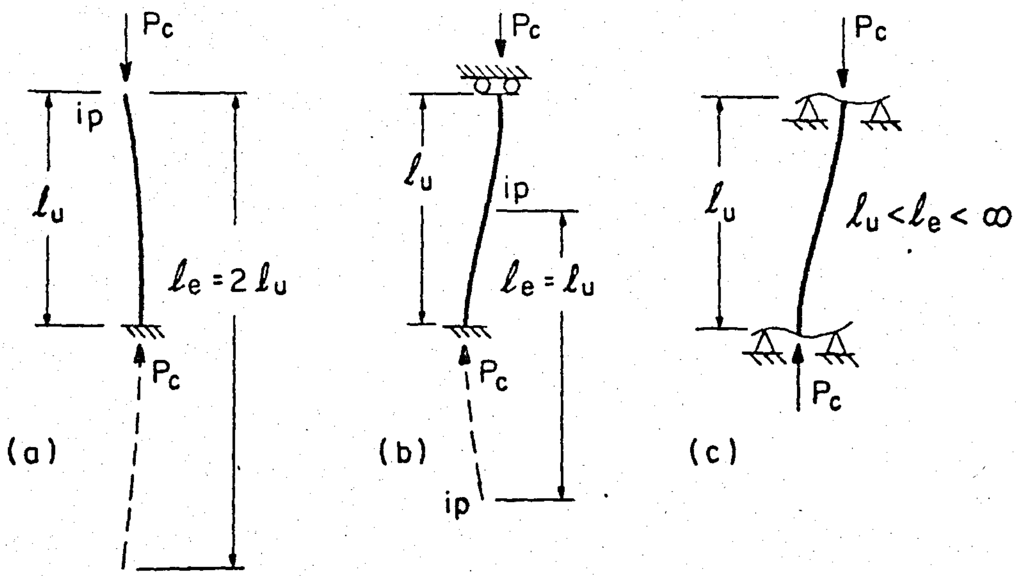


FIGURE 2.19 Effective Length ℓ_e (sidesway Not Prevented)

would be similar to the upper end of the sine curve. The effective length would be twice the height. If the column is fixed against rotation at both ends but one end can move laterally, it will buckle as shown in Fig. 2.19(b). The effective length would be equal to the height with an inflection point occuring as shown. If the buckling load of the column in Fig. 2.19(b) were compared to that of the column in Fig. 2.18(b) which is braced against sidesway, it would be only a quarter of wheh sidesway is permitted. Again, rarely are the ends of columns either hinged or fixed, but rather they are partially restrained against rotation by abutting members and thus the effective length, where sidesway is not prevented, will vary between ℓ_u and ∞ as shown in Fig. 2.19(c). If, on the other hand, the beams are fairly flexible, a hinged condition is approached at both ends and the structure would not be very stable.

In summary, following comments can be made :

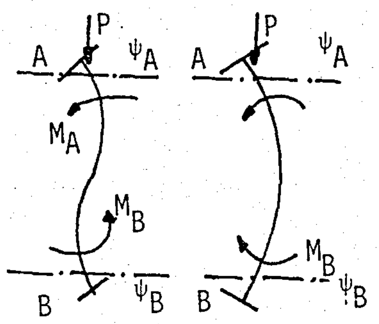
1. For columns braced against sidesway, the effective length falls between $\ell_u/2$ and ℓ_u, where ℓ_u is the actual unsupported length of column.
2. For columns not braced against sidesway the effective length is always longer than the actual length of the column ℓ_u and may be more like $2\ell_u$ and higher. A value of ℓ_e or $k\ell_u$ less than 1.2 for columns not braced against sidesway normally would not be realistic.

In computing the effective length factors for braced and unbraced members, the equations taken from the 1972 British Code of practice can be used(2).For braced compression members, an upper bound to the effective length factor may be taken as the smaller of the following two expressions,

$$k = 0.7 + 0.05\,(\psi_A + \psi_B) = 1.0 \qquad (2.15a)$$

$$k = 0.85 + 0.05\,\psi_{min} = 1.0 \qquad (2.15b)$$

where ψ_A and ψ_B (Fig. 2.20) are the values of ψ at the ends of the column and ψ_{min} is the smaller of two values



$$\psi = \frac{\Sigma \dfrac{EI}{\ell}\ \text{cols}}{\Sigma \dfrac{EI}{\ell}\ \text{beams}} \qquad (2.16)$$

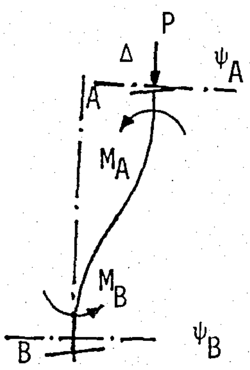
FIGURE 2.20 Ratio of Relative Stiffnesses (sidesway Prevented)

For unbraced compression members restrained at both ends, the effective length may be taken as (Furlong Equation) (2) :

$$\text{For}\quad \psi_a < 2, \qquad k = \frac{20 - \psi_a}{20}\ \sqrt{1 + \psi_a} \qquad (2.17)$$

$$\text{For}\quad \psi_a > 2, \qquad k = 0.9\sqrt{1 + \psi_a} \qquad (2.18)$$

where ψ_a is the average of the ψ values (Fig. 2.21) at the two ends of the compression member.



$$\psi = \frac{\Sigma\ EI/_{\ell}\ \text{cols}}{\Sigma \frac{EI}{\ell}\ \text{beams}}$$

FIGURE 2.21 Ratio of Relative Stiffnesses (Sidesway Not Prevented)

c. Consideration of Slenderness Effects (1)

For compression members braced against sidesway, the effects of slenderness may be neglected when $k\ell_u/r$ is less than 34-12 M_{1b}/M_{2b}. For compression members not braced against sidesway, the effects of slenderness may be neglected when $k\ell_u/r$ is less than 22.

The upper limit for compression members which may be designed by the approximate method is $k\ell_u/r$ equal to 100. When $k\ell_u/r$ is greater than 100, an analysis which takes into account the influence of axial loads and variable moment of inertia on member stiffness and fixed-end moments, effect of deflections on the moments and forces, and the effects of the duration of the loads, must be used.

d. Moment Magnification

Compression members shall be designed using the factored axial load P_u from a conventional frame analysis and a magnified factored moment M_c defined by

$$M_c = \delta_b M_{2b} + \delta_s M_{2s} \tag{2.19}$$

where

$$\delta_b = \frac{C_m}{1 - \dfrac{P_u}{\phi P_c}} > 1.0 \tag{2.20}$$

$$\delta_s = \frac{1}{1 - \dfrac{\Sigma P_u}{\phi \Sigma P_c}} > 1.0 \tag{2.21}$$

and

$$P_c = \frac{\pi^2 EI}{(k\ell_u)^2} \tag{2.22}$$

ΣP_u and ΣP_c are the summations for all columns in a story. For frames not braced against sidesway, both δ_b and δ_s shall be computed. For frames braced against sidesway, δ_s shall be taken as 1.0. In calculation of P_c, k shall be computed according to equations (2.15a) and (2.15b) for δ_b and according to equations (2.17) and (2.18) for δ_s.

In lieu of a more accurate calculation, EI in Eq.(2.22) may be taken either as

$$EI = \frac{(E_c\ I_g/5) + E_s\ I_{se}}{1 + \beta_d} \tag{2.23}$$

or conservatively

$$EI = \frac{E_c\ I_g/2.5}{1 + \beta_d} \tag{2.24}$$

In Eq. (2.20), for members braced against sidesway and without transverse loads between supports C_m may be taken as

$$C_m = 0.6 + 0.4\ \frac{M_{1b}}{M_{2b}} \tag{2.25}$$

but not less than 0.4.

For all other cases, C_m shall be taken as 1.0.

If computations show that there is no moment at both ends of a braced compression member or that computed end eccentricities are less than (15 + 0.03 h) mm, M_{2b} in Eq.(2.19) shall be based on a minimum eccentricity of (15 + 0.03 h) mm about each principal axis seperately. Ratio M_{1b}/M_{2b} in Eq. (2.25) shall be determined by either of the following :

(a) When computed end eccentricities are less than (15 + 0.03 h) mm, computed end moments may be used to evaluate M_{1b}/M_{2b} in Eq. (2.25)

(b) If computations show that there is essentially no moment at both ends of a compression member, the ratio M_{1b}/M_{2b} shall be taken equal to one.

If computations show that there is no moment at both ends of a compression member not braced against sidesway or that computed end eccentricities are less than (15 + 0.03 h) mm, M_{2s} in Eq. (2.19) shall be based on a minimum eccentricity of (15 + 0.03 h) mm about each principal axis separately.

For compression members subject to bending about both principal axes, moment about each axis shall be mangified by δ, computed from corresponding conditions of restraint about that axis.

III. PROGRAMMING

The arrival of the personal microcomputer at homes and design offices of structural engineers brings a number of exciting opportunities and challenges for improved productivity, and better designs, from standpoints of both accuracy and economy. The calculations required for rectangular reinforced concrete columns are complex and lengthy, consequently various design aids have been published to simplify calculations, or to nearly eliminate them.

These design aids are certanly worthwile, but are, in some cases limited.Throughthe use of a computer program such as this the engineer is afforded maximum creativity, yet spared the necessity of repetitive calculations.

This program was developed for an Apple II+computer with 48 K bytes Random-Access-Memory. The conversion to other versions of the basic programming language should be straight-forward. The program is self - explanatory. This is accomplished by inserting REM statements at the begining of each subroutine.

3.1 GENERAL METHOD

The program is based on two relatively simple ideas. First, for a given reinforced concrete cross section, if the depth to neutral axis at the strength condition compression face strain of 0.003, is known, the corresponding axial load and eccentricity (or moment) can be computed by the strength design methods of (ACI 318M-83). Second, the solution to a variaty of problems can be reached through the method of interval halving technique, a successive approximation procedure. With these two ideas in mind, a solution scheme can be developed based on the idea of the load-moment interaction diagram, and framed in the context of either analysis or design.

Load-moment interaction diagram of Fig. 2.2 can be idealized as shown by the curve O-A-B-C of Fig. 3.1

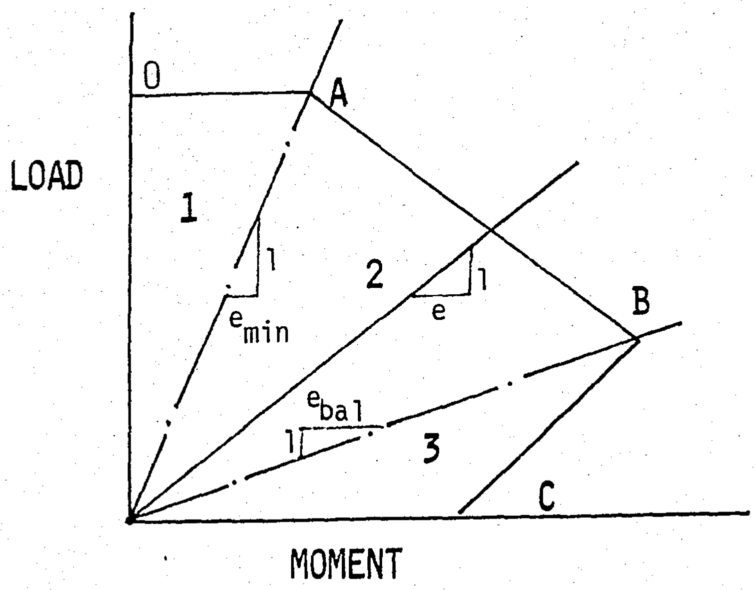


FIGURE 3.1 Idealized interaction diagram

The horizontal line O-A represents the upper limit of usable strength (Eq. 2.1) recognized by(ACI 318M-83), Building code Requirements for Reinforced Concrete (1).The commentary to (ACI 318M-83) suggests that point "A" is essentially the same as the point on the interaction diagram corresponding to load eccentricity of h/10. Line A-B represents compression failure, where point "B" is the condition of balanced failure. Line B-C represents tension failure where point "C" represents the section capacity in pure flexure. Region 1 then represents a minimum eccentricity, or maximum load criterian, region 2 is a zone of compression failure, and region 3 corresponds to tension failure.

Fig. 3.1 might represent the theoretical, or "nominal" capacity of a particular column section. For design, the capacity should be reduced by some factor (as shown in Fig. 2.1), 0.7 for tied columns as explained before. Fig. 2.1 can be idealized in the form of Fig. 3.2.

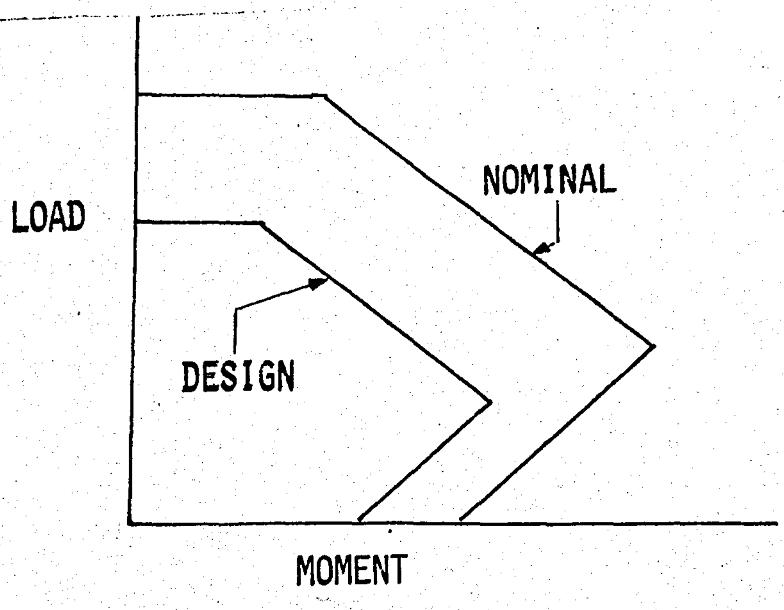


FIGURE 3.2 Nominal and Design Interaction Diagrams.

For reinforced concrete columns, a particular cross section and reinforcement layout will be adequate for a load and moment combination if a point which represents the factored design loads lies inside the design interaction curve. Thus, the checking process involves a particular column, and determining if the load and moment point falls inside the curve. A similar approach is taken for design. A trial section is first assumed, it is checked, and if found not adequate, a revised section with greater capacity considered. If the section is adequate, it is accepted, but the search continues for a more economical section within particular design constraints.

3.1.1 Determination of Interaction Diagram

The interaction diagram for a particular rectangular cross section depends on concrete and steel strength, section geometry, steel amount and layout. Any combination of axial load and moment can be treated as an eccentric axial load with the same magnitude, since M = P.e. To check a given load case it is convenient to determine the column capacity at the particular eccentricity.

The approach taken in the program is to find the resultant load and moment (eccentricity) corresponding to a particular assumed neutral axis in the rectangular cross-section. This requires that force magnitudes and resultants be determined for the concrete, and various reinforcing elements, as shown in Fig. 3.3. A subprogram is provided to evaluate the resultant load and eccentricity for an assumed neutral axis in the rectangular cross-section.

For the purposes of this program, the steel reinforcement is not considered as individual bars, but rather the steel is smeared throughout the section as an equivalent "I" shape as shown in Fig. 3.3 (b). The symbols indicated on this figure correspond to the identifier names used to code the program in Applesoft Basic. They are described in the dictionary of identifiers which is given in Appendix III. The amount of side steel at yield stress level is calculated by proportioning the depth of uniform stress level to the depth of web of "I" shape reinforcement.

