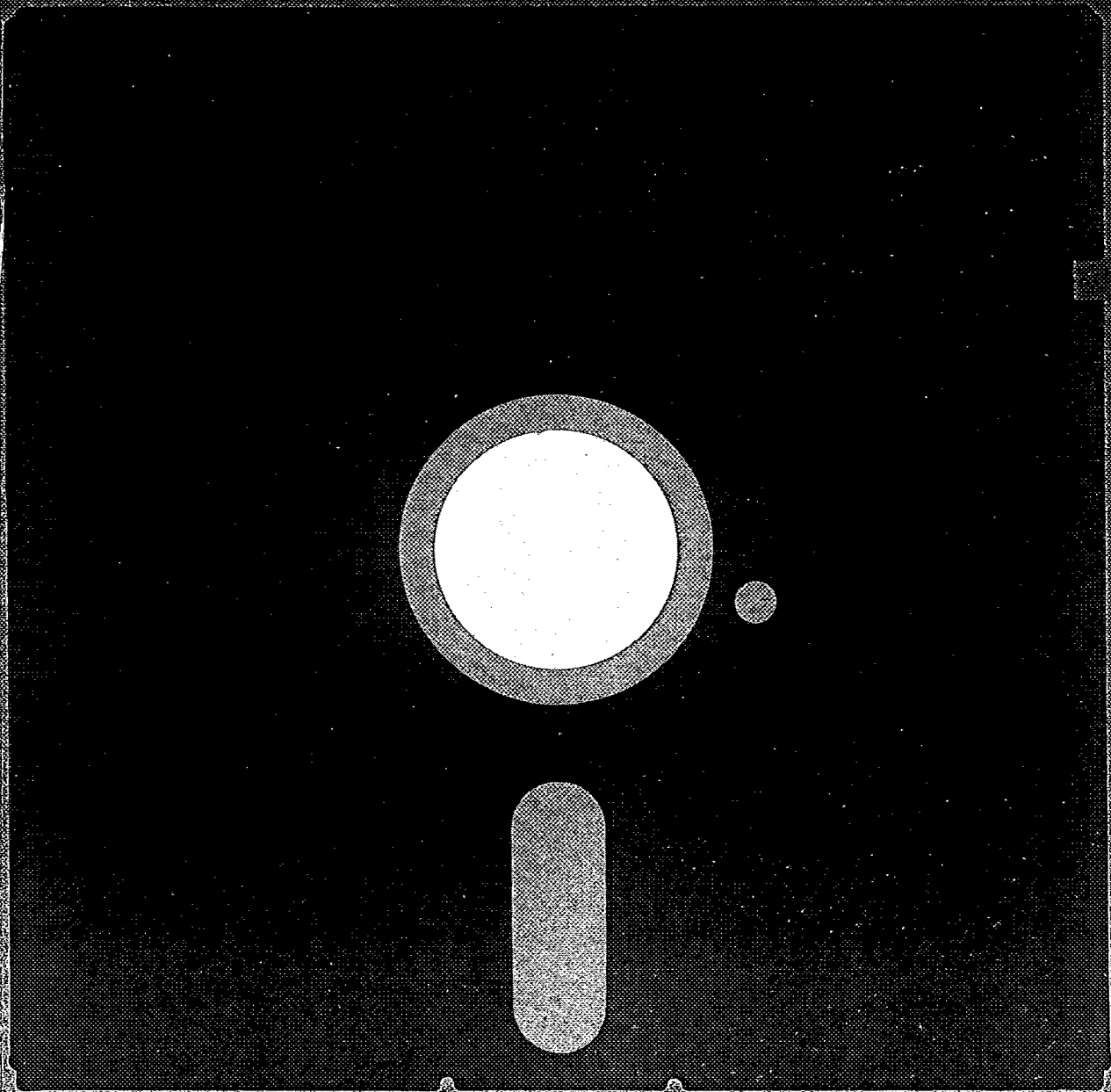


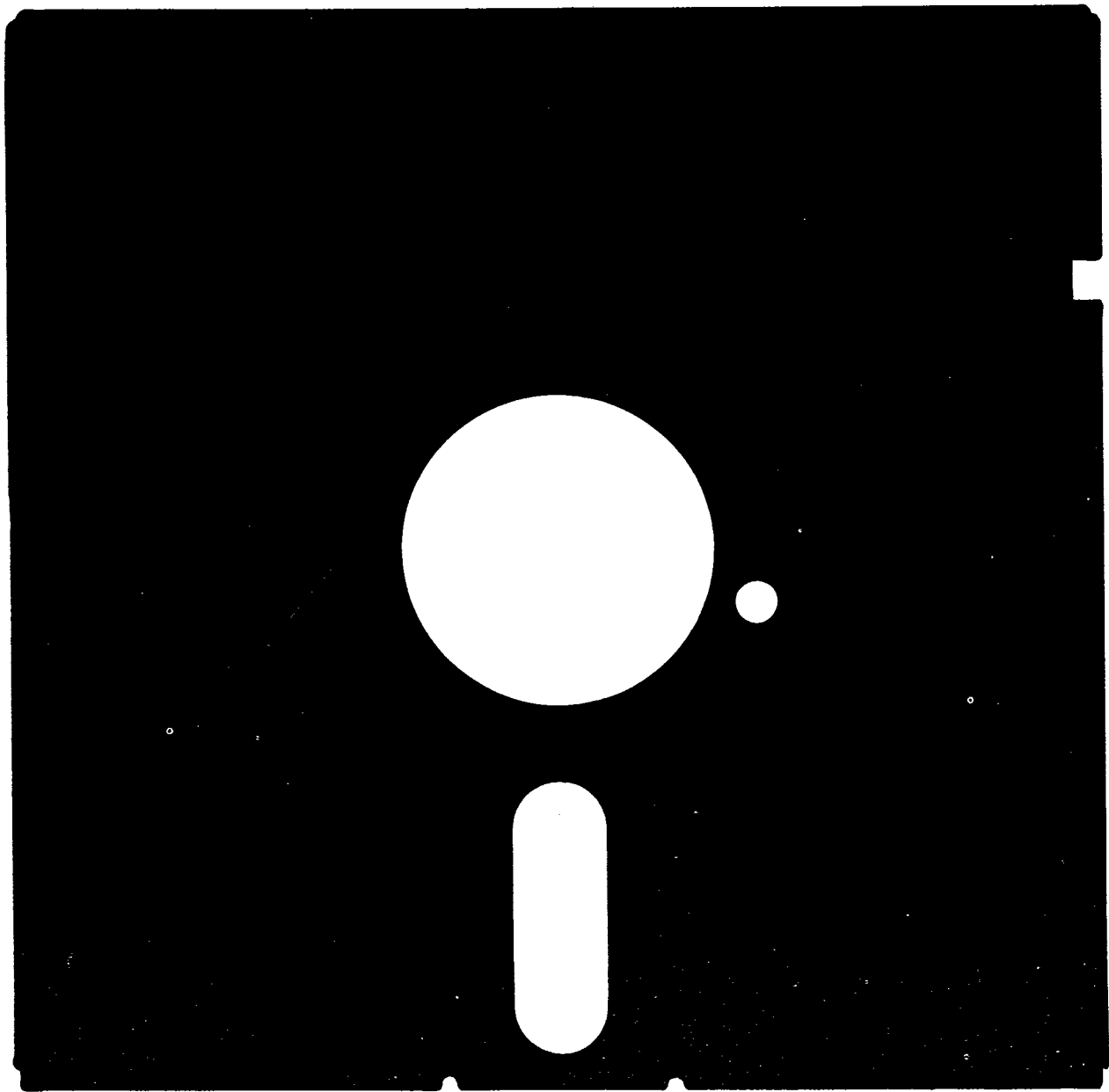
State-of-the-Art Computer Applications in Concrete Technology



Fernando E. Fagundo
Editor

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State-of-the-Art Computer Applications in Concrete Technology



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Editor

American Concrete Institute, Detroit



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PREFACE

This volume contains selected papers presented during two technical sessions on "State-of-the-Art Computer Applications in Concrete Technology" sponsored by ACI Committee 118, Use of Computers. They were held at the 1988 Annual Convention of the American Concrete Institute in Orlando, Florida.

The area of computer applications in concrete technology is growing rapidly. Where it was once the exclusive domain of researchers and scientists, it is now an area where fabricators, detailers, designers, students, and educators are making significant contributions. The different applications presented here will be of interest to individuals dedicated to the correct use of concrete as a construction material. The information is intended to enhance the technological tools available for the correct, safe, and economic use of concrete.

The editor wishes to acknowledge the authors and participants for their contributions to the success of the technical sessions. The effort required to prepare a manuscript for publication can be appreciated only by a person who has experienced the process first hand. We gratefully acknowledge the contribution of committee members who reviewed the articles for clarity and quality. The production of this volume has been possible because of the efforts of many individuals.

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SP111-1

Microcomputer-Aided Design of Reinforced and Prestressed Concrete Elements

by F. C. Filippou

Synopsis: The potential advantages of using electronic spreadsheets in advanced reinforced and prestressed concrete design are presented. The power of this tool is demonstrated through a series of examples which include the moment-curvature relation of reinforced and prestressed concrete beams, the axial load-uniaxial and biaxial bending moment interaction diagram of R/C sections and the tendon layout of simply supported beams prestressed with draped or straight tendons.