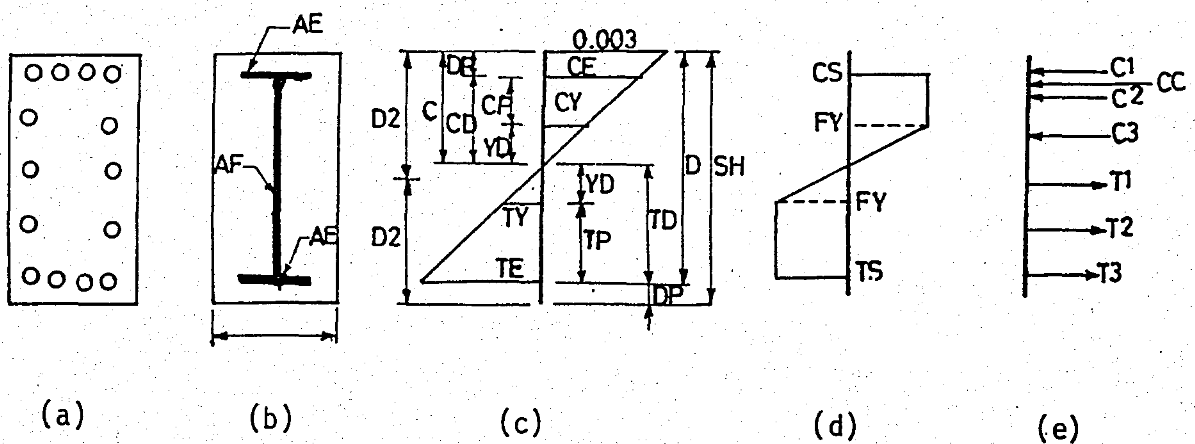


FIGURE 3.3 Cross Section with Strain Diagram and Force Resultants
(a) Actual bar layout (b) Idealized reinforcement
(b) Strain diagram (d) Steel stresses (e) Force resultants

For a given neutral axis position, the resultant load and moment (eccentricity) can be calculated, considering Fig.3.3, as the following:

critical distances :

$H2 = SH/2$ 3.1 (a)

$CD = C-DP$ 3.1 (b)

$TD = D-C$ 3.1 (c)

$YD = EPY \times C/K3$ 3.1 (d)

$JD = D-DP$

$CP = CD-YD$ If $CP<0$ then $CP = 0$ 3.1 (f)

$TP = TD-YD$ If $TP<0$ then $TP = 0$ 3.1 (g)

Critical Strains:

$CE = CD/C \times K3$ 3.2 (a)

$TE = TD/C \times K3$ 3.2 (b)

$CF = EPY$, If $CP = 0$ THEN $CF = CE$ 3.2 (c)

$IF = EPY$, If $TP = 0$ THEN $IF = TE$ 3.2 (d)

Outer Face Steel Stresses

$CS = K29 \times CE$ If $CS>FY$ THEN $CS = FY$ 3.3 (a)

$TS = K29 \times TE$ If $TS>FY$ THEN $TS = FY$ 3.3 (b)

Force Resultants :

a) Compressive :

$C1 = (CS - 0.85 \times FPC) \times AE$ 3.4 (a)

$C2 = (FY - 0.85 \times FPC) \times AF \times CP/JD$ 3.4 (b)

$C3 = CS/2 \times AF \times (CD - CP)/JD$ 3.4 (c)

$CC = 0.85 \times Z1 \times FPC \times SB \times C$ 3.4 (d)

b) Tensile :

$$T1 = TSxAE \qquad 3.5\ (a)$$

$$T2 = FY \times AF \times TP/JD \qquad 3.5\ (b)$$

$$T3 = (TS/2) \times AF \times (TD-TP)/JD \qquad 3.5\ (c)$$

c) Axial Force

$$P = CC + C1 + C2 + C3 + - T1 - T2 - T3 \qquad 3.6$$

d) Moment

$$M = CC \times (H2 - Z1 \times C/2) + C1 \times (H_2 - DP) \qquad 3.7$$
$$C2 \times (H2 - DP - CP/2) + C3 \times (H2 - DP - CP$$
$$- (CD - CP)/3) + T1 \times (H2 - DP) + T2 \times (Hx - DP$$
$$- TP/2) + T3 \times (H2 - DP - TP - (TD - TP/3)$$

e) Eccentricity

$$E = M/P \qquad 3.8$$

If the column is biaxially loaded, the capacity in the other direction is calculated by replacing width to depth, depth to width and flange reinforcement to web, web reinforcement to flange. The web reinforcement, AF is diveded into two, AF/2, and treated as flange reinforcement, the flange reinforcement is multiplied by two, 2 x AE, and it is treated as web reinforcement. (Appendix I, on lines 4450 and 4440 respectively)

3.1.2 Interval Halving Technique

The interval halving is a simple searching algorithm useful where a parameter varies monotonously between two limits. As an example consider finding a number between 1 and 100. Suppose the number that must be found is 36 and the first guess is 50 = (1 + 100)/2 and it is then told the guess is high then uncertainty is halved by making a second guess of 25 = (1 + 50)/2, and the guess is low this time, the interval of uncertainty is halved again. The third guess must be 37 = (25 + 50)/2. After several additional cycles the number will either be guessed exactly, or the remaining limits of the interval will converge to make the answer obvious. This approach is also applicable where a search is made for an answer that is close enough. For example, plus or minus 1 of the right answer might be a tolerable value. So the right number is reached in three guesses. This latter approach is generally necessary when searching noninteger values.

In the program the method of interval halving technique is used in two ways for both analysis and design.

The primary application is in analysis. It is used in the subprogram (Appendix I between lines 3490 and 3650) which determines axial load capacity for a specified eccentricity. Since load capacity decreases with eccentricity, the relation-ship is monotonous. Unfortunately, eccentricity tends toward infinity at very low loads, so it is not convenient to use eccentricity in the interval halving procedure explicitely. Instead the depth to neutral axis, C is used, since there are physical limits to its value. The depth to neutral axis,

C_b, corresponding to a balanced condition is used as the initial value to begin the iteration. If the eccentricity found is greater than the actual value then the C value for the second iteration is increased by the application of internal halving between the first value C_b and the higher value $CL = 1.5 \times D$ then new depth to neutral axis C is $(C_b + CL)/2$.. If the eccentricity is less than the actual value, then the C value for the second iteration is decreased by the application of the interval halving between the first value C_b and a lower value corresponding to pure flexure. These trials continue until finding an eccentricity that is tolerable. In this program, the tolerable limit, is plus or minus 5% of the actual eccentricity. When the trial eccentricity falls between these limits then iteration stops. In the design, interval halving is used to find the appropriate reinforcement for a specified cross section and choice of materials. First, the maximum reinforcement ratio given is checked if it is adequate then interval halving is used between maximum and minimum reinforcement ratios in order to find the appropriate ratio. (Appendix I between lines 4630 and 4690)

3.1.3 Program Capabilities

The program is capable of analysis and design of reinforced concrete columns under axial load and uniaxial or biaxial bending moment when the program is run, main menu will appear on the screen as shown below :

RC. COLUMN 1.0

PROBLEM MENU :

ANALYSIS

1) UNIXAIAL

2) BI-AXIAL

3) CHECK A DESIGN

DESIGN

4) UNIAXIAL, INPUT SECTION

5) UNIAXIAL, FIND SECTION

6) BIAXIAL, INPUT SECTION

7) BIAXIAL, FIND SECTION

ENTER TYPE OF PROBLEM

OR 'φ' TO QUIT .....

It must be entered the number of the choice.

3.2 METHOD OF ANALYSIS

In the analysis menu of the program there are three choices.

1 - Uniaxal bending

2 - Bi-axial bending

3 - Check a design

3.2.1 Analysis

For analysis of any section, the user must input concrete and steel strengths, cross section properties, number, size and layout of reinforcing bars, capacity reduction factor, and the load and moment (or eccentricity) for which the capacity is to be checked.

To make the program as "user-friendly" as possible many of these parameters are established by default, and the user need only change the value as desired the new values become the default condition for succeeding problems.

When the program is run and main menu appear on the screen, "1" for uniaxial analysis and "2" for biaxial analysis must be entered. Then program goes to the subroutine GET LOADS (Appendix I between lines 2890-3140) Capacity reduction factor will appear as 0.7 on the screen unless another value for it is fed in as the new value 1 might be input if the factored loads have been previously divided by the capacity reduction factor. The axial load, PN, and moment, M, or eccentricity, E, are entered. If the analysis is biaxial then moments (or eccentricities) in both x and Y directions are entered. If moments are entered eccentricities, E = M/P, if eccentricities are entered moments, M = P.E, are calculated.

The material properties (Subroutine GET MATERIALS Appendix II between lines 3190-3350) will be selected next. On the screen concrete strength as 30 MPa and steel strength as 400 MPa will appear. If the material properties are different new values must be entered. With these values beta-1 (as Z1) of (1) and steel yield strain E_s(as EPY) are calculated in this subroutine.

Then, section geometry (subroutine INPUT THE SECTION, Appendix I between lines 2560-2650), section width, B, and depth, H, and cover to bar centerline, DP, are entered. Gross area of section, AG = B x H, is calculated.

In order to idealize the reinforcing steel as an embedded "I" section, it is necessary to specify how much of the steel corresponds to the flange, and how much to the web. The term "side bars" is used to describe intermediate steel corresponding to the web. The section of fig. 3.3 has 6 side bars, and 14 total bars. Side bars are assumed to have the same amount of cover as the end bars. The reinforcement of the section (subroutine INPUT STEEL Appendix I between lines 2700-2840) is given in three steps

BAR SIZE (BS)
NUMBER OF BARS (BN)
NUMBER OF SIDE BARS (BF)

As bar sizes, bar numbers of ACI-318-83 are used. In this subroutine, ratio of web reinforcement to total reinforcement, RS = BF/BN, total reinforcement, AS = BN x BS, reinforcement ratio RHO = AS/(B * H), web reinforcement, AF = AS x RS, and flange reinforcement, AE = AS x (1 - RS)/2, are calculated.

In the subroutine "FIND CAPACITY FROM P-M DIAGRAM" (Appendix I, between lines 3410-3670), axial load capacity, P, corresponding to a given eccentricity, E, is calculated as explained in sections 3.1.1 and 3.1.2. If the analysis is uniaxial then axial load capacity P is multiplied by the capacity reduction factor and nominal axial load capacity is found (NN = P x PHI). If the analysis is biaxial, calculated P is accepted as Px (Px = P) and column axial load capacity, PY, in the other direction

(Y direction) is calculated by the same procedure. Pure axial load strength P_O(Eq. 2.2) is calculated (line 4420, Appendix I). Then with Px, Py and Po values on hand, Bresler's Reciprocal Load Equation (Eq. 2.12) is applied (line 2370, Appendix I) and nominal axial load capacity (NN = PHI/(1/Px + 1/Py - 1/p_o)) is calculated.

Output of the analysis routine consists of a summary of section and reinforcement properties, the magnitude of the applied load PN and the section capacity, NN. A message will also be displayed, informing that the design is either "ADEQUATE" when capacity, NN, is greater then applied load, PN, or "NOT ADEQUATE" when NN is less than PN.

3.2.2 Check a Design

As a special case of the analysis routine, there is an option to "check a design". This option is included so that additional load combinations can be tested for a given section with minimal additional input. Additionally slenderness effects can also be checked for the columns where slenderness must be considered. It can be used in the case of sections designed within the program, or entered through the analysis routine.

In programming, an approximate method explained in section 2.3.2 is used for slenderness design. There is a subroutine "DESIGN FOR SLENDERNESS" (Appendix I between lines 4820-8510), the following steps are followed in slenderness design :

a) Braced or Unbraced Frame

There are two options in the program for deciding the type of frame (Appendix I, between lines 4880-5370). First is to input and the second is to find at the end of a series of computations. In the second option Eq. 2.13b is used for decision and it is necessary to input the height of building (BH), total vertical load (VL), number of stories,(NS) types of bracing elements (TE) and the width (B) and depth (D) of bracing elements.

b) Effective Length Factors

For the effective length factors there are two options as well. They are either given or calculated (Appendix I between lines 5420-7330). If the frame is braced only the braced effective length factor of the column designed, if it is unbraced all the unbraced effective length factors of the columns in the story must be calculated.

It is necessary to feed the length, width and depth of the lower, upper and middle columns and upper and lower beams in order to calculate relative end stiffnesses (Eq.2.16). In Eq. 2.16, it is assumed that columns and beams have the same material properties. If the frame is braced Eq.2.15(a) and 2.15(b) and if it is unbraced Eq. 2.17 and 2.18 are used in the calculation of effective length factors.

c) Comparison of Slenderness Criteria

$k\ell_u/r$ ratios for both braced and unbraced frames are computed. This ratio is compared with $34-12\ M_{1b}/M_{2b}$ for braced frames and with 22 for unbraced frames.

If $k\ell_u/r$ is less than the values above,,slenderness may be neglected in the direction consideredand"SLENDERNESS IS NEGLIGIBLE IN X(Y) - DIRECTION" will appear on the screen. If the problem is uniaxial the program will return to the main program and will continue the analysis. If it is biaxial then the moment is not magnified in this direction, other direction is considered.

If klu/r is greater than 100 then the approximate procedure of the program will not be adaquate and "THIS PROGRAM IS INSUFFICIENT IN X(Y) - DIRECTION" will appear on the screen. Then there are three options :

A) CHANGE SECTION

B) CHANGE REINFORCEMENT LIMIT

C) RETURN TO MAIN MENU

One of them must be chosen in order to continue the solution.

If the klu/r values are between 34-12 M_{1b}/M_{2b} for braced or 22 for unbraced and 100 then,

$$34-12\ M_{1b}/M_{2b}(22) < k\ell_u/r \leq 100$$

moments are magnified in the direction considered.

d) Moment Magnification

Moments are magnified in the subroutine MOMENT MAGNIFICATION (Appendix II between lines 7690-8510). If the frame is unbraced then effective length factors (k) must be calculated for the other columns in the story by the same procedure in order to find critical load

Pc (Eq.2.22) of each column. EI value in Eq. 2.22 is obtained by Eq. 2.24 which is more conservative and simple than Eq. 2.23. In order to obtain β_d of Eq. 2.24, dead and live load moments must be input. Total critical load of the story ΣPc is calculated by adding critical loads of each column in the story. Unbraced magnification factor δ_s (Eq. 2.21)is determined by the input of total story load ΣPu.

If the frame is braced, effective length factor of the column considered is enough in order to obtain braced moment magnification factor δ_b (Eq. 2.20). Cm in Eq. 2.20 is calculated by Eq. (2.25). For the braced frame, δ_s is taken as 1.0. Eccentricities corresponding M_{2b} and M_{2s} (in Eq. 2.19) are less than (15 + 0.03 h) mm, then M_{2b} and M_{2s} are replaced with moments corresponding the minimum eccentricity (15 + 0.03 h) mm. seperately.

Moments are magnified by Eq. 2.19. New eccentricity for analysis is obtained by dividing the magnified moment ,Mc, to the axial design load, Pu. If the column is biaxially loaded the same procedure is fallowed for the other (y) direction. Analysis is continued with the new magnified moment and corresponding eccentricity.

3.3 METHOD OF DESIGN

The design capabilities of the program include both, uniaxial and biaxial problems where a cross section has already been established (input section), and cases in which the cross section is established by the program (Find Section).

3.3.1 Input Section

Most design problems with either uniaxial or biaxial bending concern finding the minimum satisfactory reinforcing steel ratio for a specified cross-section and a given load case. In this case, axial load and corresponding moment or eccentricity (Appendix II GET LOADS between lines 2890-3140), material strengths (Appendix II, GET MATERIALS between lines 3190-3390), cross section size (Appendix II between lines 1620-1670) and permissible range of reinforcement (Appendix II between lines 4280-4410) must be input. The default reinforcement ratios are 0.01 and 0.04. This upper limit is lower than ACI maximum for columns to make easily constructible designs. However, as with all the other defaults in this program, these limits can be changed freely. In design the amount of side steel is established by specifying a fraction of the total steel that is to be placed as intermediate bars along the sides of cross-section. The section of Fig. 3.3 would correspond to a ratio of 6/14 = 0.43.

With these values on hand the design starts. First the capacity corresponding to maximum reinforcement is computed if the capacity at maximum reinforcement (NN) is less than the design load (PN) then "INSUFFICIENT CAPACITY AT MAXIMUM REINFORCEMENT" appears on the screen and it is advised to change section or reinforcement limit. If (NN) is greater than (PN) then capacity at minimum reinforcement is computed. If new axial load capacity (NN) is greater than design axial load (PN) then minimum reinforcement is accepted if (NN) is less than (PN) then the program proceeds in order to find appropriate reinforcement ratio by internal halving technique (Appendix II, between lines 4630-4690) when the reinforcement ratio is found then the results are printed. Results of design problems will present total amounts of steel necessary at each face and each side of the section.

3.3.2 Find Section

Here the design problem is to find the smallest satisfactory section within some given range, and with reinforcing ratio specified to be within certain limits. For this type of problems, least depth, greatest depth, width depth ratio, increment of depth and steel ratios and fractions (Appendix I busroutine LEAST SECTION between lines 1840-1970) are input.

First the axial load capacity corresponding to greatest depth and maximum reinforcement is computed. If axial load capacity (NN) is less than applied load (PN) then "NOT ADEQUATE WITH MAXIMUM DEPTH, REINFORCEMENT" will appear on the screen and it is advised to change section or reinforcement limit. If (NN) is greater than (PN) then axial load capacity at minimum depth and maximum reinforcement is computed. If new axial load capacity (NN) is greater than design axial load (PN) then the results are printed. If it is not satisfactory. The depth is increased by increment of depth and program proceeds to find the smallest section adequate with largest reinforcement. When the minimum section that is adequate is found then the reinforcement is computed by interval halving technique (Appendix I, between lines 4630-4690). When optimum reinforcement is found, the results are printed as in section 3.3.1

3.3 SAMPLE PROBLEMS

This section is intended to illustrate the use of the program.

a) Uniaxial Analysis

A 30 x 60 cm. column with 8#25 bars (f_y = 400 MPa); four in each face placed with a cover of 5 cm., has concrete strength of 30 MPa. Is the column section adequate for a load of 2220 kN at an eccentricity of 20 cm. about the strong axis (Sample problem 1, Appendix II)

Upon first running of the program, the main menu will appear on the screen. Since this is a uniaxial analysis, enter "1" and hit the return key.

At this point load information will be entered as shown. The capacity reduction factor for this problem is 0.7, so accept the value by hitting the return key, the choice is made to enter eccentricity rather than moment, and the axial load "2220" and eccentricity "20" are entered.

Once loads are entered, it is time to input the material properties, accept the both values, and proceed.

Next input the section properties as shown, Enter dimensions of width (30), depth (60) and cover to bar centerline (5). Next reinforcement pattern, bar size (25), number of bars (8) and number of side bars (0) is entered.

At this point the screen will clear, and the "SOLVING" message will appear. After a short interval the results will flash on the screen, as shown. Since the capacity of the section is less than the applied load, the column is not adequate. After recording the solution, it is possible to return to the main menu by hitting the return key.

b) Biaxial Design

Design a 65 x 65 cm column with equal steel on all faces, using f'_c = 30 MPa concrete, and f_y = 300 MPa steel. The axial load is 4160 kN, moment about x-axis 1310 kN.m, the y-axis moment is 505 kN.m. For this problem assume the factored loads have been previously divided by the capacity reduction factor, so PHI = 1, (Sample Problem 5 Appendix II)

When the menu appears on the screen, since this is a problem where the section is known, and biaxially loaded, enter "6" and hit the return key.

At this point load information will be entered as shown. The default capacity reduction factor is to be changed to 1, the choice is made to enter moments, rather than eccentricity, and the loads and moments are entered.

After the loads are entered, it is time to input the material properties. Accept the given concrete strength, 30 MPa, but change the steel yield stress to 300 MPa.

Next input the section properties as shown. Enter dimensions of width and depth, accept the default minimum, 0.01 and maximum, 0.04 steel ratios, and type in 0.5 to distribute the steel equally about all four faces of the column. Finally enter a cover of 5 cm.

Once this has been entered, the screen will again clear, and the message "SOLVING" will appear until the solution has been reached. The message will appear to flicker time to time; this is a signal that the program is operating and working toward a solution. Finally the results will appear on the screen as shown.

c) Check a Design

The section designed (Sample Problem 6) has been checked for another load case considering slenderness (Sample Problem 8). After appearing the solution (sample Problem 6), hit the return key in order to return to the main menu. When "3" is typed on the main menu, the analysis part of the menu will only be appear on the secreen. Since this is a biaxially loaded section, enter "2" for biaxial analysis.

Next loads are entered. In order to see the effect of slenderness loads are unchanged. Since the slenderness will be checked print "Y" as the answer to the question "DO YOU WANT TO CHECK SLENDERNESS?".

At this point informations for slenderness will be entered. Since the type of frame will be determined by the program, type "F". Then enter hight of building, total vertical load, number of stories and dimensions of bracing elements. At the end of some computations, the message "BRACED IN X-DIRECTION" and "BRACED IN Y. DIRECTION" will appear. Since the effective length factors, k, will be input, print "I". Then the column dimensions (width, depth and length, braced and effective length factor, larger and smaller end moments, unbraced end moment and unbraced axial load and dead end live lead moments are entered for both, x and y directions. Once these are entered, braced and unbraced moment magnification factors, magnified moment and resulting eccentricity will be appear, as shown.

After this point, ordinary analysis procedure will be carried on with the given design axial load and magnified moments.

IV. SUMMARY AND CONCLUSIONS

The computer program presented is capable of performing design or analysis of reinforced concrete columns, considering slenderness effects also. The load capacity at a particular eccentricity is determined from the load and moment interaction diagram generated for a particular section. Successive approximations are made through the method of interval halving in order to find column capacity from the interaction diagram and appropriate reinforcement ratio.

Since the design procedure complies with ACI 318 M.83 (1) where SI units are used and most of the design parameters such as material properties and capacity reduction factor are fed as input, the design part of the program can be used for designs that comply with the Turkish Standarts, TS 500 (7) which is very similar to ACI 318M-83,as well. The column sections can also be analyzed in complience with TS 500 by adding a new subroutine for the input of reinforcement pattern due to the fact that bar numbers of ACI 318M-83 is different than that of TS 500.

Since about 90 percent of the columns in braced frames and 40 percent in unbraced frames can be designed as short column, slenderness effects are only considered in the analysis part of the program. Nevertheless, slenderness effects can easily be introduced into the design part of the program by calling the slenderness subroutine. It is also possible to incorporate slenderness effects by directly feeding the magnified moments determined from the analysis as input into the design part of the program.

A new approach is adapted for the design of reinforced concrete tied columns by using the iterative method and interactive mode in the computer program. Thus a microcomputer and a disk can turn this tedious everyday design work into a simple and interesting task. Voluminous tables and charts are no longer necessarry.

V. REFERENCES

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8 - F.N. Pannel, "Failure Surfaces for Members in Compression and Biaxial bending," Jour. ACI, No.1, Jan. 1967.