Keywords: axial loads, beams (supports), bending moments, microcomputers, prestressed concrete; reinforced concrete; structural design

ACI Member F.C. Filippou is an Assistant Professor in the Department of Civil Engineering at the University of California at Berkeley. He is involved in teaching and research connected with the nonlinear behavior of reinforced and prestressed concrete elements and structures. He is a member of ACI Committee 435, Deflections.

INTRODUCTION

The last ten years have seen a dramatic change in the way engineers perform their work. The most important factor contributing to this change was the introduction of microcomputers into the market in the late 70's. A number of related factors led to the rapid spread of microcomputers in engineering offices of any size:

- (1) the low cost of microcomputers afforded small engineering offices access to the new technology; the large market has led to a technological race which further reduced prices and dramatically increased performance of the typical workstation
- (2) the development of powerful software packages which are easy to learn encouraged non-specialists to use the new technology. The typical engineer cannot afford to spend a large portion of his time learning new tools which in a few years will become obsolete.
- (3) microcomputers help the average engineer in a variety of tasks; these include proposal and report preparation, calculations, budget preparation, scheduling, drafting, presentation etc. The lack of software on mini- and mainframe computers makes many of these tasks difficult, if not impossible, to perform.

The widespread use of microcomputers by professional engineers and their appearance in large numbers on university campuses necessitates their introduction in graduate and undergraduate courses. This fact presents the instructor with the challenge of adjusting course content and presentation to take advantage of the new technology. The use of microcomputers as a learning tool is, however, not an easy task. The pitfalls of the project have led many educators to rejecting microcomputers as a learning tool altogether and adhering to the old and established ways of teaching engineering principles.

The work presented in this paper attempts to show the tremendous potential of the use of electronic spreadsheets in graduate and undergraduate courses of structural design. To demonstrate the potential of this tool attention will be focused on advanced applications which are suited for graduate level courses on behavior and design of structural elements.

ADVANTAGES OF ELECTRONIC SPREADSHEETS

Microcomputers appeared in the market in the late 70's. Electronic spreadsheet programs followed in their footsteps. Some observers of the microcomputer world argue that the first electronic spreadsheet program Visi-Calc launched the entire microcomputer industry.

Electronic spreadsheets can find many applications in the area of structural engineering. The integration of a very powerful electronic calculator with database functions and easy-to-use graphics appears to be especially appealing in structural design. Engineering calculations are often organized in tabular fashion which is precisely the structure that lies at the heart of electronic spreadsheets. The power of these software packages is combined with great flexibility of use allowing the designer to readily develop procedures to fit his special needs. Modification of existing spreadsheets is extremely easy, since the spreadsheet structure is transparent to the user.

These characteristics make electronic spreadsheets an ideal tool in the instruction of structural design. While teaching tools are available in structural analysis [1], no such tools have been developed to date for structural design.

To be successful as a teaching tool a program should satisfy the following requirements:

- (1) the amount of time the student spends learning the details of the program should be kept to a minimum; this amount of time should enable the student to actively interact with the program by modifying portions of it or developing new features and solution schemes;
- (2) the program should be based on a language that is easy to learn and use; most engineering tasks are based on simple functions and solution schemes;
- (3) the program organization should resemble that of the engineering notepad; this facilitates going back and forth between hand and computer calculations

Electronic spreadsheets satisfy all these requirements. Some of the most striking features are:

- (1) ease of programming of complex formulas which include scientific functions and logical operators
- (2) integration of calculation of complex formulas with easy-to-use graphics and basic database concepts; this corresponds to the manual "calculate and look-up" activity of design engineers; the integration of graphics into the spreadsheet allows the student to quickly check his calculations and at the same time enhances his understanding of the interrelationship between key variables of the particular design problem
- (3) the spreadsheet is transparent to the user and can be modified with great ease; this satisfies the need for flexibility in the solution of design problems; at the same time it allows the student to follow the logical flow of the solution process in a spreadsheet that he did not develop himself
- (4) the program does not have to be compiled and linked before it can be executed; execution rather takes place immediately; any syntax errors are flagged as they occur reducing the frustrations associated with debugging the program

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- (5) electronic spreadsheets are geared towards resources that all engineers will possess in the very near future; this fact makes their impact immediate; they illustrate to students the power of computers in a very direct way, since the organization and internal structure of electronic spreadsheets very closely resembles that of manual calculations
- (6) electronic spreadsheets allow the user to investigate the effect of variation of key parameters on the solution of the problem under study; this can be done with a few keystrokes; this should be an important ingredient of instruction tools for structural design
- (7) most electronic spreadsheets are very user-friendly with extensive on-line help facilities; this helps allay the fear of inexperienced users towards microcomputers.

In developing the spreadsheets presented in this paper it was decided to adopt Lotus 1-2-3, because of its features and its widespread use in the business world. Since most of today's successful electronic spreadsheet programs have features similar to those of Lotus 1-2-3, the same spreadsheets can be developed using other packages.

MOMENT-CURVATURE OF REINFORCED AND PRESTRESSED CONCRETE SECTIONS

General

The development of a procedure to derive the moment-curvature relation of reinforced (RC) and prestressed concrete (PC) sections is an important part of any advanced graduate course on reinforced or prestressed concrete design. Understanding the mechanisms that contribute to the characteristic nonlinear behavior of RC sections and being able to assess the influence of important parameters such as reinforcing ratio, amount of compression steel and mechanical properties of reinforcing steel and concrete on the ultimate moment capacity and the curvature ductility of the section is an important first step towards grasping the actual behavior of reinforced and prestressed concrete elements and the approximations involved in the procedures proposed in current design specifications. A thorough investigation of the moment-curvature relation becomes an indispensable tool in connection with the philosophy of capacity design.

In this section the basic procedure for determining the moment-curvature relation of R/C sections will be presented. It is proposed that students enrolled in a basic graduate course on reinforced concrete design be asked to develop a spreadsheet which incorporates this procedure. In this way they will become familiar with the fundamental concepts exactly as they would, if they were asked to solve the same problem by hand.

Following the introduction of the basic spreadsheet the enhancements which form part of spreadsheet **MOMCURV** will be described. These enhancements include the effect of confinement through stirrup-ties or hoops on the stress-strain relation of concrete, the buckling of reinforcing bars under compression, the effect of the slab and the modeling of the uncracked section behavior. Most importantly, the enhanced spreadsheet incorporates an automatic iteration

procedure for finding the location of the neutral axis at each load step and culminates with the graphic representation of the entire moment-curvature relation of the section.

The spreadsheet **MOMCURV** determines the moment-curvature relation of rectangular, T and I-shaped reinforced and prestressed concrete sections. It is capable of accurately representing the load-deformation behavior of RC and PC sections for the entire load range from zero moment up to failure, which is typically initiated by crushing of the cover concrete and buckling of the compression steel. Fracture of the tensile reinforcing steel occurs only in very lightly reinforced sections and T-beams with very wide flanges. The spreadsheet follows the behavior of the section through the uncracked and cracked stages of response. While the uncracked section behavior is of limited value in the case of reinforced concrete sections, it is of great importance in the study of prestressed concrete sections which are typically designed to remain uncracked under service loads.

The spreadsheet **MOMCURV** can then be used for a number of studies of the effects of various parameters on the moment-curvature relation of RC and PC sections. Depending on time constraints in the course these studies can be conducted by the students or provided by the instructor as a set of lecture notes.

It is important to emphasize that the approach outlined above ensures that the student concentrates almost entirely on the concepts behind the derivation of the moment-curvature relation exactly as he would do in the case of hand calculations. His involvement in the refinements of spreadsheet **MOMCURV** or the numerical procedures which automate the calculations is limited to a minimum. Since he has been exposed to the basic concepts, he is, however, able to appreciate the details of the enhanced spreadsheet.

Basic procedure

The procedure for establishing the moment-curvature relation of RC sections is well known and is described in a number of graduate textbooks of reinforced concrete design [2]. It will be presented below for the case of rectangular RC sections which are subjected to an axial load P . In the case of girders with no axial load P is set equal to zero. It is assumed that the section is cracked and that a single material model is used to describe the stress-strain relation of concrete (Fig. 1). The basic assumptions of the bending theory of reinforced concrete sections such as plane sections remaining plane after deformation and the complete compatibility of strains between reinforcing steel and surrounding concrete at the same distance from the neutral axis are adopted here.