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APPENDIX I PROGRAM LISTING

```
1000  REM  ****************************************************
1010  REM              -----  RC.COLUMN  -----
1020  REM  ****************************************************
1030  REM  ****************************************************
1040  REM  MASTER THESIS
1050  REM  REINFORCED CONCRETE COLUMN DESIGN PROGRAM
1060  REM  HAYATI ALTUN
1070  REM  DEPARTMENT OF CIVIL ENGINEERING
1080  REM  BOGAZICI UNIVERSITY
1090  REM  ISTANBUL-TURKEY
1100  REM  ****************************************************
1110  REM  ****************************************************
1120 R$ = "RE - ENTER"
1130 PHI = 0.7:DP = 0.05:RS = 0:RN = 0.01:RX = 0.04
1140 TL = 0.005:FPC = 30:FY = 400:Z1 = 0.85
1150 B = 0:H = 0:HN = 0:HM = 0:HI = 0.05:HB = 0.02
1160 K3 = 0.003:K29 = 200000:EPY = 0.002
1170 LU$ = "UNIAXIAL":LB$ = "BI-AXIAL":LD$ = "DESIGN"
1180 LA$ = "ANALYSIS"
1190 LX$ = "SECTION":LF$ = " FIND ":LI$ = " INPUT":LC$ = " COLUMN"
1200  REM
1210  REM
1220  REM
1230  REM   PROBLEM MENU
1240  HOME :BX% = 0
1250  INVERSE : PRINT "RC.COLUMN 1.0": NORMAL
1260  PRINT "PROBLEM MENU:"
1270  PRINT "============"
1280  GOSUB 4500: PRINT LA$
1290  PRINT "--------";
1300  HTAB 10: PRINT "1) ";LU$
1310  HTAB 10: PRINT "2) ";LB$
1320  IF PT% = 3 THEN 1410
1330  HTAB 10: PRINT "3) CHECK A DESIGN"
1340  GOSUB 4500: PRINT LD$
1350 CK% = 0
1360  PRINT "------";
1370  HTAB 10: PRINT "4) ";LU$;",";LI$;LX$
1380  HTAB 10: PRINT "5) ";LU$;",";LF$;LX$
1390  HTAB 10: PRINT "6) ";LB$;",";LI$;LX$
1400  HTAB 10: PRINT "7) ";LB$;",";LF$;LX$
1410  GOSUB 4500: PRINT "ENTER TYPE OF PROBLEM"
1420  INPUT "          OR '0' TO QUIT, ... ";PT%
1430  IF PT% < 0 OR PT% > 7 THEN  GOSUB 4500: PRINT R$: GOTO 1240
1440  IF PT% = 3 THEN CK% = 1: GOTO 1240
1450  IF PT% = 2 OR PT% = 7 OR PT% = 6 THEN BX% = 1
1460  IF PT% = 0 THEN  END
1470  REM
1480  REM
1490  REM
1500  REM  GET THE LOADS
1510  GOSUB 2890
1520  IF CK% = 1 THEN 4820
```

```
1530  GOSUB 3190: REM  GET THE MATERIALS
1540  REM  ANALYSIS OR DESIGN ?
1550  IF PT% <  = 3 THEN 2210
1560  REM
1570  REM
1580  REM
1590  REM  BEGIN THE DESIGN
1600  REM  INPUT OR FIND SECTION
1610  HOME : IF PT% = 5 OR PT% = 7 THEN 1840
1620  PRINT "SECTION WIDTH, CM = ";B * 100: GOSUB 4270
1630  IF AN$ = "" THEN 1650
1640 B =  VAL (AN$) / 100
1650  PRINT "SECTION DEPTH, CM = ";H * 100: GOSUB 4270
1660  IF AN$ = "" THEN 1680
1670 H =  VAL (AN$) / 100
1680  GOSUB 4280
1690 AG = B * H
1700  REM   OK @ MAX REINFORCEMENT ?
1710 RHO = RX: GOSUB 4460: GOSUB 2250
1720  IF NN =  > PN THEN 1770
1730  GOSUB 4500: PRINT "INSUFFICIENT CAPACITY AT MAX REINF."
1740  GOSUB 4500: GOSUB 4530
1750  ON QP% GOTO 1620,1680,1240
1760  REM  OK @ MIN REINFORCEMENT ?
1770 RHO = RN: GOSUB 4460: GOSUB 2250
1780  IF NN =  > PN THEN 2150
1790  REM   ITERATE TO FIND OPTIMUM REINFORCEMENT WITHIN RANGE
1800  GOSUB 4630: GOTO 2150
1810  REM   FIND LEAST AREA SECTION
1820  REM   ENTER RANGE OF DEPTH,WIDTH-DEPTH RATIO, AND
1830  REM   RANGE OF REINFORCEMENT
1840  HOME : VTAB 5
1850  PRINT "LEAST DEPTH, CM = ";HN * 100: GOSUB 4270
1860  IF AN$ = "" THEN 1880
1870 HN =  VAL (AN$) / 100
1880  PRINT "GREATEST DEPTH, CM = ";HM * 100: GOSUB 4270
1890  IF AN$ = "" THEN 1910
1900 HM =  VAL (AN$) / 100
1910  PRINT "INCREMENT OF DEPTH, CM = ";HI * 100: GOSUB 4270
1920  IF AN$ = "" THEN 1940
1930 HI =  VAL (AN$) / 100
1940  PRINT : PRINT "RATIO OF DEPTH TO WIDTH = ";HB * 100: GOSUB
1950  IF AN$ = "" THEN 1970
1960 HB =  VAL (AN$) / 100
1970  GOSUB 4280: REM  STEEL RATIOS & FRACTION
1980  REM  OK @ HMAX,RMAX ?
1990 H = HM: GOSUB 4470
2000  IF NN >  = PN THEN 2060
2010  HOME : VTAB 5: PRINT "NOT ADEQUATE WITH"
2020  PRINT "MAXIMUM DEPTH, REINFORCEMENT"
2030  GOSUB 4500: GOSUB 4530:
2040  ON QP% GOTO 1840,1970,1240
2050  REM  OK @ HMIN, RMAX ?
2060 H = HN: GOSUB 4470
2070  IF NN >  = PN THEN 2140: REM  MIN STEEL OK
```

```
2080  REM   FIND SMALLEST H=OK
2090  FOR H = HN + HI TO HM - HI STEP HI
2100  GOSUB 4470
2110  IF NN >  = PN THEN 2140
2120  NEXT H
2130 H = HM
2140  GOSUB 4630: REM  ITERATE FOR OPTIMUM RHO
2150  GOSUB 4520
2160  GOSUB 4030: GOSUB 4730
2170  REM
2180  REM
2190  REM
2200  REM  ANALYSIS:
2210  GOSUB 2560: REM  FIND SECTION
2220  REM
2230  REM
2240  GOSUB 2700: REM  REINFORCEMENT
2250  GOSUB 4440: REM   SET SECTION X
2260  GOSUB 3410: REM   CAPACITY X
2270 NN = P * PHI
2280  IF BX% = 0 THEN 2380
2290 PX = P
2300  GOSUB 4450: REM   SET SECTION Y
2310  GOSUB 4430: REM   ROTATE STEEL
2320  GOSUB 3410: REM   CAPACITY Y
2330  GOSUB 4430: REM   ROTATE STEEL BACK
2340 PY = P
2350  GOSUB 4420: REM  GET PO
2360  REM   BRESLER'S RECIPROCAL LOAD METHOD:
2370 NN = PHI / (1 / PX + 1 / PY - 1 / PO)
2380  IF PT% > 3 THEN  RETURN
2390  REM  OUTPUT RESULTS OF ANALYSIS
2400  GOSUB 4520
2410  GOSUB 4030
2420 LT$ = LU$: IF BX% <  > 0 THEN LT$ = LB$
2430  PRINT : PRINT LT$;" ";LC$;" ";LA$
2440  PRINT "-------------------------"
2450  PRINT : PRINT "APPLIED LOAD -- "; INT (PN);" KN ";
2460  HTAB 28: PRINT "PHI = ";PHI
2470  PRINT : PRINT "CAPACITY ------ "; INT (NN);" KN ";
2480  IF NN < PN THEN 2510
2490  HTAB 32: PRINT "ADEQUATE": HTAB 32: PRINT "========"
2500  GOTO 4730
2510  HTAB 28: PRINT "NOT ADEQUATE": HTAB 28: PRINT "=== --------
2520  GOTO 4730
2530  REM
2540  REM
2550  REM  INPUT THE SECTION
2560  HOME : GOSUB 4500: PRINT "ENTER THE SECTION GEOMETRY"
```

```
2570  PRINT "=========================="
2580  PRINT "SECTION WIDTH, CM = ";B * 100: GOSUB 4270
2590  IF AN$ = "" THEN 2610
2600 B =  VAL (AN$) / 100
2610  PRINT "SECTION DEPTH, CM = ";H * 100: GOSUB 4270
2620  IF AN$ = "" THEN 2640
2630 H =  VAL (AN$) / 100
2640 AG = B * H
2650  GOSUB 4380
2660  RETURN
2670  REM
2680  REM
2690  REM   INPUT STEEL
2700  HOME : GOSUB 4500: PRINT "INPUT THE REINFORCEMENT PATTERN"
2710  PRINT "================================"
2720  GOSUB 4500: INPUT "BARSIZE,... ";BS
2730  GOSUB 4500: INPUT "NUMBER OF BARS,... ";BN
2740  GOSUB 4500: INPUT "NUMBER OF SIDE BARS, ...";BF
2750  IF BF > BN THEN  GOSUB 4500: PRINT "IMPOSSIBLE !, ";R$: GOTO 2740
2760 RS = BF / BN
2770  IF BS =  < 20 THEN BA = (BS * 20 - 100) / 1000000
2780  IF BS = 25 OR BS = 35 THEN BA = (BS ^ 2 - 125 - (BS - 25) ^ 2) / 100
2790  IF BS = 30 THEN BA = 0.0007
2800  IF BS = 45 THEN BA = 0.0015
2810  IF BS = 55 THEN BA = 0.0025
2820 AS = BA * BN:RHO = AS / (B * H)
2830 AF = AS * RS:AE = (1 - RS) * AS / 2
2840  RETURN
2850  REM
2860  REM
2870  REM
2880  REM   GET LOADS
2890  HOME : VTAB 5
2900  PRINT "CAPACITY REDUCTION FACTOR = ";PHI
2910  GOSUB 4270
2920  IF AN$ = "" THEN 2970
2930 V =  VAL (AN$)
2940  IF V < 0 OR V > 1 THEN 2900
2950 PHI = V
2960  PRINT "NEW PHI = ";PHI
2970  GOSUB 4500: PRINT "INPUT THE LOADS"
2980  PRINT "==============="
2990  GOSUB 4500: INPUT "AXIAL LOAD, PN, ... ";PN
3000  GOSUB 4500: PRINT "DO YOU WANT TO ENTER MOMENT OR"
3010  INPUT "ECCENTRICITY (M/E), ... ";AN$
3020  IF AN$ = "M" THEN 3050
3030  IF AN$ = "E" THEN 3100
3040  GOSUB 4500: PRINT R$: GOTO 3000
3050  GOSUB 4500: INPUT "MOMENT, MX, ... ";MX
3060 EX = (MX / PN) * 100
3070  IF BX% > 0 THEN  GOSUB 4500: INPUT "MOMENT, MY, ... ";MY
```

```
3080 EY = (MY / PN) * 100
3090  RETURN
3100  GOSUB 4500: INPUT "ECCENTRICITY,EX,...CM = ";EX
3110 MX = PN * EX / 100
3120  IF BX% > 0 THEN  GOSUB 4500: INPUT "ECCENTRICITY,EY,...CM =
3130 MY = PN * EY / 100
3140  RETURN
3150  REM
3160  REM
3170  REM
3180  REM   ENTER MATERIAL PROPERTIES
3190  HOME : GOSUB 4500: PRINT "INPUT MATERIAL PROPERTIES"
3200  PRINT "========================="
3210  GOSUB 4500
3220  PRINT "CONCRETE STRENGTH, MPA = ";FPC
3230  GOSUB 4270
3240  IF AN$ = "" THEN 3290
3250 FPC =  VAL (AN$)
3260 Z1 = 0.85 - 0.008 * (FPC - 30)
3270  IF Z1 < 0.65 THEN Z1 = 0.65
3280  IF Z1 > 0.85 THEN Z1 = 0.85
3290  VTAB 14
3300  PRINT "STEEL YIELD STRESS, MPA = ";FY
3310  GOSUB 4270
3320  IF AN$ = "" THEN  RETURN
3330 FY =  VAL (AN$)
3340 EPY = FY / K29
3350  RETURN
3360  REM
3370  REM
3380  REM
3390  REM
3400  REM  FIND CAPACITY FROM P-M DIAGRAM
3410  GOSUB 4020
3420 D = SH - DP:EM = SH / 10
3430  GOSUB 4420
3440  IF ET <  = EM THEN P = 0.8 * PO: GOTO 3670
3450  REM  GET BALANCED P,M
3460 C = 600 / (600 + FY) * D
3470  GOSUB 3720
3480 PB = P:MB = M
3490  IF ET < E THEN CH = C:CL = 1.5 * D: GOTO 3620
3500  REM  FIND FAILURE REGION
3510  IF E = ET THEN 3670
3520  REM  E<ET (TENSION FAILURE)
3530 CL = C
3540  REM   CH FROM MO CONDITION
3550 E1 = 0.05:E2 = FY / K29
3560 E3 = (E1 + E2) / 2:C = D * K3 / (K3 + E3)
3570  GOSUB 3720
3580  IF P > (PO / 1000) THEN E2 = E3: GOTO 3560: REM  C > T
```

```
3590  IF  ABS (P) <  = (PO / 1000) THEN 3610
3600 E1 = E3: GOTO 3560: REM  TOO MUCH TENSION
3610 CH = C
3620 C = (CL + CH) / 2
3630  GOSUB 3720
3640  IF E < ET * 0.995 THEN CL = C: GOTO 3620
3650  IF E > ET * 1.005 THEN CH = C: GOTO 3620
3660  REM  WITHIN TOLERANCE
3670  RETURN
3680  REM  SUBROUTINE TO FIND LOAD AND MOMENT
3690  REM  FOR A GIVEN SECTION AND
3700  REM  DEPTH TO NEUTRAL AXIS
3710  REM  IDENTIFY CRITICAL DISTANCES
3720 H2 = SH / 2:CD = C - DP:TD = D - C
3730 YD = EPY * C / K3:JD = D - DP
3740 CP = CD - YD: IF CP <  = 0 THEN CP = 0
3750 TP = TD - YD: IF TP <  = 0 THEN TP = 0
3760  REM   CRITICAL STRAINS
3770 CE = CD / C * K3:TE = TD / C * K3
3780 CF = EPY: IF CP = 0 THEN CF = CD / C * K3
3790 TF = EPY: IF TP = 0 THEN TF = TD / C * K3
3800  REM  OUTER FACE STEEL STRESSES
3810 CS = K29 * CE: IF CS > FY THEN CS = FY
3820 TS = K29 * TE: IF TS > FY THEN TS = FY
3830  REM  FORCE RESULTANTS
3840 C1 = (CS - 0.85 * FPC) * AE
3850 C2 = (FY - 0.85 * FPC) * AF * CP / JD
3860 C3 = CS / 2 * AF * (CD - CP) / JD
3870 T1 = TS * AE
3880 T2 = FY * AF * TP / JD
3890 T3 = TS / 2 * AF * (TD - TP) / JD
3900 CC = 0.85 * Z1 * FPC * SB * C
3910 P = (CC + C1 + C2 + C3 - T1 - T2 - T3) * 1000
3920 M = CC * (H2 - Z1 * C / 2)
3930 M = M + C1 * (H2 - DP)
3940 M = M + C2 * (H2 - DP - CP / 2)
3950 M = M + C3 * (H2 - DP - CP - (CD - CP) / 3)
3960 M = M + T1 * (H2 - DP)
3970 M = M + T2 * (H2 - DP - TP / 2)
3980 M = (M + T3 * (H2 - DP - TP - (TD - TP) / 3)) * 1000
3990 E = M / P
4000  RETURN
4010  END
4020  HOME : VTAB 12: HTAB 17: PRINT "SOLVING": RETURN
4030  GOSUB 4500: PRINT "F'C  =  ";FPC;" MPA";
4040  HTAB 20: PRINT "B  = ";B * 100;" CM. "
4050  PRINT "FY   = ";FY;" MPA";: HTAB 20: PRINT "H  = ";H * 100;"
4060  HTAB 20: PRINT "D' =  ";DP * 100;".CM. ": GOSUB 4500
4070  IF PT% > 3 OR CK% = 1 THEN 4100
4080  PRINT BN;"- #";BS;"'S, ";BF;" SIDE BARS "
4090  RETURN
```

```
4100  IF PT% = 4 OR PT% = 5 THEN  PRINT "UNIAXIAL COLUMN DESIGN";
4110  IF PT% = 6 OR PT% = 7 THEN  PRINT "BI-AXIAL COLUMN DESIGN";
4120  IF PT% = 5 OR PT% = 7 THEN  PRINT ",FOUND SECTION": GOTO 4140
4130  PRINT : IF CK% = 1 THEN 4220
4140  GOSUB 4510: PRINT "LOAD";: HTAB 15: PRINT "MOMENT";
4150  HTAB 25: PRINT "ECCEN.": GOSUB 4510
4160  PRINT  INT (PN);: HTAB 17: PRINT  INT (MX);: HTAB 27
4170  PRINT  INT (EX * 100) / 100;: HTAB 35: PRINT "X"
4180  IF BX% = 0 THEN 4210
4190  HTAB 17: PRINT  INT (MY);: HTAB 27
4200  PRINT  INT (EY * 100) / 100;: HTAB 35: PRINT "Y"
4210  GOSUB 4510: PRINT
4220  PRINT "RHO          ="; INT (RHO * 1000) / 1000
4230  PRINT "STEEL AREA   ="; INT (AS * 1000000) / 100;" SQ CM., (TOTAL)
4240  HTAB 14: PRINT  INT (AE * 1000000) / 100;" SQ CM., (EACH FACE)"
4250  HTAB 14: PRINT  INT (AF * 500000) / 100;" SQ CM., (EACH SIDE)"
4260  RETURN
4270  PRINT : INPUT "<RETURN> OR NEW VALUE, ... ";AN$: RETURN
4280  PRINT : PRINT "MINIMUM STEEL RATIO = ";RN: GOSUB 4270
4290  IF AN$ = "" THEN 4310
4300 RN =  VAL (AN$)
4310  PRINT : PRINT "MAXIMUM STEEL RATIO = ";RX: GOSUB 4270
4320  IF AN$ = "" THEN 4340
4330 RX =  VAL (AN$)
4340  PRINT : PRINT "FRACTION OF STEEL AT SIDES"
4350  PRINT "OF SECTION = ";RS: GOSUB 4270
4360  IF AN$ = "" THEN 4380
4370 RS =  VAL (AN$)
4380  PRINT : PRINT "COVER TO BAR CENTERLINE, CM = ";DP * 100: GOSUB 42
4390  IF AN$ = "" THEN 4410
4400 DP =  VAL (AN$) / 100
4410  RETURN
4420 PO = 1000 * 0.85 * FPC * AG + 1000 * AS * (FY - 0.85 * FPC): RETUR
4430 XT = AE:AE = AF * 0.5:AF = 2 * XT: RETURN
4440 SB = B:SH = H:ET = EX / 100: RETURN
4450 SB = H:SH = B:ET = EY / 100: RETURN
4460 AS = AG * RHO:AF = RS * AS:AE = AS * (1 - RS) / 2: RETURN
4470 B =  INT (H / HB) / 100
4480  IF  INT (B / HI) * HI <  > B THEN B = (1 +  INT (B / HI)) * HI
4490 RHO = RX:AG = B * H: GOSUB 4460: GOSUB 2250: RETURN
4500  PRINT : PRINT : RETURN
4510  FOR I = 1 TO 38: PRINT "-";: NEXT : PRINT "-": RETURN
4520  HOME : GOSUB 4500: PRINT "RESULTS": PRINT "=======": RETURN
4530  PRINT "OPTIONS:": PRINT "======="
4540  PRINT "     A) CHANGE SECTION"
4550  PRINT "     B) CHANGE REINFORCEMENT LIMIT"
4560  PRINT "     C) RETURN TO MAIN MENU"
4570  GOSUB 4500
4580  INPUT "YOUR CHOICE ? ... ";AN$
4590  IF AN$ = "A" THEN QP% = 1: RETURN
```

```
4600  IF AN$ = "B" THEN QP% = 2: RETURN
4610  IF AN$ = "C" THEN QP% = 3: RETURN
4620  GOTO 4570
4630 RL = RN:RU = RX
4640 RHO = (RU + RL) / 2
4650  GOSUB 4460
4660  GOSUB 2250
4670  IF NN < PN THEN RL = RHO: GOTO 4640
4680  IF NN >  = PN AND  ABS ((RU - RL) / RL) < 0.05 THEN  RETURN
4690 RU = RHO: GOTO 4640
4700  REM
4710  REM
4720  REM
4730  GOTO 8560
4740  REM
4750  REM
4760  REM
4770  REM
4780  REM
4790  REM  SUBROUTINE
4800  REM  DESIGN FOR SLENDERNESS
4810  REM
4820  PRINT "DO YOU WANT TO CHECK SLENDERNESS ?"
4830  INPUT "Y/N ....";AN$
4840  IF AN$ = "Y" THEN 4870
4850  IF AN$ = "N" THEN 2250
4860  REM  BRACED OR UNBRACED FRAME ?
4870 EC = 4700 *  SQR (FPC)
4880  PRINT "INPUT OR FIND TYPE OF FRAME"
4890  PRINT "BRACED OR UNBRACED ?"
4900  INPUT "I/F ...";AN$
4910  IF AN$ = "F" THEN 4930
4920  IF AN$ = "I" THEN 5260
4930  INPUT "HIGHT OF BUILDING,.M..BH=";BH
4940  INPUT "TOTAL VERTICAL LOAD,.KN..VL = ";VL
4950  INPUT "NUMBER OF STORIES ...NS = ";NS
4960  INPUT "TYPES OF VERTICAL ELEMENTS...TE = ";TE
4970 IX = 0:IY = 0:S = 1
4980  INPUT "# OF SIMILAR VERTICAL ELEMENTS..NE = ";NE
4990  INPUT "SECTION DEPTH ...";D
5000  INPUT "SECTION WIDTH ...";W
5010 IX = IX + NE / 12 * W * D ^ 3
5020 IY = IY + NE / 12 * D * W ^ 3
5030 S = S + 1
5040  IF S <  = TE THEN 4980
5050 CX = BH *  SQR (VL / (EC * 1000 * IX))
5060 CY = BH *  SQR (VL / (EC * 1000 * IY))
5070 BV = 0.2 + 0.1 * NS
5075  PRINT "CX=";CX: PRINT "CY=";CY: PRINT "BV=";BV
5080  IF BV > 0.6 THEN BV = 0.6
5090  IF BV >  = CX THEN 5130
5100  PRINT "UNBRACED IN X-DIRECTION"
5110 SX% = 1
5120  GOTO 5150
```

```
5130  PRINT "BRACED IN X-DIRECTION"
5140 SX% = 0
5150  IF BX% = 1 THEN 5180
5160 SY% = 0
5170  GOTO 5420
5180  IF BV >  = CY THEN 5220
5190  PRINT "UNBRACED IN Y-DIRECTION"
5200 SY% = 1
5210  GOTO 5420
5220  PRINT "BRACED IN Y-DIRECTION"
5230 SY% = 0
5240  GOTO 5420
5250  REM  INPUT BRACED OR NOT
5260  PRINT "BRACED IN X-DIRECTION"
5270  INPUT "Y/N...";AN$
5280  IF AN$ = "Y" THEN SX% = 0
5290  IF AN$ = "N" THEN SX% = 1
5300  IF BX% = 1 THEN 5330
5310 SY% = 0
5320  GOTO 5420
5330  PRINT "BRACED IN Y-DIRECTION"
5340  INPUT "Y/N....";AN$
5350  IF AN$ = "Y" THEN SY% = 0
5360  IF AN$ = "N" THEN SY% = 1
5370  GOTO 5420
5380  REM
5390  REM
5400  REM
5410  REM  INPUT OR FIND EFFECTIVE LENGTH FACTORS
5420  IF SX% = 0 AND SY% = 0 THEN 5440
5430  INPUT "TYPES OF COLUMNS IN THE STOREY...";TC
5440 S = 1
5450 TX = 0:TY = 0
5460  PRINT "INPUT OR FIND K VALUES ?"
5470  INPUT "I/F....";AN$
5480  IF AN$ = "I" THEN FI% = 0
5490  IF AN$ = "F" THEN FI% = 1
5500  IF SX% = 0 AND S >  = 2 THEN 5700
5510  INPUT "COLUMN-X..B,H,L..  ";W,H4,CX
5520  IF FI% = 0 THEN 6650
5530  PRINT "UPPER COLUMN-X..B,H,L..  ";W;" , ";H4;" , ";CX
5540  GOSUB 8520
5550  IF BN$ = "" THEN 5580
5560 W1 =  VAL (BN$):H5 =  VAL (DN$):UX =  VAL (LN$)
5570  GOTO 5590
5580 W1 = W:H5 = H4:UX = CX
5590  PRINT "LOWER COLUMN-X..B,H,L..  ";W;" , ";H4;" , ";CX
5600  GOSUB 8520
5610  IF BN$ = "" THEN 5640
5620 W2 =  VAL (BN$):H2 =  VAL (DN$):LX =  VAL (LN$)
```

```
5630  GOTO 5650
5640 W2 = W:H6 = H4:LX = CX
5650  IF SX% = 0 THEN 5670
5660  INPUT "# OF SIMILAR COLUMNS IN X-DIR.";NX
5670 O1 = W1 * H5 ^ 3 / UX + W * H4 ^ 3 / CX
5680 O2 = W2 * H6 ^ 3 / LX + W * H4 ^ 3 / CX
5690  IF BX% = 0 THEN 6280
5700  IF SY% = 0 AND S >  = 2 THEN 6280
5710  IF S <  = 1 THEN 5750
5720  IF SX% = 0 AND SY% = 1 THEN 6070
5730  REM
5740  REM
5750  PRINT "INPUT NEW COLUMN LENGTHS FOR Y-DIRECTION"
5760  INPUT "Y/N...";AN$
5770  IF AN$ = "N" THEN 5790
5780  IF AN$ = "Y" THEN 5830
5790 CY = CX
5800 UY = UX
5810 LY = LX
5820  GOTO 6020
5830  REM  INPUT NEW COLUMN LENGTHS
5840  PRINT "COLUMN-Y...L = ";CX
5850  GOSUB 4270
5860  IF AN$ = "" THEN 5890
5870 CY =  VAL (AN$)
5880  GOTO 5900
5890 CY = CX
5900  PRINT "UPPER COLUMN-Y...L = ";CY
5910  GOSUB 4270
5920  IF AN$ = "" THEN 5950
5930 UY =  VAL (AN$)
5940  GOTO 5960
5950 UY = CY
5960  PRINT "LOWER COLUMN-Y...L = ";CY
5970  GOSUB 4270
5980  IF AN$ = "" THEN 6010
5990 LY =  VAL (AN$)
6000  GOTO 6020
6010 LY = CY
6020  IF SY% = 0 THEN 6040
6030  INPUT "# OF SIMILAR COLUMNS IN Y-DIR...";NY
6040 O3 = H5 * W1 ^ 3 / UY + H4 * W ^ 3 / CY
6050 O4 = H6 * W2 ^ 3 / LY + H4 * W ^ 3 / CY
6060  GOTO 6270
6070  INPUT "COLUMN-Y..B,H,L..";H4,W,CY
6080  PRINT "UPPER COLUMN-Y..B,H,L..  ";H4;" , ";W;" , ";CY
6090  GOSUB 8520
6100  IF BN$ = "" THEN 6130
6110 H5 =  VAL (BN$):W1 =  VAL (DN$):UY =  VAL (LN$)
6120  GOTO 6140
6130 H5 = H4:W1 = W:UY = CY
6140  PRINT "LOWER COLUMN-Y..B,H,L..  ";H4;" , ";W;" , ";CY
```

```
6150  GOSUB 8520
6160  IF BN$ = "" THEN 6190
6170 H6 =  VAL (BN$):W2 =  VAL (DN$):LY =  VAL (LN$)
6180  GOTO 6200
6190 H6 = H4:W2 = W:LY = CY
6200  IF SY% = 0 THEN 6220
6210  INPUT "# OF SIMILAR COLUMNS...";NY
6220 O3 = H4 * W ^ 3 / CY + H5 * W1 ^ 3 / UY
6230 O4 = H4 * W ^ 3 / CY + H6 * W2 ^ 3 / LY
6240  REM
6250  REM  INPUT BEAM DIMENSIONS IN X-DIRECTION
6260  REM
6270  IF SX% = 0 AND S >  = 2 THEN 6790
6280  INPUT "RIGHT UPPER BEAM-X..B,D,L..  ";B1,D1,L1
6290  PRINT "LEFT UPPER BEAM-X..B,D,L..  ";B1;" , ";D1;" , ";L1
6300  GOSUB 8520
6310  IF BN$ = "" THEN 6340
6320 B2 =  VAL (BN$):D2 =  VAL (DN$):L2 =  VAL (LN$)
6330  GOTO 6350
6340 B2 = B1:D2 = D1:L2 = L1
6350  PRINT "RIGHT LOWER BEAM-X..B,D,L..  ";B1;" , ";D1;" , ";L1
6360  GOSUB 8520
6370  IF BN$ = "" THEN 6400
6380 B3 =  VAL (BN$):D3 =  VAL (DN$):L3 =  VAL (LN$)
6390  GOTO 6410
6400 B3 = B1:D3 = D1:L3 = L1
6410  PRINT "LEFT LOWER BEAM-X..B,D,L..   ";B2;" , ";D2;" , ";L2
6420  GOSUB 8520
6430  IF BN$ = "" THEN 6460
6440 B4 =  VAL (BN$):D4 =  VAL (DN$):L4 =  VAL (LN$)
6450  GOTO 6470
6460 B4 = B2:D4 = D2:L4 = L2
6470 X1 = B1 * D1 ^ 3 / L1 + B2 * D2 ^ 3 / L2
6480 X2 = B3 * D3 ^ 3 / L3 + B4 * D4 ^ 3 / L4
6490 FAX = O1 / X1
6500 FBX = O2 / X2
6510  IF SX% = 1 AND S >  = 2 THEN 6600
6520 K1 = 0.7 + 0.05 * (FAX + FBX)
6530 FMX = FAX
6540  IF FAX > FBX THEN FMX = FBX
6550 K4 = 0.85 + 0.05 * FMX
6560 KB = K1
6570  IF K1 > K4 THEN KB = K4
6580  IF KB > 1 THEN KB = 1
6590  IF SX% = 0 THEN 6640
6600 FX = (FAX + FBX) / 2
6610  IF FX < 2 THEN KU = ((20 - FX) / 20) *  SQR (1 + FX)
6620  IF FX >  = 2 THEN KU = 0.9 *  SQR (1 + FX)
6630  IF KU < 1 THEN KU = 1
6640  IF FI% = 1 THEN 6680
6650  IF S >  = 2 THEN 6665
6660  INPUT "BRACED EFFECTIVE LENGTH FACTOR-X, KBX=";KB
6662  IF SX% = 0 THEN 6680
6665  INPUT "# OF SIMILAR COLUMNS-X...";NX
```

```
6670  INPUT "UNBRACED EFFECTIVE LENGTH FACTOR-X, KUX=";KU
6680  IF S >  = 2 THEN 6780
6690 R1 = 0.3 * H
6700  IF SX% = 1 THEN 6760
6710  INPUT "LARGER END MOMENT-X..MX2=";M2
6720  INPUT "SMALLER END MOMENT-X..MX1=";M1
6730 F1 = KB * CX / R1
6740 F2 = 34 - 12 * M1 / M2
6750  IF SX% = 0 THEN 6780
6760 F1 = KU * CX / R1
6770 F2 = 22
6780  IF BX% = 0 THEN 7380
6790  IF SY% = 0 AND S >  = 2 THEN 7380
6800  IF FI% = 0 THEN 7210
6810  REM
6820  REM  INPUT BEAM DIMENSIONS IN Y-DIRECTION
6830  REM
6840  INPUT "RIGHT UPPER BEAM-Y..B,D,L..  ";B1,D1,L1
6850  PRINT "LEFT UPPER BEAM-Y..B,D,L..  ";B1;" , ";D1;" , ";L1
6860  GOSUB 8520
6870  IF BN$ = "" THEN 6900
6880 B2 =  VAL (BN$):D2 =  VAL (DN$):L2 =  VAL (LN$)
6890  GOTO 6910
6900 B2 = B1:D2 = D1:L2 = L1
6910  PRINT "RIGHT LOWER BEAM-Y..B,D,L..  ";B1;" , ";D1;" , ";L1
6920  GOSUB 8520
6930  IF BN$ = "" THEN 6960
6940 B3 =  VAL (BN$):D3 =  VAL (DN$):L3 =  VAL (LN$)
6950  GOTO 6970
6960 B3 = B1:D3 = D1:L3 = L1
6970  PRINT "LEFT LOWER BEAM-Y..B,D,L..  ";B2;" , ";D2;" , ";L2
6980  GOSUB 8520
6990  IF BN$ = "" THEN 7020
7000 B4 =  VAL (BN$):D4 =  VAL (DN$):L4 =  VAL (LN$)
7010  GOTO 7030
7020 B4 = B2:D4 = D2:L4 = L2
7030 Y1 = B1 * D1 ^ 3 / L1 + B2 * D2 ^ 3 / L2
7040 Y2 = B3 * D3 ^ 3 / L3 + B4 * D4 ^ 3 / L4
7050 FAY = O3 / Y1
7060 FBY = O4 / Y2
7070  IF SY% = 1 AND S >  = 2 THEN 7160
7080 K1 = 0.7 + 0.05 * (FAY + FBY)
7090 FMY = FAY
7100  IF FAY > FBY THEN FMY = FBY
7110 K4 = 0.85 + 0.05 * FMY
7120 KB = K1
7130  IF K1 > K4 THEN BK = K4
7140  IF BK > 1 THEN BK = 1
7150  IF SY% = 0 THEN 7200
7160 GY = (FAY + FBY) / 2
7170  IF GY < 2 THEN UK = (20 - GY) / 20 *  SQR (1 + GY)
7180  IF GY >  = 2 THEN UK = 0.9 *  SQR (1 + GY)
```

```
7190  IF UK < 1 THEN UK = 1
7200  IF FI% = 1 THEN 7240
7210  IF S >  = 2 THEN 7225
7215 CY = CX
7220  INPUT "BRACED EFFECTIVE LENGTH FACTOR-Y, KBY=";BK
7222  IF SY% = 0 THEN 7240
7225  INPUT "# OF SIMILAR COLUMNS-Y...";NY
7230  INPUT "UNBRACED EFFECTIVE LENGTH FACTOR-Y, KUY=";UK
7235 CY = CX
7240  IF S >  = 2 THEN 7700
7250 R1 = 0.3 * B
7260  IF SY% = 1 THEN 7320
7270  INPUT "LARGER END MOMENT-Y, MY2=";N2
7280  INPUT "SMALLER END MOMENT-Y, MY1=";N1
7290 G1 = BK * CY / R1
7300 G2 = 34 - 12 * N1 / N2
7310  IF SY% = 0 THEN 7380
7320 G1 = UK * CY / R1
7330 G2 = 22
7340  REM
7350  REM
7360  REM  COMPARISON OF SLENDERNESS CRITERIAS
7370  REM  X-DIRECTION
7380  IF S >  = 2 THEN 7700
7390  IF F1 <  = 100 THEN 7440
7400  PRINT "THIS PROGRAM IS INSUFFICIENT IN X-DIRECTION"
7410 IX% = 1
7420  IF BX% = 1 THEN 7530
7430  GOTO 7610
7440  IF F1 > F2 THEN 7490
7450  PRINT "SLENDERNESS IS NEGLIGIBLE IN X-DIRECTION"
7460 IX% = 2: PRINT "F1=";F1: PRINT "F2=";F2
7470  IF BX% = 1 THEN 7530
7480  GOTO 7610
7490 IX% = 3
7500  IF BX% = 1 THEN 7530
7510  GOTO 7610
7520  REM  Y-DIRECTION
7530  IF G1 <  = 100 THEN 7570
7540  PRINT "THIS PROGRAM IS INSUFFICIENT IN Y-DIRECTION"
7550 IY% = 1
7560  GOTO 7610
7570  IF G1 > G2 THEN 7600
7580 IY% = 2
7590  GOTO 7610
7600 IY% = 3
7610  IF BX% = 0 THEN IY% = IX%
7620  IF IX% = 1 OR IY% = 1 THEN 2030
7630  IF IX% = 2 AND IY% = 2 THEN 2250
7640  IF IX% = 2 THEN SX% = 0
7650  IF IY% = 2 THEN SY% = 0
7660  REM
7670  REM
7680  REM
7690  IF IX% = 2 THEN 7920
```

```
7700  IF SX% = 0 AND S >  = 2 THEN 7920
7710  IF SX% = 0 THEN 7740
7715  IF S >  = 2 THEN 7760
7720  INPUT "LARGER END MOMENT-X, MX2=";M2
7730  INPUT "SMALLER END MOMENT-X, MX1=";M1
7740  INPUT "UNBRACED END MOMENT-X, MX3=";M3
7750  INPUT "UNBRACED AXIAL LOAD-X, UPX=";XU
7760  INPUT "DEAD AND LIVE LOAD MMOMENTS-X, DM,LM=";XD,XL
7770  IF S >  = 2 THEN 7830
7780  IF SX% = 0 THEN 7800
7790  INPUT "TOTAL STORY LOAD, TP=";PT
7800 XR = M1 / M2
7810  IF M1 = 0 AND M2 = 0 THEN XR = 1
7820 JX = 0.6 + 0.4 * XR
7830 BX = XD / (XD + XL)
7840 XE = EC * W * 1000 * (H4 * 1000) ^ 3 / (30 * (1 + BX))
7850  IF S >  = 2 THEN 7910
7860 PX = 3.14 ^ 2 * XE / (1000 * (KB * CX * 1000) ^ 2)
7870 DX = JX / (1 - PN / (PHI * PX))
7875  PRINT "DX=";DX
7880  IF DX < 1 THEN DX = 1
7890  IF SX% = 0 THEN SX = 1
7900  IF SX% = 0 THEN 7920
7910 TX = NX * 3.14 ^ 2 * XE / (1000 * (KU * CX * 1000) ^ 2) + TX
7920  IF BX% = 0 THEN 8210
7930  IF IY% = 2 THEN 8210
7940  IF SY% = 0 AND S >  = 2 THEN 8210
7950  IF SY% = 0 THEN 8010
7960  REM
7970  REM
7980  IF S >  = 2 THEN 8030
7990  INPUT "LARGER END MOMENT-Y, MY2=";N2
8000  INPUT "SMALLER END MOMENT-Y, MY1=";N1
8010  INPUT "UNBRACED END MOMENT-Y, MY3=";N3
8020  INPUT "UNBRACED AXIAL LOAD-Y, UPY=";YU
8030  INPUT "DEAD AND LIVE LOAD MOMENTS-Y, DM,LM=";YY,YL
8040  IF S >  = 2 THEN 8120
8050  REM
8060 YR = N1 / N2
8070  IF N1 = 0 AND N2 = 0 THEN YR = 1
8080 JY = 0.6 + 0.4 * YR
8090  IF SX% = 1 THEN 8120
8100  IF SY% = 0 THEN 8120
8110  INPUT "TOTAL STORY LOAD, TP=";PT
8120 BY = YY / (YY + YL)
8130 YE = EC * 1000 * H4 * (W * 1000) ^ 3 / (30 * (1 + BY))
8140  IF S >  = 2 THEN 8200
8150 PY = 3.14 ^ 2 * YE / (1000 * (BK * CY * 1000) ^ 2)
8160 DY = JY / (1 - PN / (PHI * PY))
8165  PRINT "DY=";DY
8170  IF DY < 1 THEN DY = 1
8180  IF SY% = 0 THEN SY = 1
8190  IF SY% = 0 THEN 8210
8200 TY = NY * 3.14 ^ 2 * YE / (1000 * (UK * CY * 1000) ^ 2) + TY
8210  IF SX% = 0 AND SY% = 0  THEN 8260
```

```
8220 S = S + 1
8230  IF S <  = TC THEN 5500
8240  REM
8250  REM
8260  IF IX% = 2 THEN 8380
8270  IF SX% = 0 THEN 8300
8280 SX = 1 / (1 - PT / (PHI * TX))
8285  PRINT "SX=";SX
8290  IF SX < 1 THEN SX = 1
8300 AX = M2 / PN
8310  IF AX < (0.015 + 0.03 * H) THEN M2 = PN * (0.015 + 0.03 * H)
8320 BX = M3 / XU
8330  IF BX < (0.015 + 0.03 * H) THEN M3 = XU * (0.015 + 0.03 * H)
8340 MX = DX * M2 + SX * M3
8350 EX = 100 * MX / PN
8360  PRINT "DBX = ";DX;"              DSX = ";SX
8370  PRINT "MX = ";MX;"                EX = ";EX
8380  IF BX% = 0 THEN 2250
8390  IF IY% = 2 THEN 2250
8400  IF SY% = 0 THEN 8430
8410 SY = 1 / (1 - PT / (PHI * TY))
8415  PRINT "SY=";SY
8420  IF SY < 1 THEN SY = 1
8430 AY = N2 / PN
8440  IF AY < (0.015 + 0.03 * B) THEN N2 = PN * (0.015 + 0.03 * B)
8450 BY = N3 / YU
8460  IF BY < (0.015 + 0.03 * B) THEN N3 = YU * (0.015 + 0.03 * B)
8470 MY = DY * N2 + SY * N3
8480 EY = 100 * MY / PN
8490  PRINT "DBY = ";DY;"              DSY = ";SY
8500  PRINT "MY = ";MY;"                EY = ";EY
8510  GOTO 2250
8520  PRINT : INPUT "<RETURN> OR NEW VALUES...   ";BN$
8530  IF BN$ = "" THEN  RETURN
8540  INPUT DN$
8550  INPUT LN$: RETURN
8560  INPUT "HIT <RETURN> TO CONTINUE...";AN$
8570  GOTO 1240
8580  END
```

```
RC.COLUMN 1.0
PROBLEM MENU:
=============


ANALYSIS
-------   1) UNIAXIAL
          2) BI-AXIAL
          3) CHECK A DESIGN


DESIGN
------    4) UNIAXIAL, INPUTSECTION
          5) UNIAXIAL, FIND SECTION
          6) BI-AXIAL, INPUTSECTION
          7) BI-AXIAL, FIND SECTION


ENTER TYPE OF PROBLEM
       OR '0' TO QUIT, ... 1
CAPACITY REDUCTION FACTOR = .7

<RETURN> OR NEW VALUE, ...


INPUT THE LOADS
===============


AXIAL LOAD, PN, ... 2220


DO YOU WANT TO ENTER MOMENT OR
ECCENTRICITY (M/E), ... E


ECCENTRICITY,EX,...CM = 20


INPUT MATERIAL PROPERTIES
=========================


CONCRETE STRENGTH, MPA = 30

<RETURN> OR NEW VALUE, ...
STEEL YIELD STRESS, MPA = 400

<RETURN> OR NEW VALUE, ...


ENTER THE SECTION GEOMETRY
==========================
SECTION WIDTH, CM = 0
```

```
<RETURN> OR NEW VALUE, ... 30
SECTION DEPTH, CM = 0

<RETURN> OR NEW VALUE, ... 60

COVER TO BAR CENTERLINE, CM = 5

<RETURN> OR NEW VALUE, ...


INPUT THE REINFORCEMENT PATTERN
===============================


BARSIZE,... 25


NUMBER OF BARS,... 8


NUMBER OF SIDE BARS, ...0
                 SOLVING


RESULTS
=======


F'C  =  30 MPA      B  = 30 CM.
FY   = 400 MPA      H  = 60 CM.
                    D' =  5 CM.


8- #25'S, 0 SIDE BARS

UNIAXIAL  COLUMN ANALYSIS
--------------------------

APPLIED LOAD -- 2220 KN    PHI = .7

CAPACITY ------ 2154 KN    NOT ADEQUATE
                           === --------
```

```
HIT <RETURN> TO CONTINUE...
RC.COLUMN 1.0
PROBLEM MENU:
=============


ANALYSIS
--------   1) UNIAXIAL
           2) BI-AXIAL
           3) CHECK A DESIGN


DESIGN
------     4) UNIAXIAL, INPUTSECTION
           5) UNIAXIAL, FIND SECTION
           6) BI-AXIAL, INPUTSECTION
           7) BI-AXIAL, FIND SECTION


ENTER TYPE OF PROBLEM
       OR '0' TO QUIT, ... 2
CAPACITY REDUCTION FACTOR = .7

<RETURN> OR NEW VALUE, ... 1
NEW PHI = 1


INPUT THE LOADS
===============


AXIAL LOAD, PN, ... 4160


DO YOU WANT TO ENTER MOMENT OR
ECCENTRICITY (M/E), ... M


MOMENT, MX, ... 1310


MOMENT, MY, ... 505


INPUT MATERIAL PROPERTIES
=========================


CONCRETE STRENGTH, MPA = 30

<RETURN> OR NEW VALUE, ...
STEEL YIELD STRESS, MPA = 400

<RETURN> OR NEW VALUE, ... 300
```

```
ENTER THE SECTION GEOMETRY
============================
SECTION WIDTH, CM = 30

<RETURN> OR NEW VALUE, ... 65
SECTION DEPTH, CM = 60

<RETURN> OR NEW VALUE, ... 65

COVER TO BAR CENTERLINE, CM = 5

<RETURN> OR NEW VALUE, ...


INPUT THE REINFORCEMENT PATTERN
================================


BARSIZE,... 45


NUMBER OF BARS,... 10


NUMBER OF SIDE BARS, ...5
                SOLVING
                SOLVING


RESULTS
=======


F'C  =  30 MPA     B  = 65 CM.
FY   = 300 MPA     H  = 65 CM.
                   D' =  5 CM.


10- #45'S, 5 SIDE BARS

BI-AXIAL  COLUMN ANALYSIS
---------------------------

APPLIED LOAD -- 4160 KN    PHI = 1

CAPACITY ------ 4566 KN        ADEQUATE
                               ========
```

```
HIT <RETURN> TO CONTINUE...
RC.COLUMN 1.0
PROBLEM MENU:
=============


ANALYSIS
-------   1) UNIAXIAL
          2) BI-AXIAL
          3) CHECK A DESIGN


DESIGN
------    4) UNIAXIAL, INPUTSECTION
          5) UNIAXIAL, FIND SECTION
          6) BI-AXIAL, INPUTSECTION
          7) BI-AXIAL, FIND SECTION


ENTER TYPE OF PROBLEM
       OR '0' TO QUIT, ... 4
CAPACITY REDUCTION FACTOR = 1

<RETURN> OR NEW VALUE, ... .7
NEW PHI = .7


INPUT THE LOADS
===============


AXIAL LOAD, PN, ... 2220


DO YOU WANT TO ENTER MOMENT OR
ECCENTRICITY (M/E), ... E


ECCENTRICITY,EX,...CM = 20


INPUT MATERIAL PROPERTIES
=========================


CONCRETE STRENGTH, MPA = 30

<RETURN> OR NEW VALUE, ...
STEEL YIELD STRESS, MPA = 300

<RETURN> OR NEW VALUE, ... 400
SECTION WIDTH, CM = 65
```

```
<RETURN> OR NEW VALUE, ... 30
SECTION DEPTH, CM = 65

<RETURN> OR NEW VALUE, ... 60

MINIMUM STEEL RATIO = .01

<RETURN> OR NEW VALUE, ...

MAXIMUM STEEL RATIO = .04

<RETURN> OR NEW VALUE, ...

FRACTION OF STEEL AT SIDES
OF SECTION = .5

<RETURN> OR NEW VALUE, ... 0

COVER TO BAR CENTERLINE, CM = 5

<RETURN> OR NEW VALUE, ...
                SOLVING
                SOLVING
                SOLVING
                SOLVING
                SOLVING
                SOLVING
                SOLVING
                SOLVING


RESULTS
=======


F'C  =  30 MPA      B  = 30 CM.
FY   = 400 MPA      H  = 60 CM.
                    D' =  5 CM.


UNIAXIAL COLUMN DESIGN
---------------------------------------
LOAD           MOMENT     ECCEN.
---------------------------------------
2220            444         20      X
---------------------------------------

RHO          =.024
STEEL AREA   =44.15 SQ CM., (TOTAL)
              22.07 SQ CM., (EACH FACE)
              0 SQ CM., (EACH SIDE)
```

```
HIT <RETURN> TO CONTINUE...
RC.COLUMN 1.0
PROBLEM MENU:
=============


ANALYSIS
--------    1) UNIAXIAL
            2) BI-AXIAL
            3) CHECK A DESIGN


DESIGN
------      4) UNIAXIAL, INPUTSECTION
            5) UNIAXIAL, FIND SECTION
            6) BI-AXIAL, INPUTSECTION
            7) BI-AXIAL, FIND SECTION


ENTER TYPE OF PROBLEM
       OR '0' TO QUIT, ... 5
CAPACITY REDUCTION FACTOR = .7

<RETURN> OR NEW VALUE, ...


INPUT THE LOADS
===============


AXIAL LOAD, PN, ... 2220


DO YOU WANT TO ENTER MOMENT OR
ECCENTRICITY (M/E), ... E


ECCENTRICITY,EX,...CM = 20


INPUT MATERIAL PROPERTIES
=========================


CONCRETE STRENGTH, MPA = 30

<RETURN> OR NEW VALUE, ...
STEEL YIELD STRESS, MPA = 400

<RETURN> OR NEW VALUE, ...
LEAST DEPTH, CM = 0
```

```
<RETURN> OR NEW VALUE, ... 20
GREATEST DEPTH, CM = 0

<RETURN> OR NEW VALUE, ... 65
INCREMENT OF DEPTH, CM = 5

<RETURN> OR NEW VALUE, ...

RATIO OF DEPTH TO WIDTH = 2

<RETURN> OR NEW VALUE, ...

MINIMUM STEEL RATIO = .01

<RETURN> OR NEW VALUE, ...

MAXIMUM STEEL RATIO = .04

<RETURN> OR NEW VALUE, ...

FRACTION OF STEEL AT SIDES
OF SECTION = 0

<RETURN> OR NEW VALUE, ...

COVER TO BAR CENTERLINE, CM = 5

<RETURN> OR NEW VALUE, ...
                SOLVING
                SOLVING
                SOLVING
                SOLVING
                SOLVING
                SOLVING
                SOLVING
                SOLVING
                SOLVING
                SOLVING
                SOLVING
                SOLVING
                SOLVING
                SOLVING
                SOLVING
                SOLVING
```

```
RESULTS
=======

F'C   =  30 MPA      B  = 30 CM.
FY    = 400 MPA      H  = 55 CM.
                     D' =  5 CM.

UNIAXIAL COLUMN DESIGN,FOUND SECTION
---------------------------------------
LOAD             MOMENT    ECCEN.
---------------------------------------
2220               444         20        X
---------------------------------------

RHO            =.036
STEEL AREA     =59.42 SQ CM., (TOTAL)
                29.71 SQ CM., (EACH FACE)
                0 SQ CM., (EACH SIDE)
HIT <RETURN> TO CONTINUE...
RC.COLUMN 1.0
PROBLEM MENU:
=============

ANALYSIS                      SAMPLE PROBLEM 5
-------   1) UNIAXIAL
          2) BI-AXIAL
          3) CHECK A DESIGN

DESIGN
------    4) UNIAXIAL, INPUTSECTION
          5) UNIAXIAL, FIND SECTION
          6) BI-AXIAL, INPUTSECTION
          7) BI-AXIAL, FIND SECTION

ENTER TYPE OF PROBLEM
       OR '0' TO QUIT, ... 6
CAPACITY REDUCTION FACTOR = .7

<RETURN> OR NEW VALUE, ... 1
NEW PHI = 1

INPUT THE LOADS
===============

AXIAL LOAD, PN, ... 4160
```

```
DO YOU WANT TO ENTER MOMENT OR
ECCENTRICITY (M/E), ... M


MOMENT, MX, ... 1310


MOMENT, MY, ... 505


INPUT MATERIAL PROPERTIES
=========================


CONCRETE STRENGTH, MPA = 30

<RETURN> OR NEW VALUE, ...
STEEL YIELD STRESS, MPA = 400

<RETURN> OR NEW VALUE, ... 300
SECTION WIDTH, CM = 30

<RETURN> OR NEW VALUE, ... 65
SECTION DEPTH, CM = 55

<RETURN> OR NEW VALUE, ... 65

MINIMUM STEEL RATIO = .01

<RETURN> OR NEW VALUE, ...

MAXIMUM STEEL RATIO = .04

<RETURN> OR NEW VALUE, ...

FRACTION OF STEEL AT SIDES
OF SECTION = 0

<RETURN> OR NEW VALUE, ... 0.5

COVER TO BAR CENTERLINE, CM = 5

<RETURN> OR NEW VALUE, ...
                SOLVING
                SOLVING
                SOLVING
                SOLVING
                SOLVING
                SOLVING
                SOLVING
```

```
                    SOLVING
                    SOLVING
                    SOLVING
                    SOLVING
                    SOLVING
                    SOLVING
                    SOLVING
                    SOLVING
                    SOLVING


RESULTS
=======


F'C  =  30 MPA      B  = 65 CM.
FY   = 300 MPA      H  = 65 CM.
                    D' =  5 CM.


BI-AXIAL COLUMN DESIGN
---------------------------------------
LOAD            MOMENT    ECCEN.
---------------------------------------
4160              1310      31.49   X
                  505       12.13   Y
---------------------------------------

RHO          =.028
STEEL AREA   =119.48 SQ CM., (TOTAL)
               29.87 SQ CM., (EACH FACE)
               29.87 SQ CM., (EACH SIDE)
```

```
HIT <RETURN> TO CONTINUE...        SAMPLE PROBLEM 6
RC.COLUMN 1.0
PROBLEM MENU:
=============


ANALYSIS
-------    1) UNIAXIAL
           2) BI-AXIAL
           3) CHECK A DESIGN


DESIGN
------     4) UNIAXIAL, INPUTSECTION
           5) UNIAXIAL, FIND SECTION
           6) BI-AXIAL, INPUTSECTION
           7) BI-AXIAL, FIND SECTION


ENTER TYPE OF PROBLEM
       OR '0' TO QUIT, ... 7
CAPACITY REDUCTION FACTOR = 1

<RETURN> OR NEW VALUE, ...


INPUT THE LOADS
===============


AXIAL LOAD, PN, ... 4160


DO YOU WANT TO ENTER MOMENT OR
ECCENTRICITY (M/E), ... M


MOMENT, MX, ... 1310


MOMENT, MY, ... 505


INPUT MATERIAL PROPERTIES
=========================


CONCRETE STRENGTH, MPA = 30

<RETURN> OR NEW VALUE, ...
STEEL YIELD STRESS, MPA = 300
```

```
<RETURN> OR NEW VALUE, ...
LEAST DEPTH, CM = 20

<RETURN> OR NEW VALUE, ... 60
GREATEST DEPTH, CM = 65

<RETURN> OR NEW VALUE, ... 70
INCREMENT OF DEPTH, CM = 5

<RETURN> OR NEW VALUE, ...

RATIO OF DEPTH TO WIDTH = 2

<RETURN> OR NEW VALUE, ... 1

MINIMUM STEEL RATIO = .01

<RETURN> OR NEW VALUE, ...

MAXIMUM STEEL RATIO = .04

<RETURN> OR NEW VALUE, ...

FRACTION OF STEEL AT SIDES
OF SECTION = .5

<RETURN> OR NEW VALUE, ...

COVER TO BAR CENTERLINE, CM = 5

<RETURN> OR NEW VALUE, ...
                  SOLVING
                  SOLVING
                  SOLVING
                  SOLVING
                  SOLVING
                  SOLVING
                  SOLVING
                  SOLVING
                  SOLVING
                  SOLVING
                  SOLVING
                  SOLVING
                  SOLVING
                  SOLVING
                  SOLVING
                  SOLVING
                  SOLVING
```

SAMPLE PROBLEM 7

```
                 SOLVING


RESULTS
=======


F'C   =  30 MPA       B   = 65 CM.
FY    = 300 MPA       H   = 65 CM.
                      D'  =  5 CM.


BI-AXIAL COLUMN DESIGN,FOUND SECTION
---------------------------------------
LOAD              MOMENT    ECCEN.
---------------------------------------
4160                1310      31.49   X
                   505       12.13   Y
---------------------------------------

RHO            =.028
STEEL AREA   =119.48 SQ CM., (TOTAL)
              29.87 SQ CM., (EACH FACE)
              29.87 SQ CM., (EACH SIDE)
HIT <RETURN> TO CONTINUE...
RC.COLUMN 1.0
PROBLEM MENU:
=============


ANALYSIS
-------    1) UNIAXIAL
           2) BI-AXIAL
           3) CHECK A DESIGN


DESIGN
------     4) UNIAXIAL, INPUTSECTION
           5) UNIAXIAL, FIND SECTION
           6) BI-AXIAL, INPUTSECTION
           7) BI-AXIAL, FIND SECTION


ENTER TYPE OF PROBLEM
       OR '0' TO QUIT, ... 3
RC.COLUMN 1.0
PROBLEM MENU:
=============


ANALYSIS
-------    1) UNIAXIAL
           2) BI-AXIAL


ENTER TYPE OF PROBLEM
       OR '0' TO QUIT, ... 2
```

```
CAPACITY REDUCTION FACTOR = 1

<RETURN> OR NEW VALUE, ...


INPUT THE LOADS
===============


AXIAL LOAD, PN, ... 4160


DO YOU WANT TO ENTER MOMENT OR
ECCENTRICITY (M/E), ... M


MOMENT, MX, ... 1310


MOMENT, MY, ... 505
DO YOU WANT TO CHECK SLENDERNESS ?
Y/N ....N
                SOLVING
                SOLVING


RESULTS
=======


F'C  =  30 MPA      B  = 65 CM.
FY   = 300 MPA      H  = 65 CM.
                    D' =  5 CM.


RHO            =.028
STEEL AREA     =119.48 SQ CM., (TOTAL)
                 29.87 SQ CM., (EACH FACE)
                 29.87 SQ CM., (EACH SIDE)

BI-AXIAL  COLUMN ANALYSIS
-------------------------

APPLIED LOAD -- 4160 KN    PHI = 1

CAPACITY ------ 4163 KN       ADEQUATE
                              ========
```

```
HIT <RETURN> TO CONTINUE...     SAMPLE PROBLEM  8
RC.COLUMN 1.0
PROBLEM MENU:
=============


ANALYSIS
-------    1) UNIAXIAL
           2) BI-AXIAL
           3) CHECK A DESIGN


DESIGN
------     4) UNIAXIAL, INPUTSECTION
           5) UNIAXIAL, FIND SECTION
           6) BI-AXIAL, INPUTSECTION
           7) BI-AXIAL, FIND SECTION


ENTER TYPE OF PROBLEM
       OR '0' TO QUIT, ... 3
RC.COLUMN 1.0
PROBLEM MENU:
=============


ANALYSIS
-------    1) UNIAXIAL
           2) BI-AXIAL


ENTER TYPE OF PROBLEM
       OR '0' TO QUIT, ... 2
CAPACITY REDUCTION FACTOR = 1

<RETURN> OR NEW VALUE, ...


INPUT THE LOADS
===============


AXIAL LOAD, PN, ... 4160


DO YOU WANT TO ENTER MOMENT OR
ECCENTRICITY (M/E), ... M


MOMENT, MX, ... 1310
```

```
MOMENT, MY, ... 505
DO YOU WANT TO CHECK SLENDERNESS ?
Y/N ....Y
INPUT OR FIND TYPE OF FRAME
BRACED OR UNBRACED ?
I/F ...F
HIGHT OF BUILDING,.M..BH=15
TOTAL VERTICAL LOAD,.KN..VL = 5500
NUMBER OF STORIES ...NS = 5
TYPES OF VERTICAL ELEMENTS...TE = 3
# OF SIMILAR VERTICAL ELEMENTS..NE = 15
SECTION DEPTH ...1.6
SECTION WIDTH ...0.3
# OF SIMILAR VERTICAL ELEMENTS..NE = 21
SECTION DEPTH ...1.1
SECTION WIDTH ...0.9
# OF SIMILAR VERTICAL ELEMENTS..NE = 24
SECTION DEPTH ...0.4
SECTION WIDTH ...1.8
CX=.111556907
CY=.088606155
BV=.7
BRACED IN X-DIRECTION
BRACED IN Y-DIRECTION
INPUT OR FIND K VALUES ?
I/F....I
COLUMN-X..B,H,L..  .65,.65,5.5
BRACED EFFECTIVE LENGTH FACTOR-X, KBX=.9
LARGER END MOMENT-X..MX2=1310
SMALLER END MOMENT-X..MX1=1200
BRACED EFFECTIVE LENGTH FACTOR-Y, KBY=.95
LARGER END MOMENT-Y, MY2=505
SMALLER END MOMENT-Y, MY1=400
UNBRACED END MOMENT-X, MX3=350
UNBRACED AXIAL LOAD-X, UPX=5500
DEAD AND LIVE LOAD MMOMENTS-X, DM,LM=600,710
DX=1.07189136
UNBRACED END MOMENT-Y, MY3=270
UNBRACED AXIAL LOAD-Y, UPY=4550
DEAD AND LIVE LOAD MOMENTS-Y, DM,LM=205,300
DY=1.02522489
DBX = 1.07189136         DSX = 1
MX = 1754.17769           EX = 42.1677328
DBY = 1.02522489         DSY = 1
MY = 787.738572           EY = 18.9360234
                 SOLVING
                 SOLVING
```

```
RESULTS
=======


F'C  =  30 MPA        B   = 65 CM.
FY   = 300 MPA        H   = 65 CM.
                      D'  =  5 CM.



RHO            =.028
STEEL AREA     =119.48 SQ CM., (TOTAL)
                29.87 SQ CM., (EACH FACE)
                29.87 SQ CM., (EACH SIDE)

BI-AXIAL  COLUMN ANALYSIS
--------------------------

APPLIED LOAD -- 4160 KN     PHI = 1

CAPACITY ------ 2885 KN     NOT ADEQUATE
                            === --------
HIT <RETURN> TO CONTINUE...
RC.COLUMN 1.0
PROBLEM MENU:
=============


ANALYSIS                          SAMPLE PROBLEM  9
-------   1) UNIAXIAL
          2) BI-AXIAL
          3) CHECK A DESIGN


DESIGN
------    4) UNIAXIAL, INPUTSECTION
          5) UNIAXIAL, FIND SECTION
          6) BI-AXIAL, INPUTSECTION
          7) BI-AXIAL, FIND SECTION


ENTER TYPE OF PROBLEM
       OR '0' TO QUIT, ... 3
RC.COLUMN 1.0
PROBLEM MENU:
=============


ANALYSIS
-------   1) UNIAXIAL
          2) BI-AXIAL


ENTER TYPE OF PROBLEM
       OR '0' TO QUIT, ... 2
```

```
CAPACITY REDUCTION FACTOR = 1

<RETURN> OR NEW VALUE, ...


INPUT THE LOADS
===============


AXIAL LOAD, PN, ... 4160


DO YOU WANT TO ENTER MOMENT OR
ECCENTRICITY (M/E), ... M


MOMENT, MX, ... 1310


MOMENT, MY, ... 505
DO YOU WANT TO CHECK SLENDERNESS ?
Y/N ....Y
INPUT OR FIND TYPE OF FRAME
BRACED OR UNBRACED ?
I/F ...I
BRACED IN X-DIRECTION
Y/N...Y
BRACED IN Y-DIRECTION
Y/N....Y
INPUT OR FIND K VALUES ?
I/F....F
COLUMN-X..B,H,L..  .65,.65,6
UPPER COLUMN-X..B,H,L..  .65 , .65 , 6

<RETURN> OR NEW VALUES...  .5
?.7
?5
LOWER COLUMN-X..B,H,L..  .65 , .65 , 6

<RETURN> OR NEW VALUES...
INPUT NEW COLUMN LENGTHS FOR Y-DIRECTION
Y/N...Y
COLUMN-Y...L = 6

<RETURN> OR NEW VALUE, ...
UPPER COLUMN-Y...L = 6

<RETURN> OR NEW VALUE, ... 5.5
LOWER COLUMN-Y...L = 6

<RETURN> OR NEW VALUE, ... 6.2
RIGHT UPPER BEAM-X..B,D,L..  .4,.7,5
LEFT UPPER BEAM-X..B,D,L..  .4 , .7 , 5

<RETURN> OR NEW VALUES...
```

```
RIGHT LOWER BEAM-X..B,D,L..  .4 , .7 , 5

<RETURN> OR NEW VALUES...  .6
?.7
?4
LEFT LOWER BEAM-X..B,D,L..   .4 , .7 , 5

<RETURN> OR NEW VALUES...
LARGER END MOMENT-X..MX2=1310
SMALLER END MOMENT-X..MX1=950
RIGHT UPPER BEAM-Y..B,D,L..  .6,.8,4
LEFT UPPER BEAM-Y..B,D,L..  .6 , .8 , 4

<RETURN> OR NEW VALUES...  .6
?.9
?5
RIGHT LOWER BEAM-Y..B,D,L..  .6 , .8 , 4

<RETURN> OR NEW VALUES...
LEFT LOWER BEAM-Y..B,D,L..  .6 , .9 , 5

<RETURN> OR NEW VALUES...
LARGER END MOMENT-Y, MY2=505
SMALLER END MOMENT-Y, MY1=445
SLENDERNESS IS NEGLIGIBLE IN X-DIRECTION
F1=24.4943876
F2=25.2977099
UNBRACED END MOMENT-Y, MY3=310
UNBRACED AXIAL LOAD-Y, UPY=4950
DEAD AND LIVE LOAD MOMENTS-Y, DM,LM=205,300
DY=1.08956797
DBY = 1.08956797     -   DSY = 1
MY = 860.231822           EY = 20.6786496
                SOLVING
                SOLVING


RESULTS
=======


F'C  =  30 MPA      B   = 65 CM.
FY   = 300 MPA      H   = 65 CM.
                    D'  =  5 CM.



RHO           =.028
STEEL AREA   =119.48 SQ CM., (TOTAL)
               29.87 SQ CM., (EACH FACE)
               29.87 SQ CM., (EACH SIDE)

BI-AXIAL  COLUMN ANALYSIS
--------------------------

APPLIED LOAD -- 4160 KN     PHI = 1

CAPACITY ------ 3611 KN     NOT ADEQUATE
                            === --------
```