The procedure consists of the following steps:

- (1) At a given curvature ϕ the depth of the neutral axis c measured from the top of the section is assumed.
- (2) The compression zone is subdivided into n layers of typically equal height Δx (Fig. 2).

$$\Delta x = \frac{c}{n} \quad (1)$$

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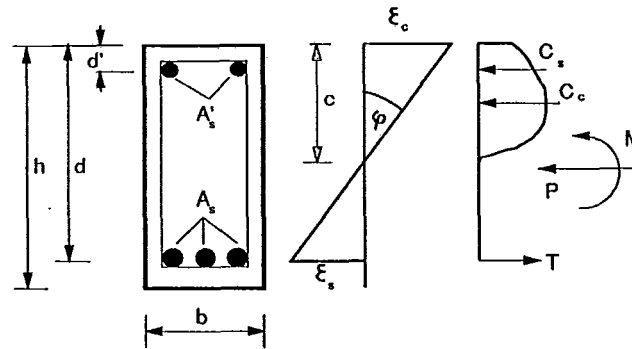


Fig. 1 Distribution of stresses and strains in a rectangular R/C section subjected to bending moment M and axial load P

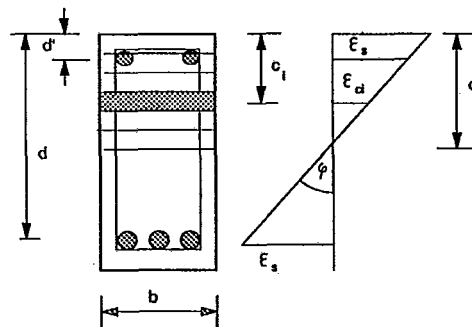


Fig. 2 Subdivision of compression zone in concrete layers

- (3) Based on the curvature ϕ and the assumed neutral axis depth c the strain at the center of gravity of each concrete layer is established

$$\epsilon_{ci} = \phi \cdot (c - c_i) \quad (2)$$

where c_i is the distance of the center of gravity of concrete layer i from the top of the section.

- (4) The strain in each layer of reinforcing steel is established in exactly the same way. For the bottom layer of reinforcing steel this yields

$$\epsilon_s = \phi \cdot (d - c) \quad (3)$$

For the top layer we have

$$\epsilon'_s = \phi \cdot (c - d') \quad (4)$$

- (5) The stress in each concrete layer is established based on the strain determined in (3) and the concrete stress-strain relation

$$f_{ci} = f(\epsilon_{ci}) \quad (5)$$

- (6) The stress in each reinforcing steel layer is established based on the strain determined in (4) and the steel stress-strain relation. For the bottom steel this yields

$$f_s = g(\epsilon_s) \quad (6)$$

For the top steel we have similarly

$$f'_s = g(\epsilon'_s) \quad (7)$$

- (7) The concrete compressive force is calculated by summing up the contribution of all concrete layers

$$C_c = \sum_i f_{ci} \cdot \Delta x \cdot b \quad (8)$$

where b is the width of each layer and is here assumed constant

- (8) The force in the reinforcing steel layers is calculated by multiplying the steel stress in the corresponding layer by the steel area. For the bottom steel

$$T = f_s \cdot A_s \quad (9)$$

and for the top steel

$$C_s = f'_s \cdot A'_s \quad (10)$$

- (9) The summation of all forces from steps (7) and (8) should equal the externally applied axial force P . In the first iteration this is unlikely to be the case. The difference establishes a residual or axial force unbalance P_{unb} which is used in an iterative scheme based on the bisection method to achieve convergence to the correct neutral axis depth c .

$$C_c + C_s - T - P = P_{unb} \quad (11)$$

In Eq. (11) all terms represent the absolute value of the corresponding force with the exception of the externally applied axial load P which is considered positive if compressive and negative if tensile.

- (10) Once the correct neutral axis location is established, the moment resisted by the section at the specific curvature can be determined by summing up the contribution of all concrete and reinforcing steel layers about a convenient reference point. The contribution of the externally applied axial load P should also be accounted for. Taking moments about the bottom reinforcing steel layer yields

$$M = \sum_i f_{ci} \cdot b \cdot \Delta x \cdot (d - c_i) + C_s \cdot (d - d') - P \cdot (h - y_p) \quad (12)$$

where y_p is the distance of the plastic centroid from the bottom fiber of the section. The applied axial load P is assumed positive, if it is compressive, and all other terms involve the absolute value of the corresponding forces.

Implementation in Lotus 1-2-3

The basic procedure outlined above is relatively simple to implement in an electronic spreadsheet. Following the concept of tabular organization which lies at the heart of electronic spreadsheets the student must clearly identify the basic steps of the process, since each step has to be contained in a single cell of the spreadsheet. In addition, he has to translate the entire procedure into a tidy and readable spreadsheet. The spreadsheet is subdivided into two basic parts: the first part contains the input data of the problem while the second part is made up of the cells which calculate the forces and moments associated with all steel and concrete layers. The data of all concrete layers which make up the compression

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zone of the section are represented in a series of columns containing the layer number, the distance of the center of gravity from the top fiber, the strain, the stress, the force and the moment contribution of each concrete layer.

An interesting aspect of the implementation of the basic procedure is the description of the material models for reinforcing steel and concrete. These will be described in more detail later.

The automatic iteration process which establishes the neutral axis depth at each load step presents a serious challenge. This can only be accomplished with the aid of the macro command language of Lotus 1-2-3. It is not expected that the students will be interested in devoting time to such a task in a course concerned with the behavior and design of structural elements. The automatic iteration process should, therefore, be provided by the instructor.

Since the purpose of the first homework assignment related to the moment-curvature relation of R/C sections is to stress the fundamental concepts, the students should only be asked to determine a few characteristic points of the $M-\phi$ diagram by trial and error. There is, therefore, no need for the automatic iteration procedure at this stage. The trial and error approach has the added advantage of providing insight into the nonlinear behavior of R/C sections. It is this ability of electronic spreadsheets to operate in different modes from complete automation down to an entirely interactive mode that makes them great tools of computer-aided instruction.

Moment-curvature of rectangular, T- and I-shaped RC and PC sections

The basic model for establishing the moment-curvature relation of RC sections described in the previous section is satisfactory from the educational standpoint, since it conveys the basic concepts behind the problem. It has, however, severe limitations with regard to the actual load-deformation behavior of typical RC sections.

First and foremost, typical RC sections include various amounts of transverse reinforcement which significantly alters the stress-strain behavior of confined concrete in the strain softening range past the maximum strength. This aspect is very important in the study of the ultimate moment capacity and curvature ductility of plastic hinge regions in beams and columns.

Secondly, most reinforced concrete floor girders are cast monolithically with the slab resulting in T-shaped sections in the positive moment region of the girder span.

With respect to prestressed (PC) and partially prestressed concrete (PPC) sections the following additional points need to be taken into consideration:

- (1) The precracking behavior of the section becomes very important, since most prestressed concrete girders are designed to remain uncracked under service loads.
- (2) The initial strain due to prestressing should be accounted for.
- (3) Prestress losses due to creep, shrinkage, relaxation of prestressing steel and elastic shortening at transfer of prestress should be accounted for.

- (4) Commonly used shapes include I-shaped girders in precast prestressed construction and box girders in post-tensioned bridge construction.

All these important factors have been taken into account in the development of spreadsheet **MOMCURV**. The spreadsheet **MOMCURV** determines the moment-curvature relation of rectangular, T- and I-shaped reinforced and prestressed concrete sections. The spreadsheet follows the behavior of the section through the uncracked and cracked stages of response up to failure which is usually initiated by crushing of the concrete cover and buckling of the compression steel. It takes into account the effect of transverse steel in confining the concrete enclosed by stirrup-ties or hoops. It includes accurate models describing the monotonic stress-strain relation of reinforcing and prestressing steel and accounts for the inelastic buckling failure of reinforcing bars under high compressive strains. More details of **MOMCURV** are presented in [3].

Material models

Reinforcing steel-- The material model describing the monotonic stress-strain behavior of reinforcing steel consists of a linear elastic portion, a plastic yield plateau and a strain hardening portion up to the fracture strain ϵ_u . A parabolic variation of stress with strain is assumed between the strain at the onset of strain hardening ϵ_h and the strain at fracture (Fig. 3).

To accurately represent the behavior of reinforcing steel in compression the effect of buckling of the bars under compression should be taken into account. The inclusion of this effect in instructional software serves the purpose of demonstrating to the student the important role that transverse reinforcement plays in not only confining the concrete, but also restraining the compression reinforcement against buckling. The onset of buckling of the compression steel cannot be represented with a great degree of accuracy, since it depends on a number of factors that are difficult to assess. In the spreadsheet **MOMCURV** it is assumed that the effective length of the reinforcing bars which are laterally supported by stirrups is equal to the stirrup spacing s . Then the critical buckling stress f_{cr} is given by the following relation:

$$f_{cr} = \frac{E_t \cdot \pi^2}{(s/r)^2} \quad (13)$$

where E_t is the tangent modulus of the steel stress-strain relation. To simplify the calculation and because of the inherent uncertainty of the problem an average strain hardening modulus is assumed in calculating the buckling stress. Thus

$$E_t = \frac{(f_u - f_y)}{(\epsilon_u - \epsilon_h)} \quad (14)$$

r in Eq. (13) is the radius of gyration of the reinforcing bar and is equal to 0.25 times the bar diameter.

Prestressing steel-- The stress-strain relation of prestressing steel does not exhibit a well-defined yield point as do most reinforcing steels. As a result, the stress-strain relation presented in the previous section for mild reinforcing steel cannot adequately represent the behavior of prestressing steel. Based on previous studies and the recommendations of Naaman in [4] the model proposed by Menegotto and Pinto was adopted in the spreadsheet **MOMCURV**. The model has the great advantage that the stress is an explicit function of the strain. Since

the strain in the prestressing steel is known for a given curvature and an assumed depth of the neutral axis, the stress can be directly established and no iterations are needed. More details are presented in [3].

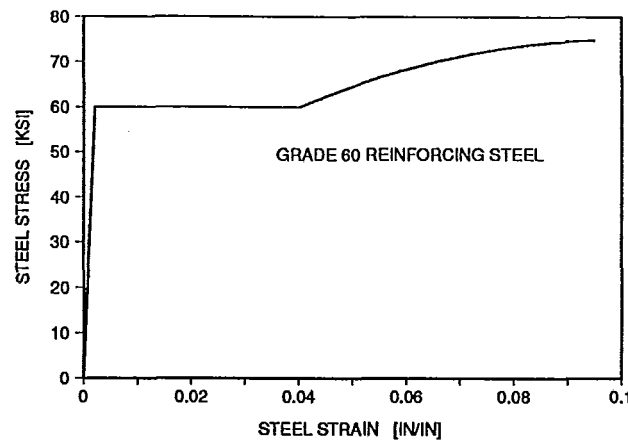


Fig. 3 Material model for reinforcing steel

Concrete-- Many models representing the monotonic stress-strain behavior of plain concrete in uniaxial compression have been proposed in the past [2]. Relatively few models have been proposed for confined concrete. The latter models attempt to account for the effect of longitudinal and transverse reinforcement on the stress-strain relation of concrete which lies within the stirrup-ties or hoops. The model originally proposed by Kent and Park [2] and later modified by Scott, Priestley and Park [5] includes only the effect of transverse reinforcement on confinement. The confinement provided by longitudinal reinforcement in conjunction with transverse reinforcement is not taken into account.

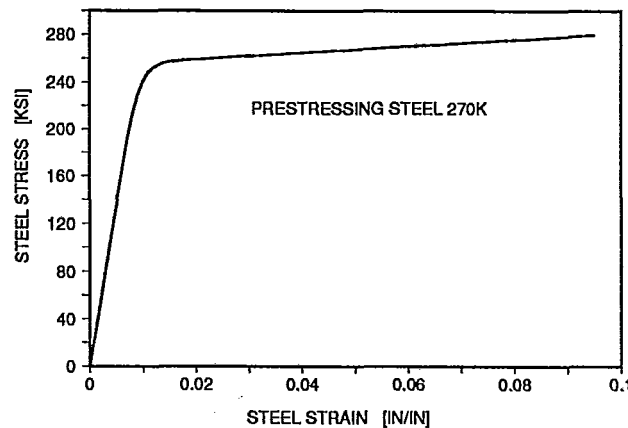


Fig. 4 Material model for prestressing steel

The accurate representation of the effect of confinement on the monotonic stress-strain relation of concrete is of great importance in the study of the load-deformation response of potential plastic hinge regions in beams and columns. By incorporating a confined concrete model in the moment-curvature spreadsheet realistic estimates of the actual moment capacity and curvature ductility of the section can be made.

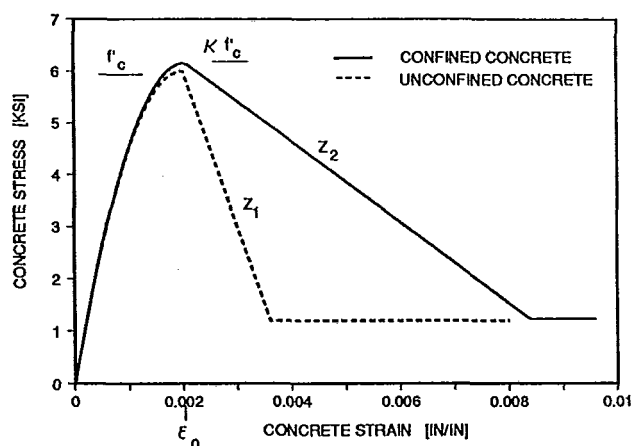


Fig. 5 Material models for confined and unconfined concrete

In the spreadsheet **MOMCURV** the model of Scott et al. is used, because of its relative simplicity (Fig. 5). The monotonic stress-strain behavior of concrete is characterized by an ascending curve, approximated by a parabola, followed by a strain softening range of response represented by a straight line with slope Z . The effect of transverse reinforcement is represented by the transverse reinforcement ratio ρ , which is the ratio of the volume of transverse reinforcement to the volume of concrete core measured to mid-diameter of the hoops. More details of the model are presented in [3,5].

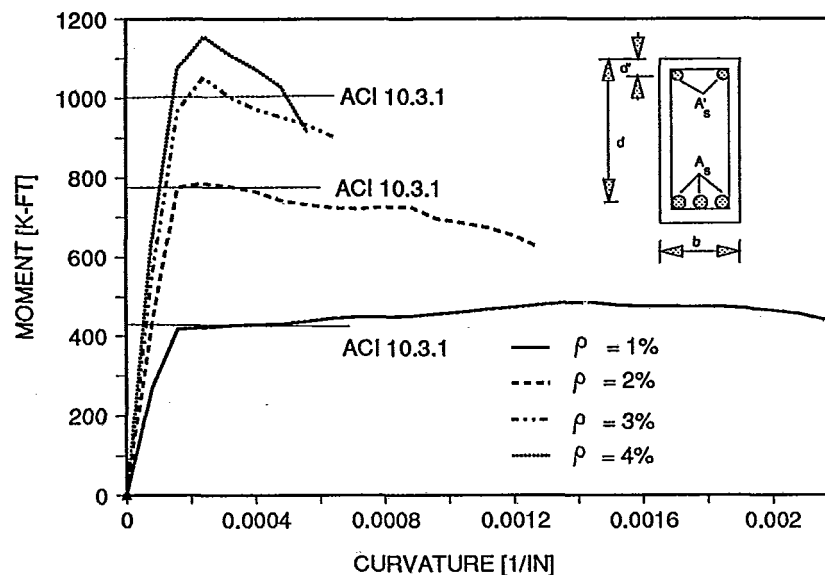


Fig. 6 Effect of tensile reinforcement ratio ρ on moment-curvature relation.
 $\rho' = 0$, $f'_c = 4$ ksi

Examples and parametric studies

Moment-curvature of reinforced concrete sections-- Parametric studies of the effect of various parameters on the monotonic moment-curvature relation of

reinforced and prestressed concrete sections can be conducted using the spreadsheet program described above. Only a small sample of such studies will be presented below.

A typical rectangular reinforced concrete section with top and bottom reinforcement, an effective depth of 25" and a width of 15" has been used in the following studies. Longitudinal and transverse reinforcement consists of Grade 60 steel. Fig. 6 shows the effect of tensile reinforcement ratio ρ on strength and curvature ductility of the section. Fig. 7 shows the effect of concrete compressive strength on strength and curvature ductility. In both cases transverse reinforcement consists of #3 bars spaced at 4" o.c. and no compression reinforcement is used.

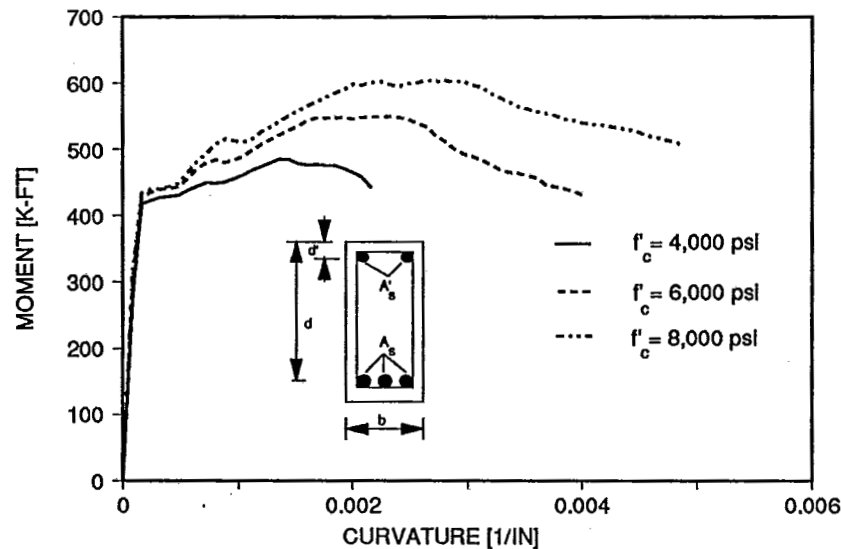


Fig. 7 Effect of concrete compressive strength f'_c on moment-curvature relation. $\rho = 1\%$, $\rho' = 0$

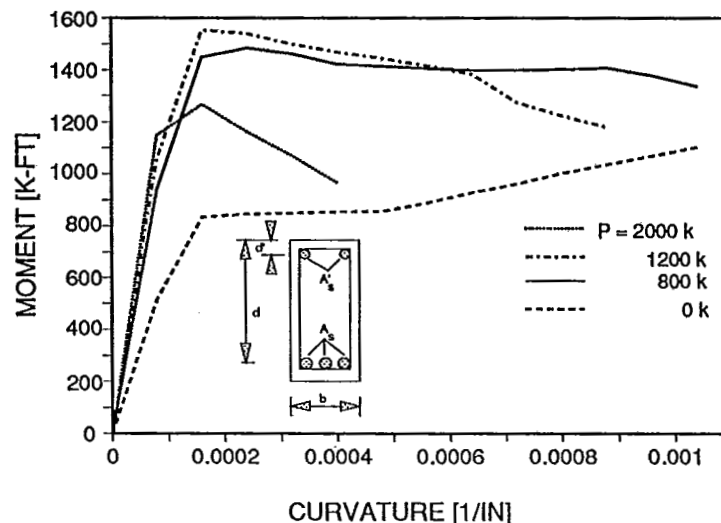


Fig. 8 Effect of axial load P on moment-curvature relation. $\rho = 2\%$, $\rho' = 2\%$, $f'_c = 4$ ksi

Fig. 8 shows the effect of axial load on the moment-curvature relation of the section.