SAMPLE PROBLEM 10

```
HIT <RETURN> TO CONTINUE...
RC.COLUMN 1.0
PROBLEM MENU:
=============


ANALYSIS
-------   1) UNIAXIAL
          2) BI-AXIAL
          3) CHECK A DESIGN


DESIGN
------    4) UNIAXIAL, INPUTSECTION
          5) UNIAXIAL, FIND SECTION
          6) BI-AXIAL, INPUTSECTION
          7) BI-AXIAL, FIND SECTION


ENTER TYPE OF PROBLEM
       OR '0' TO QUIT, ... 3
RC.COLUMN 1.0
PROBLEM MENU:
=============


ANALYSIS
-------   1) UNIAXIAL
          2) BI-AXIAL


ENTER TYPE OF PROBLEM
       OR '0' TO QUIT, ... 2
CAPACITY REDUCTION FACTOR = 1

<RETURN> OR NEW VALUE, ...


INPUT THE LOADS
===============


AXIAL LOAD, PN, ... 4160


DO YOU WANT TO ENTER MOMENT OR
ECCENTRICITY (M/E), ... M


MOMENT, MX, ... 1310


MOMENT, MY, ... 505
DO YOU WANT TO CHECK SLENDERNESS ?
```

```
Y/N ....Y
INPUT OR FIND TYPE OF FRAME
BRACED OR UNBRACED ?
I/F ...I
BRACED IN X-DIRECTION
Y/N...N
BRACED IN Y-DIRECTION
Y/N....N
TYPES OF COLUMNS IN THE STOREY...2
INPUT OR FIND K VALUES ?
I/F....I
COLUMN-X..B,H,L..  .65,.65,5
BRACED EFFECTIVE LENGTH FACTOR-X, KBX=.95
# OF SIMILAR COLUMNS-X...7
UNBRACED EFFECTIVE LENGTH FACTOR-X, KUX=3
BRACED EFFECTIVE LENGTH FACTOR-Y, KBY=.85
# OF SIMILAR COLUMNS-Y...7
UNBRACED EFFECTIVE LENGTH FACTOR-Y, KUY=2.5
LARGER END MOMENT-X, MX2=1310
SMALLER END MOMENT-X, MX1=785
UNBRACED END MOMENT-X, MX3=400
UNBRACED AXIAL LOAD-X, UPX=5550
DEAD AND LIVE LOAD MMOMENTS-X, DM,LM=650,660
TOTAL STORY LOAD, TP=7500
DX=.925778678
LARGER END MOMENT-Y, MY2=505
SMALLER END MOMENT-Y, MY1=410
UNBRACED END MOMENT-Y, MY3=330
UNBRACED AXIAL LOAD-Y, UPY=4900
DEAD AND LIVE LOAD MOMENTS-Y, DM,LM=195,310
DY=.993251964
COLUMN-X..B,H,L..  .4,.5,4
# OF SIMILAR COLUMNS-X...5
UNBRACED EFFECTIVE LENGTH FACTOR-X, KUX=2.7
# OF SIMILAR COLUMNS-Y...5
UNBRACED EFFECTIVE LENGTH FACTOR-Y, KUY=2.6
DEAD AND LIVE LOAD MMOMENTS-X, DM,LM=270,400
DEAD AND LIVE LOAD MOMENTS-Y, DM,LM=265,300
SX=1.20364468
DBX = 1              DSX = 1.20364468
MX = 1791.45787               EX = 43.0638912
SY=1.15050865
DBY = 1              DSY = 1.15050865
MY = 884.667854               EY = 21.2660542
                  SOLVING
                  SOLVING
```

```
RESULTS
=======


F'C  =  30 MPA      B  = 65 CM.
FY   = 300 MPA      H  = 65 CM.
                    D' =  5 CM.


RHO            =.028
STEEL AREA     =119.48 SQ CM., (TOTAL)
                 29.87 SQ CM., (EACH FACE)
                 29.87 SQ CM., (EACH SIDE)

BI-AXIAL  COLUMN ANALYSIS
-------------------------

APPLIED LOAD -- 4160 KN     PHI = 1

CAPACITY ------ 2740 KN     NOT ADEQUATE
                            === --------
HIT <RETURN> TO CONTINUE...
RC.COLUMN 1.0
PROBLEM MENU:
============
```