Bending Moment-Axial force interaction diagrams-- A slight modification of the MOMCURV spreadsheet can be used to derive the axial load-bending moment interaction diagrams for rectangular reinforced concrete section. An example of such an investigation is shown in Fig. 9. In this case the strain at the outermost compression fiber is kept constant and the program calculates the axial load and the bending moment of the section for a range of strains in the tensile reinforcement layer. It is interesting to note the effect of the maximum concrete strain on the shape of the interaction diagram. The actual interaction diagram is the envelope of the curves in Fig. 9.

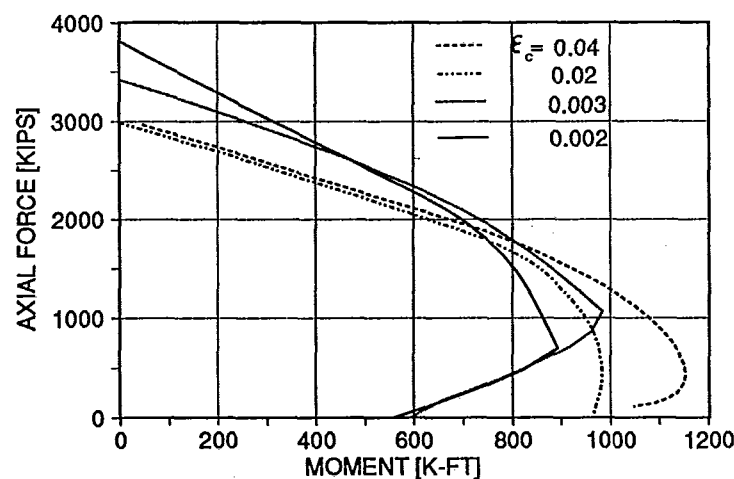


Fig. 9 Axial load-bending moment interaction diagram for a 20" square R/C column with $\rho = \rho' = 2\%$ and transverse reinforcement of #4 bars at 2" o.c., $f'_c = 8$ ksi

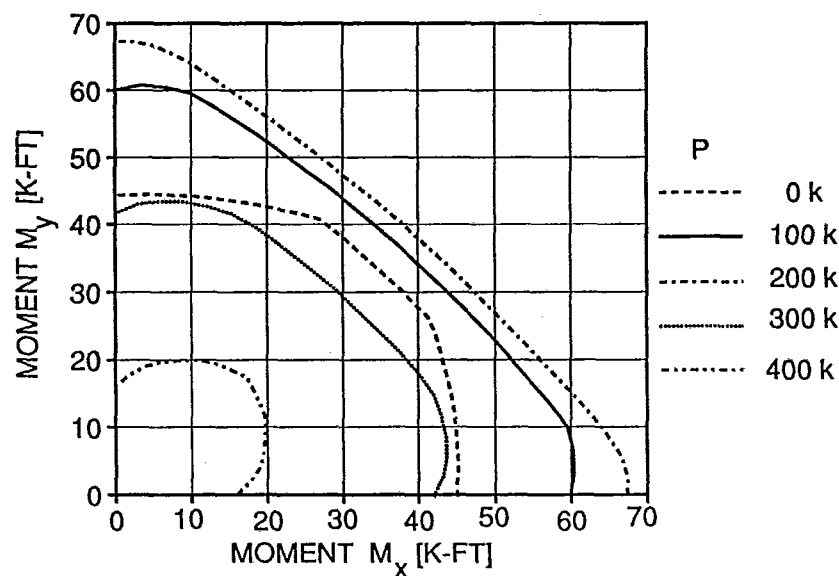


Fig. 10 Axial load-Biaxial bending moment interaction diagram for a 10" square R/C column reinforced with 4 #9 bars, $f'_c = 4$ ksi

Finally, it is possible to let the neutral axis of the section rotate with respect to a fixed coordinate system which is parallel to the edges of the rectangular section. In this case one can obtain the interaction diagram for biaxial bending and axial load. Such a diagram is shown in Fig. 10. Each curve represents the ultimate strength of the section for combinations of bending moments M_x and M_y and a constant value of axial load P .

Design of simply supported prestressed concrete beams--

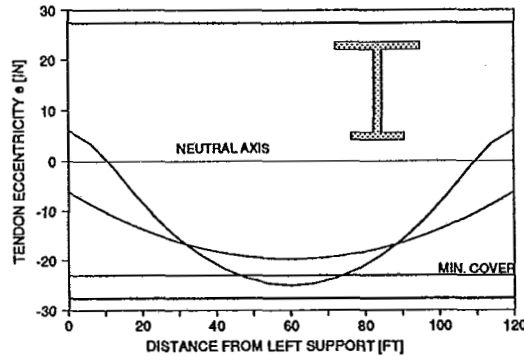


Fig. 11 Tendon limit zone for an unsuccessful trial section

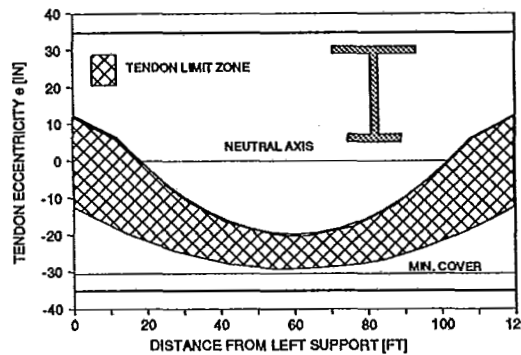


Fig. 12 Tendon limit zone for a successful trial section

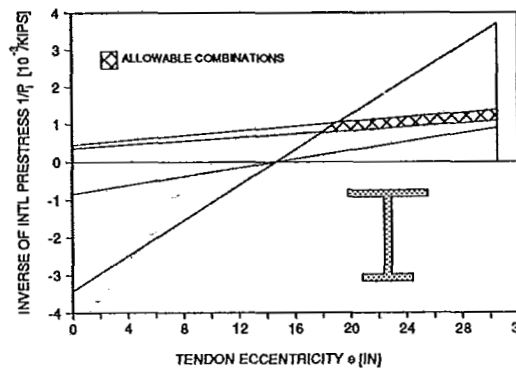


Fig. 13 Magnel diagram of the inverse of the prestressing force vs. the tendon eccentricity at the critical section of the girder of Fig. 12

Figs. 11-13 show a typical application from prestressed concrete design. The students are asked to develop a spreadsheet which will help establish the tendon layout for a simply supported beam prestressed with straight or draped tendons. The only given information is the type of materials used, the span of the beam and the magnitude of the superimposed dead and live loads that the beam carries. The student can go through a trial and error procedure to determine the cross section dimensions. After he has selected a trial section he can ask for a graphic representation of the tendon limit zone. An unsuccessful trial is shown in Fig. 11 and the final section selection is shown in Fig. 12. It takes about two to three minutes to find an optimum cross section characterized by a limit zone which has shrunk to a minimum. Upon selection of the section dimensions the student can plot the Magnel diagram of the section shown in Fig. 13 which enables him to select the minimum prestressing force and the corresponding eccentricity at the critical section.

CONCLUSIONS

This paper demonstrates the possibility of innovative use of electronic spreadsheets in advanced graduate courses on reinforced and prestressed concrete design. In addition, it hopes to advocate the exposure of undergraduate and graduate students to the power offered by electronic spreadsheets in connection with desktop microcomputers in solving various design problems. These problems range from the simplest case of shear design to highly involved nonlinear moment-curvature relations of reinforced and prestressed concrete sections. Familiarity of students with electronic spreadsheets is all the more important, since these can be used to great advantage in many other engineering tasks such as construction scheduling, budget preparation, etc.

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SP 111-2

COLUMN: A KBES for the Design of Reinforced Concrete Columns

by A. S. Bezzina and S. H. Simmonds

Synopsis: COLUMN is a computer program written to select cross sections for reinforced concrete columns by simulating the reasoning processes used by structural engineers. The logic used to select, evaluate and, if required, modify a section is described. Provision is made for the designer to impose, a priori, any restriction on the concrete dimensions or reinforcement that may be desired. The program proportions and details tied and spiral columns, braced or unbraced for either uniaxial or biaxial loading. Slenderness effects are considered using the moment magnifier method. The design of a tied column with examples of different user imposed restrictions is included.

Keywords: columns (supports); computer programs;
expert systems; reinforced concrete; spiral columns;
structural design; tied columns

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The steps in the design of a reinforced concrete member section are; select a trial section, evaluate the capacity of this section to resist a given loading and, if required, modify the section. Most computer programs written to date for the design of reinforced concrete members are limited to the second step, that is, they evaluate the capacity of a member section that has been selected by the user. This is not surprising since this step essentially involves only well defined numerical algorithms which are readily programmed. On the other hand, the engineering logic and decisions inherent in the other two steps are subjective and implicit in nature and are dependent on the acquired knowledge and expertise of the structural engineer.

Feigenbaum (1) defines an expert system as:

"An expert system is an intelligent program that uses knowledge and inference procedures to solve problems that are difficult enough to require significant human expertise in their solution. The knowledge necessary to perform at such a level, plus the inference procedure used can be thought of as a model of the expertise of the best practitioners in the field."

Thus, a program that possesses the necessary knowledge and the ability to use it intelligently to proportion and detail structural elements is a KBES (knowledge-based expert system) in that domain.

Knowledge has been classified as public and experiential (2). Public knowledge in structural engineering includes well defined theories and facts as such the principles of equilibrium in mechanics. That reinforcement bars are available commercially only in certain diameters is a more domain specific example. Code specifications are frequently based on experimental research. However, when these quasi-empirical

methods are codified into standards, they become established information constituting part of the public knowledge available to the engineer.

Experiential knowledge consists of rules of thumb, intuitive design concepts and judgement unique to a specific field that the engineer possesses that marks him as an expert in that field to others. In KBES for structural design this knowledge is generally represented by production rules and heuristics which are rules that provide the direction in solving problems that lack formal mathematical reasoning or relationships.

A reinforced concrete member cross section, to be considered satisfactory, must satisfy the following requirements:

1. Safe and serviceable
2. Constructable
3. Economical
4. In accordance with any limitations or requirements imposed by the engineer.

The first three requirements are self-explanatory. The fourth may require elaboration.

In practice, the engineer is required to take ultimate responsibility for the design. In most cases, adherence to a particular building code is stipulated in the Contract of Works. In addition, for a variety of reasons such as availability of forms, unique features of the structure or adjustments required to accommodate changes in architectural and mechanical aspects, the engineer may need to impose limitations on the selection of, or prescribe specific values to, the design parameters. Therefore, although the overall solution strategy is the complete cycle approach, at each decision step there must be the opportunity for the designer to impose, a priori, a different heuristic or strategy that will be recognized by the program.

This paper discusses the implementation of these concepts in the program COLUMN (3). COLUMN was written to explore the requirements of a program that, given the same information, will proportion and detail a reinforced concrete column section that is comparable to that produced by an experienced structural engineer. COLUMN is, therefore, a KBES in the domain of designing reinforced concrete columns.

PROGRAM COLUMN

The least data required to proportion a column section by even the most experienced designer are the material properties, the axial load and bending moments acting and the effective

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column length. This last parameter is dependent on the end conditions and the degree of bracing. Furthermore, the current code design procedures for unbraced columns require distinguishing between moments from loading that cause no appreciable lateral deflection and moments from loading that do cause appreciable lateral deflection. If the program were part of a comprehensive CAD system, initial values for this data would be available through the data base to which it was linked. When the program is used on a stand alone basis, this essential data must be input.

Some data, such as the concrete cover and minimum reinforcement ratios are specified in the codes. Other data, such as dimension increments and minimum and maximum bar sizes, are designer selected. Since these and similar data apply to most designs, they can be incorporated into the knowledge base as default values and need not be entered unless a different value is to be imposed.

COLUMN incorporates the requirements of the American (4) and Canadian (5) building codes. Although these two codes use the same overall philosophy for the design of columns, in addition to the differences in units and available reinforcement sizes, there are major differences in the details of application of load factors, material strength reduction factors and stability factors. Designating the applicable code automatically sets default values for many of the design parameters.

To facilitate placing in the field, it is common practice to use patterns of reinforcement that are symmetric about each principal axis and have reinforcement in only one layer. Such patterns are shown in Fig. 1. Although COLUMN will proportion columns that are described as tied or spiral, short or long, braced or unbraced with uniaxial or biaxial loading, these are the only patterns that will be selected.

Initial Concrete Dimensions

Selecting the initial trial concrete dimensions first involves selecting the column type and shape. The following heuristics are used in COLUMN to achieve this. The column type defaults to tied unless spiral is specified. Further, the default shapes are rectangular and circular for tied and spiral columns, respectively. Hence, unless the designer indicates otherwise, a priori, the column section will be tied and rectangular.

The approach used to select dimensions for the initial section is to find an initial value of the gross concrete area that will be close to the required value but is known to be on the small side. Thus, if the section is found to be inadequate, the section need only be incremented. The initial value of the

gross concrete area is obtained from expressions for the maximum factored axial load contained in the codes. By introducing the reinforcement ratio, ρ , the equation in clause 10.3.5. (4) can be written as:

$$A_g = P_u / BC(0.85f'_c + \rho(f_y - 0.85f'_c)) \quad [1]$$

where B is 0.7 and 0.75 and C is 0.8 and 0.85 for tied and spiral columns, respectively. All terms on the RHS are known except ρ . By selecting a maximum value for ρ , a minimum value for A_g will be obtained.

When the shape has been specified as square or circular or, if rectangular, one column dimension or the column aspect ratio has been input, the corresponding theoretical dimensions are easily calculated. The actual dimensions for the trial section are obtained by increasing these theoretical dimensions to be a multiple of the dimension increment. The dimension increment has default values of 2 in. or 50 mm depending on the units used. These trial dimensions are also checked against minimum dimensions having default values of 8 in. or 200 mm.

When no restrictions to the column shape have been specified, the shape is rectangular. To obtain trial dimensions from the gross area it is first necessary to select a column aspect ratio, R, where $R = c_x / c_y$. The inference rules for selecting R are summarized in Fig. 2 and are based on the eccentricities, e, of the axial load. A value of e that is large compared to the corresponding dimension infers that this dimension should be increased so as to resist the applied loading more efficiently. From the known eccentricities and values for column dimensions obtained assuming a square section, the ratios e_x / c_x and e_y / c_y are computed. The procedure for determining theoretical and actual dimensions for the initial section then follows that outlined above.

Longitudinal Reinforcement

The selection of longitudinal reinforcement requires determining the pattern, size and number of reinforcing bars. In the absence of user direction, COLUMN selects the reinforcement pattern again based on eccentricity to column dimension ratios except that this time the ratios are obtained using the trial dimensions. When the two ratios are similar, bars are placed on all four faces. However should one moment dominate, bars are placed only on the two short faces. The rules for this are summarized in Fig.3, where N_x is the number of bars in the x-face including corner bars and N_y is the number of bars in the y-face not including corner bars unless all of the bars are placed in this face.

Unless the splice length is a governing factor, it is usually more economical to use the smallest number of bars that

will provide the required area and satisfy spacing requirements. This reasoning leads to the heuristics implied in Fig. 4. These can be expressed mathematically as $c/k + 1$ and $c/k + 2$ for the minimum and maximum number of bars per face, respectively, where c is the face dimension and k is either 8 in. or 200 mm. Thus, a column with dimensions 12 X 16 in. and with bars on all faces would have a minimum of 6 and a maximum of 10 bars unless overridden by another heuristic triggered by a user imposed restriction. The corresponding numbers would be 4 and 6 if bars are placed only on the two short faces.

From the dimensions of the trial section and the minimum and maximum reinforcement ratios, the corresponding reinforcement areas and number of bars for each bar size can be determined.

The first trial for the longitudinal reinforcement is the smallest number of bars of the smallest size that are consistent with the column dimensions, selected pattern and required area. This section is then evaluated. If found to be inadequate, the number of bars is incremented and the section re-evaluated. When the maximum number of bars is reached the bar size is incremented until the maximum bar size is reached after which the concrete dimensions are incremented. This process continues until a satisfactory section is found.

As indicated, the above production rules apply only when no restrictions have been imposed by the user. Otherwise, other heuristics are invoked depending on the restrictions imposed. For example, for the case where the designer inputs the concrete dimensions and specifies a particular bar size, the program will use these dimensions without modification and continue to add bars of that size until a satisfactory solution is obtained or spacing requirements are violated or the maximum reinforcement ratio is exceeded. In the latter two situations the design is terminated and the designer is informed as to why the design could not be completed in accordance with the conditions imposed. Other heuristics for selecting longitudinal reinforcement are generally combinations of the above two extreme cases.

Evaluating Trial Section

Since COLUMN was written to parallel the manual solution, the trial section is evaluated using the moment magnifier method contained in the standards. Biaxial bending is considered using the Equivalent Eccentricity Method although other approaches such as the Bresler Reciprocal Method or the Load Contour Method could have been implemented.

The first step is to determine whether the moments input for a particular load case need to be modified to meet minimum moment and slenderness requirements. All moments are input,

either factored or unfactored, using the frame sign convention. For purposes of design, the program determines which of the end moments is larger and sets the value of this moment, M_2 , to be positive. The smaller end moment, M_1 , is set to a positive value when the column is bent in single curvature and negative when bent in double curvature.

The ratio, M_1/M_2 , is used in the Code in two places; for calculating C_m , the equivalent moment factor and for checking whether slenderness effects are required for a braced column. For those cases where analysis indicates the end moments are zero or result in computed end eccentricities smaller than specified values, the codes indicate the value to be assigned to the M_1/M_2 ratio for purposes of calculating C_m . The codes do not, however, give guidance for assigning values to this ratio for checking slenderness of braced columns. The approach used in COLUMN is to set M_1 equal to the smaller computed factored end moment and M_2 to the larger, increased to the minimum specified end moment, if required.