SAMPLE PROBLEM 11

```
ANALYSIS
-------    1) UNIAXIAL
           2) BI-AXIAL
           3) CHECK A DESIGN


DESIGN
------     4) UNIAXIAL, INPUTSECTION
           5) UNIAXIAL, FIND SECTION
           6) BI-AXIAL, INPUTSECTION
           7) BI-AXIAL, FIND SECTION


ENTER TYPE OF PROBLEM
       OR '0' TO QUIT, ... 3
RC.COLUMN 1.0
PROBLEM MENU:
============


ANALYSIS
-------    1) UNIAXIAL
           2) BI-AXIAL


ENTER TYPE OF PROBLEM
```

```
       OR 'O' TO QUIT, ... 2
CAPACITY REDUCTION FACTOR = 1

<RETURN> OR NEW VALUE, ...


INPUT THE LOADS
===============


AXIAL LOAD, PN, ... 4160


DO YOU WANT TO ENTER MOMENT OR
ECCENTRICITY (M/E), ... M


MOMENT, MX, ... 1600


MOMENT, MY, ... 350
DO YOU WANT TO CHECK SLENDERNESS ?
Y/N ....N
                SOLVING
                SOLVING


RESULTS
=======


F'C  =  30 MPA     B  = 65 CM.
FY   = 300 MPA     H  = 65 CM.
                   D' =  5 CM.


RHO          =.028
STEEL AREA   =119.48 SQ CM., (TOTAL)
               29.87 SQ CM., (EACH FACE)
               29.87 SQ CM., (EACH SIDE)

BI-AXIAL  COLUMN ANALYSIS
--------------------------

APPLIED LOAD -- 4160 KN    PHI = 1

CAPACITY ------ 3599 KN    NOT ADEQUATE
                           === --------
```

SAMPLE PROBLEM 12

```
RC.COLUMN 1.0
PROBLEM MENU:
=============


ANALYSIS
-------   1) UNIAXIAL
          2) BI-AXIAL
          3) CHECK A DESIGN


DESIGN
------    4) UNIAXIAL, INPUTSECTION
          5) UNIAXIAL, FIND SECTION
          6) BI-AXIAL, INPUTSECTION
          7) BI-AXIAL, FIND SECTION


ENTER TYPE OF PROBLEM
       OR '0' TO QUIT, ... 4
CAPACITY REDUCTION FACTOR = .7

<RETURN> OR NEW VALUE, ...


INPUT THE LOADS
===============


AXIAL LOAD, PN, ... 2220


DO YOU WANT TO ENTER MOMENT OR
ECCENTRICITY (M/E), ... E


ECCENTRICITY,EX,...CM = 20


INPUT MATERIAL PROPERTIES
=========================


CONCRETE STRENGTH, MPA = 30

<RETURN> OR NEW VALUE, ...
STEEL YIELD STRESS, MPA = 400

<RETURN> OR NEW VALUE, ...
SECTION WIDTH, CM = 3

<RETURN> OR NEW VALUE, ... 30
```

```
SECTION DEPTH, CM = 65

<RETURN> OR NEW VALUE, ... 60

MINIMUM STEEL RATIO = .01

<RETURN> OR NEW VALUE, ...

MAXIMUM STEEL RATIO = .04

<RETURN> OR NEW VALUE, ...

FRACTION OF STEEL AT SIDES
OF SECTION = .5

<RETURN> OR NEW VALUE, ... 0

COVER TO BAR CENTERLINE, CM = 5

<RETURN> OR NEW VALUE, ...
                SOLVING
                SOLVING
                SOLVING
                SOLVING
                SOLVING
                SOLVING
                SOLVING
                SOLVING


RESULTS
=======


F'C  =  30 MPA       B  = 30 CM.
FY   = 400 MPA       H  = 60 CM.
                     D' =  5 CM.


UNIAXIAL COLUMN DESIGN
---------------------------------------
LOAD           MOMENT    ECCEN.
---------------------------------------
2220             444       20       X
---------------------------------------

RHO          =.024
STEEL AREA   =44.15 SQ CM., (TOTAL)
              22.07 SQ CM., (EACH FACE)
              0 SQ CM., (EACH SIDE)
```

```
HIT <RETURN> TO CONTINUE...
RC.COLUMN 1.0
PROBLEM MENU:
=============


ANALYSIS
-------    1) UNIAXIAL
           2) BI-AXIAL
           3) CHECK A DESIGN


DESIGN
------     4) UNIAXIAL, INPUTSECTION
           5) UNIAXIAL, FIND SECTION
           6) BI-AXIAL, INPUTSECTION
           7) BI-AXIAL, FIND SECTION


ENTER TYPE OF PROBLEM
       OR '0' TO QUIT, ... 3
RC.COLUMN 1.0
PROBLEM MENU:
=============


ANALYSIS
-------    1) UNIAXIAL
           2) BI-AXIAL


ENTER TYPE OF PROBLEM
       OR '0' TO QUIT, ... 1
CAPACITY REDUCTION FACTOR = .7

<RETURN> OR NEW VALUE, ...


INPUT THE LOADS
===============


AXIAL LOAD, PN, ... 2220


DO YOU WANT TO ENTER MOMENT OR
ECCENTRICITY (M/E), ... M6


MOMENT, MX, ... 390
DO YOU WANT TO CHECK SLENDERNESS ?
Y/N ....N
                  SOLVING
```

```
RESULTS
=======


F'C  =  30 MPA      B  = 30 CM.
FY   = 400 MPA      H  = 60 CM.
                    D' =  5 CM.



RHO          =.024
STEEL AREA   =44.15 SQ CM., (TOTAL)
              22.07 SQ CM., (EACH FACE)
              0 SQ CM., (EACH SIDE)

UNIAXIAL  COLUMN ANALYSIS
--------------------------

APPLIED LOAD -- 2220 KN    PHI = .7

CAPACITY ------ 2406 KN        ADEQUATE
                               ========
HIT <RETURN> TO CONTINUE...
RC.COLUMN 1.0
PROBLEM MENU:
=============
```

SAMPLE PROBLEM 14

```
ANALYSIS
-------    1) UNIAXIAL
           2) BI-AXIAL
           3) CHECK A DESIGN


DESIGN
------     4) UNIAXIAL, INPUTSECTION
           5) UNIAXIAL, FIND SECTION
           6) BI-AXIAL, INPUTSECTION
           7) BI-AXIAL, FIND SECTION


ENTER TYPE OF PROBLEM
       OR '0' TO QUIT, ... 3
RC.COLUMN 1.0
PROBLEM MENU:
=============


ANALYSIS
-------    1) UNIAXIAL
           2) BI-AXIAL


ENTER TYPE OF PROBLEM
```

```
        OR 'O' TO QUIT, ... 1
CAPACITY REDUCTION FACTOR = .7

<RETURN> OR NEW VALUE, ...


INPUT THE LOADS
===============


AXIAL LOAD, PN, ... 2220


DO YOU WANT TO ENTER MOMENT OR
ECCENTRICITY (M/E), ... E


ECCENTRICITY,EX,...CM = 20
DO YOU WANT TO CHECK SLENDERNESS ?
Y/N ....Y
INPUT OR FIND TYPE OF FRAME
BRACED OR UNBRACED ?
I/F ...I
BRACED IN X-DIRECTION
Y/N...Y
INPUT OR FIND K VALUES ?
I/F....I
COLUMN-X..B,H,L..  .3,.6,5
BRACED EFFECTIVE LENGTH FACTOR-X, KBX=0.9
LARGER END MOMENT-X..MX2=445
SMALLER END MOMENT-X..MX1=380
UNBRACED END MOMENT-X, MX3=250
UNBRACED AXIAL LOAD-X, UPX=5850
DEAD AND LIVE LOAD MMOMENTS-X, DM,LM=195,250
DX=1.13234109
DBX = 1.13234109             DSX = 1
MX = 753.891786               EX = 33.9590895
                    SOLVING


RESULTS
=======


F'C  =  30 MPA      B  = 30 CM.
FY   = 400 MPA      H  = 60 CM.
                    D' =  5 CM.
```

```
RHO           =.024
STEEL AREA    =44.15 SQ CM., (TOTAL)
                22.07 SQ CM., (EACH FACE)
               0 SQ CM., (EACH SIDE)

UNIAXIAL  COLUMN ANALYSIS
--------------------------

APPLIED LOAD -- 2220 KN    PHI = .7

CAPACITY ------ 1550 KN    NOT ADEQUATE
                           === --------
HIT <RETURN> TO CONTINUE...3
RC.COLUMN 1.0
PROBLEM MENU:
=============
```

SAMPLE PROBLEM 15

```
ANALYSIS
-------  1) UNIAXIAL
         2) BI-AXIAL
         3) CHECK A DESIGN


DESIGN
------   4) UNIAXIAL, INPUTSECTION
         5) UNIAXIAL, FIND SECTION
         6) BI-AXIAL, INPUTSECTION
         7) BI-AXIAL, FIND SECTION


ENTER TYPE OF PROBLEM
       OR '0' TO QUIT, ... 3
RC.COLUMN 1.0
PROBLEM MENU:
=============


ANALYSIS
-------  1) UNIAXIAL
         2) BI-AXIAL


ENTER TYPE OF PROBLEM

       OR '0' TO QUIT, ... 1
CAPACITY REDUCTION FACTOR = .7

<RETURN> OR NEW VALUE, ...


INPUT THE LOADS
===============
```

```
AXIAL LOAD, PN, ... 1560


DO YOU WANT TO ENTER MOMENT OR
ECCENTRICITY (M/E), ... E


ECCENTRICITY,EX,...CM = 20
DO YOU WANT TO CHECK SLENDERNESS ?
Y/N ....Y
INPUT OR FIND TYPE OF FRAME
BRACED OR UNBRACED ?
I/F ...I
BRACED IN X-DIRECTION
Y/N...N
TYPES OF COLUMNS IN THE STOREY...2
INPUT OR FIND K VALUES ?
I/F....I
COLUMN-X..B,H,L..  .3,.6,5
BRACED EFFECTIVE LENGTH FACTOR-X, KBX=.9
# OF SIMILAR COLUMNS-X...7
UNBRACED EFFECTIVE LENGTH FACTOR-X, KUX=2.4
LARGER END MOMENT-X, MX2=312
SMALLER END MOMENT-X, MX1=245
UNBRACED END MOMENT-X, MX3=230
UNBRACED AXIAL LOAD-X, UPX=4500
DEAD AND LIVE LOAD MMOMENTS-X, DM,LM=112,200
TOTAL STORY LOAD, TP=5000
DX=1.02923744
COLUMN-X..B,H,L..  .4,.7,4.5
# OF SIMILAR COLUMNS-X...8
UNBRACED EFFECTIVE LENGTH FACTOR-X, KUX=1.9
DEAD AND LIVE LOAD MMOMENTS-X, DM,LM=200,300
SX=1.06921097
DBX = 1.02923744           DSX = 1.06921097
MX = 567.040604             EX = 36.3487567
                  SOLVING


RESULTS
=======


F'C  =  30 MPA      B  = 30 CM.
FY   = 400 MPA      H  = 60 CM.
                    D' =  5 CM.


RHO          =.024
STEEL AREA   =44.15 SQ CM., (TOTAL)
              22.07 SQ CM., (EACH FACE)
              0 SQ CM., (EACH SIDE)

UNIAXIAL  COLUMN ANALYSIS
--------------------------

APPLIED LOAD -- 1560 KN     PHI = .7

CAPACITY ------ 1480 KN     NOT ADEQUATE
```

APPENDIX III IDENTIFIERS

A) MAIN PROGRAM

1. AE is area of steel along sides
2. AF is area of steel in one face
3. AG is gross section area
4. AN is answer (Dummy)
5. AS is total area of steel in section
6. BA is area of one bar (temporary)
7. BF is number of bars along side of section
8. BN is number of bars
9. BS is bar size (Nominal)
10. BX% is biaxial flag (0-Uniaxial)
11. B is width of section
12. CC is the concrete compression resultant force
13. CD is the distance from NA to compression face steel
14. CE is strain at compression face steel
15. CF is the strain in the side steel at the compression face
16. CH is high limit on neutral axis
17. CX% is flag for design check
18. CL is low limit on neutral axis
19. CP is plateau depth of yielded compression steel

20. CS is the stress in the compression face steel
21. C is depth to neutral axis
22. C1 is resultant of compression face steel
22. C2 is the resultant of compression plateau side steel
23 C3 is the resultant of elastic compression side steel
24. DP is cover to bar center
25. D is effective depth of tension face steel
26. EM is eccentricity roughly corresponding to ACI maximum column load
27. ET is target eccentricity for capacity check
28. EX is eccentricity about X-axis
29. EY is eccentricity about Y-axis
30. EPY is steel yield strain
31. E is eccentricity of load
32. E1, E2, E3 are strains used in finding MØ
33. FPC is concrete strength
34. FY is steel yield stress
35. HB is ratio of section height to width
36. HI is increment of section depth
37. HM is maximum section depth
38. HN is minimum section depth
39. H is depth of section
40. H2 is H/2
41. JD is internal lever arm of face steel
42. K29 is 200000
43. K3 is Ø.ØØ3
44. MB is moment at balanced conditions
45. MØ is moment at zero axial load

46. MX is MX
47. MY is MY
48. NN is nominal column capacity
49. PB load at balanced conditions
50. PHI is capacity reduction factor
51. PN is required capacity for design
52. PT% is problem type flag
53. PX is load capacity with EX
54. PY is load capacity with EY
55. P is load capacity returned from P-M curve
56. PØ is load capacity for pure axial case
57. QP% is flag for problem revision
58. RHO is steel ratio for a section
59. RL is lower bound to steel ratio
60. RN is design minimum steel ratio
61. RS is fraction of steel at sides of section
62. RU is upper bound to steel ratio
63. RX is design maximum steel ratio
64. R is string "RE-ENTER"
65. SB is section width in P-M subroutine
66. SH is section width in P-M subroutine
67. TD is distance from NA to tension face steel
68. TE is the strain in the tension face steel
69. TF is the strain in the side steel at the tension face
70. TL is iteration tolerance
71. TP is plateau depth of yielded tension steel
72. TS is the stress in the tension face steel
73. T1 is the resultant of tension face steel

74. T2 is the resultant of tension plateau side steel

75. T3 is the resultant of elastic tension side steel

76. XT is temporary identifier

77. YD is distance from level of yield strain to neutral axis.

78. Z1 is beta-1

B) SUBROUTINE DESIGN FOR SLENDERNESS

79. AX is eccentricity corresponding M2.

80. AX is eccentricity corresponding N2.

81. BH is hight of building.

82. BK is braced effective length factor in y direction

83. BV is comparison factor for bracing.

84. BX is β_d in x direction

85. BY is β_d in y direction

86. B1 is width of right upper beam.

87. B2 is width of left upper beam.

88. B3 is width of rihgt lower beam.

89. B4 is width of left lower beam

90. CX is unsupported length of middle column in x direction

91. CY is unsupported length of middle column in y direction

92. DX is braced moment magnification factor in x direction.

93. DY is braced moment magnification factor in y direction.

94. D1 is depth of right upper beam

95. D2 is depth of left upper beam

96. D3 is depth of right lower beam

97. D4 is dept of left lower beam

98. EC is modulus of elasticity of concrete.

99. FAX is ψA in x direction.

100. FBX is ΨB in x direction.

101. FAY is ΨA in y direction

102. FBY is ΨB in y direction

103. FI% is flag for effective length factor

104. FMX is the smaller of FAX and FBX

105. FX is the average of FAX and FBX

106. F1 is $k\ell_u/r$ ratio in x direction

107. F2 is 34-12 M_1/M_2 or 22 in x direction

108. GY is the average of FAY and FBY

109. G1 is $k\ell_u/r$ ratio in y direction

110. G2 is 34-12 M_1/M_2 or 22 in y direction

111. H4 is depth of middle column in x direction

112. H5 is depth of upper column in x direction

113. H6 is depth of lower column in x direction

114. IX is total moment of inertia of bracing elements in x direction

115. IY is total moment of inertia of bracing elements in y direction

116. JX is Cm in x direction

117. JY is Cm in y direction

118. KB is braced effective length factor in x direction

119. KU is unbraced effective length factor in x direction

120. LX is unsupported length of lower column in x direction

121. LY is unsupported length of lower column in y direction

122. L1 is length of right upper beam

123. L2 is length of left upper beam.

124. L3 is length of right lower beam

125. L4 is length of left lower beam

126. M1 is smaller end moment about x axis

127. M2 is larger end moment about x axis.

128. M3 is unbraced end moment about x axis

129. NE is number of similar bracing element.

130. NS is number of stories in building.

131. NX is number of similar columns in x direction

132. NY is number of similar columns in y direction

133. N1 is larger end moment about y axis.

134. N2 is smaller end moment about y axis.

135. N3 is unbraced end moment about y axis.

136. O1 is sum of stiffnesses of columns at the upper end in x direction

137. O2 is sum of stiffnesses of columns at the lower end in x direction.

138. O3 is sum of stiffnesses of columns at the upper end in y direction.

139. O4 is sum of stiffnesses of columns at the lower end in y direction.

140. PT is total story load.

141. PX is critical load about x axis.

142. PY is critical load about y axis.

143. R1 is radius of gyration

144. Sx is unbraced moment magnification factor in x direction.

145. Sy is unbraced moment magnification factor y direction

146. SX% is bracing flag in x direction

147. SY% is bracing flag in y direction.

148. TC is number of types of columns in the story

149. TE is number of types of bracing elements.

150. TX is total critical load of columns in the story in x direction.

151. TY is total critical load of columns in the story in y direction.

152. UK is unbraced effective length factor in y direction.

153. UX is unsupported length of upper column in x direction.

154. UY is unsupported length of upper column in y direction.

155. VL is total vertical load in building.

156. W is width of middle column in x direction.

157. W1 is width of upper column in x direction.

158. W2 is width of lower column in x direction.

159. XD is dead load moment in x direction.

160. XE is EI value of middle column in x direction.

161. XL is live load moment in x direction.

162. XR is the ratio of smaller end moment to larger end moment in x direction.

163. XU is the load causing sidesway in x direction.

164. X1 is the sum of stiffnesses of beams at the upper endin x direction.

165. X2 is the sum of stiffnesses of beams at the lower end in x direction.

166. YE is EI value of middle column in y direction.

167. YL is live load moment about y axis.

168. YR is the ratio of smaller end moment to larger end moment in x direction.

169. YU is the load causing sidesway in y direction.

170. YY is dead load moment about y axis.

171. Y1 is the sum of stiffnesses of beams at the upper end in y direction.

172. Y2 is the sum of stiffnesses of beams at the lower end in y direction.