For the trial section, the value of the critical load, P_c , and hence the moment magnifier for the moments associated with no appreciable sidesway, δ_b , can be computed using expressions given in the standards. On the other hand, the moment magnifier associated with sidesway, δ_s , is dependent on all of the columns in the story. The normal approach in design practice is to assume a value of δ_s and following completion of trial designs for all compression members in the story the value of δ_s is either verified or a new value is assumed and the process repeated until the computed value of δ_s converges to the assumed value. Obviously this approach requires a linked data base and a CAD environment.

For the initial sizing of slender unbraced columns, COLUMN requests values for ΣP_u and the number of columns in the story as this information would be available in the data base if the program were linked to a CAD system. If no other information is available, the value of ΣP_u is obtained assuming that all other columns in the story are identical to the one being considered. The value for δ_s is then computed. When the value of δ_s is greater than 2 or less than 1 the concrete dimensions are incremented and the process repeated since in both cases the column is considered to be structurally unstable. For design of a single column the value of δ_s obtained on this basis must be independently verified. If linked to a CAD system, in each iteration after the first, the value of ΣP_c would be obtained by adding the updated critical load value for the column being considered to the values for the other columns in the story obtained in the previous iteration.

The biaxially loaded column is reduced to a uniaxially loaded column using the Equivalent Eccentricity Method (6). When e_x/c_x is greater than or equal to e_y/c_y , biaxial bending is

simulated by uniaxial bending about the y-axis with the factored axial load acting at an equivalent eccentricity, e_{ox} , where

$$e_{ox} = e_x + \alpha e_y c_x / c_y \quad [2]$$

Similarly, when e_y/c_y is greater than or equal to e_x/c_x , the loading is replaced by the factored axial load acting at an equivalent eccentricity, e_{oy} , where

$$e_{oy} = e_y + \alpha e_x c_y / c_x \quad [3]$$

In these expressions, when $P_u/A_g f'_c \leq 0.4$

$$\alpha = [0.5 + P_u/A_g f'_c][(f_y + 300)/700] \geq 0.6 \quad \text{SI} \quad [4]$$

$$\alpha = [0.5 + P_u/A_g f'_c][(f_y + 40000)/100000] \geq 0.6 \quad \text{US}$$

and when $P_u/A_g f'_c > 0.4$

$$\alpha = [1.3 - P_u/A_g f'_c][(f_y + 300)/700] \geq 0.5 \quad \text{SI} \quad [5]$$

$$\alpha = [1.3 - P_u/A_g f'_c][(f_y + 40000)/100000] \geq 0.5 \quad \text{US}$$

The capacity of the trial section is evaluated for the factored axial load and factored moments modified, if required, for slenderness and biaxial effects using conditions of strain compatibility and equilibrium obtained using a linear distribution of strain and the other assumptions listed in the codes. The program first checks to ensure that the pure axial compression capacity is greater than the factored axial load, otherwise the section is incremented.

The solution algorithm involves finding the position of the neutral axis such that the axial load capacity is greater than but within an arbitrary limit ($0.0025 P_u$) of the factored axial load. This is accomplished using a halving technique wherein the initial position is assumed to be midway between an upper limit, c_u , corresponding to the position of the neutral axis when the depth of rectangular compression block is equal to the depth of section and a lower limit, c_l , corresponding to the pure axial tension case. When the computed axial load capacity is greater than the factored axial load the location of the upper limit is moved to the current position of the neutral axis and a new position for the neutral axis midway between the revised limits is tried. When the computed axial load capacity is less than the factored axial load the process is similar except that it is the location of the lower limit that is moved. When convergence to the prescribed tolerance cannot be achieved, the section is incremented.

When convergence is achieved the corresponding moment capacity of the section is computed and compared to the maximum factored moment. If the computed moment capacity is greater the

section is adequate, otherwise the section is incremented.

Incrementing Column Section

When a section needs to be incremented, except where the logic dictates incrementing the concrete dimensions directly, the program first tries to increase the amount of longitudinal reinforcement using the procedures outlined earlier for selecting such reinforcement. When a satisfactory section cannot be obtained by incrementing the reinforcement, consistent with the limitations imposed, the concrete dimensions are incremented.

In COLUMN, when the concrete dimensions are to be incremented, a new value of the theoretical gross area is obtained as the sum of the gross area of the previous trial section and an increment of area equal to the dimension increment times the smaller dimension of the previous trial section. Using this new value the column shape, theoretical dimensions and actual dimensions of the next trial section are determined using the same heuristics that were used to determine the dimensions of the initial trial section. In this way the most efficient section consistent with user imposed limitations is attained.

EXAMPLE USING COLUMN

This example illustrates the differences in sections selected by COLUMN when different constraints are designer imposed. The problem is to design a braced tied column in accordance with CAN3-A23.3-M84 (5) using $f'_c = 30$ MPa, $f_y = 400$ MPa, No. 10 ties with 40 mm cover and a dimension increment of 20 mm for the following factored loading;

$$P_u = 1400.0 \text{ kN}, M_{ux} = 252.0 \text{ kNm and } M_{uy} = 560.0 \text{ kNm.}$$

Three cases of additional imposed constraints are:

- (a) no additional restraints
- (b) section to be square but no restrictions on reinforcement
- (c) section to be square with equal reinforcement on all four faces.

An example of the output format for the first case is given in Appendix A. The first sheet is an echo of the input. It is observed that for those quantities where values were input or default values are assigned, numerical values are given. Where the parameters are to be determined by the program the value assigned is 0.0. Since the loading was entered as factored it is listed as if it were due to dead loading only.

The second sheet contains computed values. The design factored loads are the same as those entered since the column was braced and the moments were not modified. When the moments

are modified, the modified moments are printed and the reason for the modification is given in the additional comments section. Since one of the moments is significantly larger than the other, the column section selected is rectangular with all of the reinforcement in the short faces. The reinforcement ratio, tie spacing and clear spacing for the longitudinal bars are also printed to assist the engineer in evaluating the design.

To conserve space, only a portion of the output is given for the other two cases of constraints. It is seen that when the section was specified as square, the concrete volume was increased by 3.8% and the reinforcement area by 33% even though bars are still placed only on two faces. When the additional constraint that bars were to be placed on all faces, the concrete volume is increased by 20% and the reinforcement area again by 33% compared to the case where no additional constraints were imposed. The example demonstrates that the heuristics used in the program will determine the most economical section consistent with user imposed constraints.

CONCLUSIONS

It is demonstrated that it is possible to develop a knowledge-based expert system that will select column sections that are comparable with those that would be selected by an experienced structural engineer given the same design information.

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APPENDIX A - EXAMPLE OUTPUTS

COLUMN

Column version 1.0 (Jul/1987) - knowledge-based computer program for the design and analysis of reinforced concrete columns.

Developed at the Department of Civil Engineering, University of Alberta, Edmonton, Alberta, Canada by Alexander Bezzina and Sidney H. Simmonds

PROJECT: BIAXIAL BENDING (a) COLUMN ID: A1 at 2nd FLOOR

MODE: DESIGN MODE CODE: CSA CAN3-A23.3-M84

MATERIAL PROPERTIES:concrete:

f c = 30.0 N/sq.mm
Wt. = 2400.0 kg/cu.m
Ec = 27691.5 N/sq.mm
type = NORMAL DENSITY

steel:

fy (long.) = 400.0 N/sq.mm
fy (lat.) = .0 N/sq.mm
Es = 200000.0 N/sq.mm

SECTION DATA:concrete section:

shape = RECTANGULAR
cx = .0 mm
cy = .0 mm
R = .0 mm
cover = .0 mm
cx incr. = 20.0 mm
cy incr. = 20.0 mm

reinforcement:

type = TIED
reinf. pattern = UNSPECIFIED
splice type = RADIAL
max. reinf. ratio = .030
max. no. of bars =
no. bars x-face =
no. bars y-face =
max. bar size = #35
min. bar size = #15

COLUMN DATA:

type = BRACED ABOUT X-AXIS
lx = 4000.0 mm
kx = .75

type = BRACED ABOUT Y-AXIS
ly = 4000.0 mm
ky = .75

INPUT LOADING:

type = FACTORED

axial loads:

Pd = 1400.0 kN
Pl = .0 kN

moments:top end:

Mxd = 252.0 kN.m
Myd = 560.0 kN.m
Mxl = .0 kN.m
Myl = .0 kN.m

bottom end:

Mxd = .0 kN.m
Myd = .0 kN.m
Mxl = .0 kN.m
Myl = .0 kN.m

Example Output - Case (a) Sheet 1

DESIGN - COLUMN ID: A1 at 2nd FLOOR

DESIGN FACTORED LOADS:

P = 1400.0 kN
M_x = 252.0 kN.m
M_y = 560.0 kN.m

SLENDERNESS RATIOS:

(k_l/r) - x = 24.7
(k_l/r) - y = 16.8

COLUMN DESIGN:

concrete section:

c_x = 620.0 mm
c_y = 420.0 mm
cover = 40.0 mm

reinforcement:

no. bars x-face = 3 - #35
no. bars y-face =
total no. of bars = 6 - #35
reinf. ratio = .023
tie sets - #10 @ 420.0 mm

ADDITIONAL DESIGN COMMENTS:

COLUMN DEFLECTS IN SINGLE CURVATURE ABOUT Y-AXIS

COLUMN DEFLECTS IN SINGLE CURVATURE ABOUT X-AXIS

SLENDERNESS EFFECTS ABOUT X-AXIS DUE TO STABILITY MAY BE
NEGLECTED

SLENDERNESS EFFECTS ABOUT Y-AXIS DUE TO STABILITY MAY BE
NEGLECTED

RADIAL SPLICES CAN BE USED

CLEAR SPACING OF BARS ON THE X-FACE = 105.1 mm

CLEAR SPACING OF BARS ON THE Y-FACE = 446.0 mm

Example Output - Case (a) Sheet 2

DESIGN - COLUMN ID: A1 at 2nd FLOOR

DESIGN FACTORED LOADS:

P = 1400.0 kN
M_x = 252.0 kN
M_y = 560.0 kN.m

SLENDERNES RATIO:

(k_l/r) - x = 20.0
(k_l/r) - y = 20.0

COLUMN DESIGN:

concrete section:

cx = 520.0 mm
cy = 520.0 mm
cover = 40.0 mm

reinforcement:

no. bars x-face = 4 - #35
no. bars y-face =
total no. of bars = 8 - #35
reinf. ratio = .030
tie sets - #10 @ 520.0 mm

ADDITIONAL DESIGN COMMENTS:

CLEAR SPACING OF BARS ON THE X-FACE = 91.5 mm

CLEAR SPACING OF BARS ON THE Y-FACE = 346.0 mm

Example Output - Case (b) Portion of Sheet 2

DESIGN - COLUMN ID: A1 at 2nd Floor

DESIGN FACTORED LOADS:

P = 1400.0 kN
M_x = 252.0 kN
M_y = 560.0 kN.m

SLENDERNES RATIO:

(k_l/r) - x = 18.6
(k_l/r) - y = 18.6

COLUMN DESIGN:

concrete section:

cx = 520.0 mm
cy = 520.0 mm
cover = 40.0 mm

reinforcement:

no. bars x-face = 3 - #35
no. bars y-face = 1 - #35
total no. of bars = 8 - #35
reinf. ratio = .026
tie sets - #10 @ 542.4 mm

ADDITIONAL DESIGN COMMENTS:

CLEAR SPACING OF BARS ON THE X-FACE = 175.2 mm

CLEAR SPACING OF BARS ON THE Y-FACE = 175.2 mm

Example Output - Case (c) Portion of Sheet 2

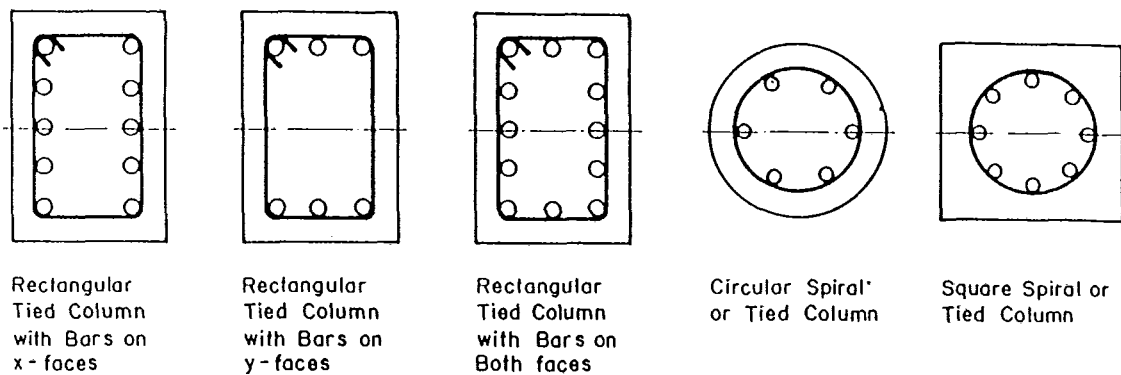


Fig. 1--Sections considered by COLUMN

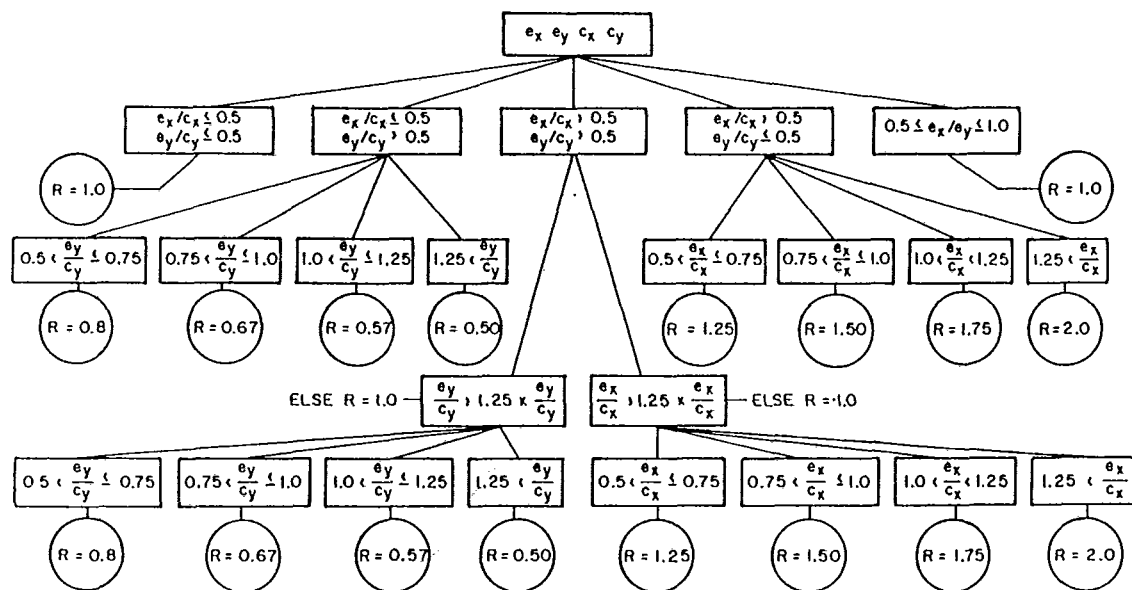


Fig. 2--Rules for selection of column aspect ratio

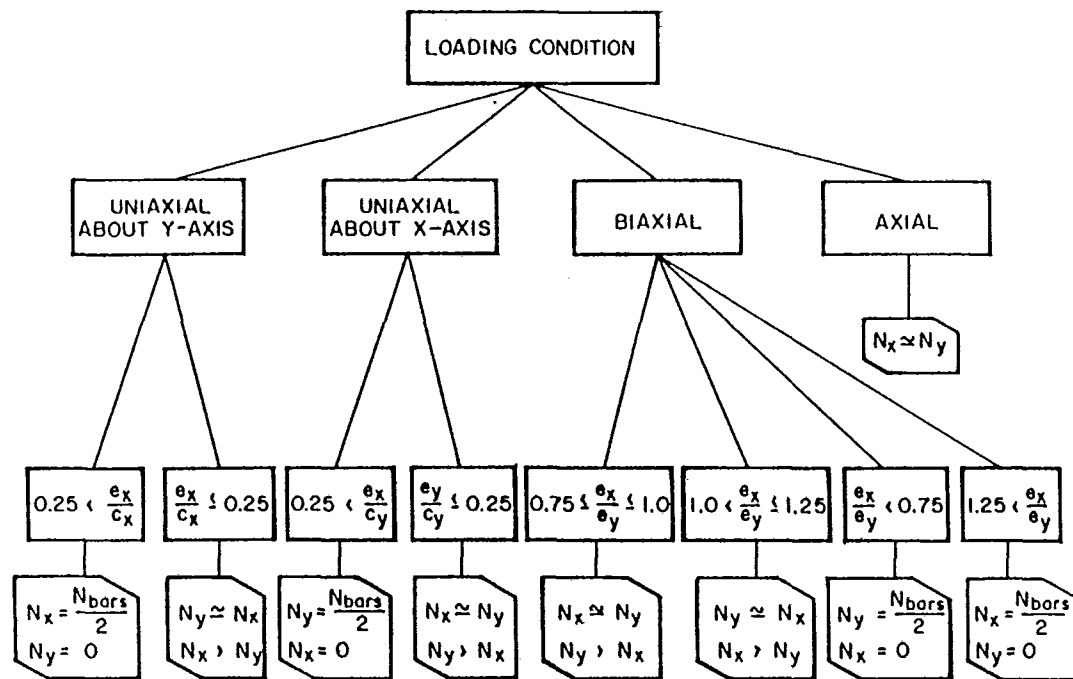


Fig. 3--Rules for selecting reinforcement pattern

Column Dimension		Number of Bars in Face
in.	mm	
8 - 14	200 - 350	2 - 3
16 - 22	400 - 550	3 - 4
24 - 30	600 - 750	4 - 5
32 - 38	800 - 950	5 - 6
40 - 46	1000 - 1150	6 - 7
48 - 54	1200 - 1350	7 - 8
etc.		

Fig. 4--Number of bars in each column face