This is to certify that the

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MICROCOMPUTER AIDED DESIGN FOR
REINFORCED CONCRETE FRAMES
SUBJECTED TO SEISMIC LOADS

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MICROCOMPUTER AIDED DESIGN FOR REINFORCED CONCRETE FRAMES SUBJECTED TO SEISMIC LOADS

by

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ABSTRACT

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A major purpose of the research is to make available, through microcomputers, for the design of "ordinary" R/C structures a highly sophisticated analysis. The latter has been limited to "special" structures because of economic reasons.

The study presents a design procedure for reinforced concrete frames based on nonlinear dynamic analysis.
The design procedure consists of four basic steps:
1) preliminary design based on member forces obtained by linear elastic analysis with equivalent static loading as prescribed by design codes, for example, the UBC code;
2) nonlinear dynamic analysis;
3) checking of damage of structure;
4) redesign of the structure if necessary.

The establishment of this design procedure required the development of the following:
a) a microcomputer program for the nonlinear dynamic analysis of R/C frames subjected to strong earthquakes;

b) design aids, if the redesign of the structure is necessary. The first group of design aids consists of charts for steel reinforcements as functions of curvature ductility. A second group of design aids provides some guidance to choose curvature ductilities of members for selected structural displacement ductility.

The software was designed to have:

a) file-oriented input data;

b) nonlinear dynamic analysis;

c) capability of displaying plots of:

-undeformed structural geometry;

-ground acceleration input;

-nodal displacement;

-deformed structure;

-finite plastic regions and damage ratios.

To my wife, Vania and my son, Lucas.

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TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

FIGURE		PAGE
4.6	Curvature Ductility versus Steel Percentages f'_c = 5 ksi ; f_y = 40 ksi ; $P/(f'_c bh)$ = 0.00	51
4.7	Curvature Ductility versus Steel Percentages f'_c = 5 ksi ; f_y = 40 ksi ; $P/(f'_c bh)$ = 0.10	52
4.8	Curvature Ductility versus Steel Percentages f'_c = 5 ksi ; f_y = 40 ksi ; $P/(f'_c bh)$ = 0.20	53
4.9	Curvature Ductility versus Steel Percentages f'_c = 5 ksi ; f_y = 40 ksi ; $P/(f'_c bh)$ = 0.30	54
4.10	Curvature Ductility versus Steel Percentages f'_c = 5 ksi ; f_y = 40 ksi ; $P/(f'_c bh)$ = 0.40	55
4.11	Curvature Ductility versus Steel Percentages f'_c = 5 ksi ; f_y = 60 ksi ; $P/(f'_c bh)$ = 0.00	56
4.12	Curvature Ductility versus Steel Percentages f'_c = 3 ksi ; f_y = 60 ksi ; $P/(f'_c bh)$ = 0.10	57
4.13	Curvature Ductility versus Steel Percentages f'_c = 3 ksi ; f_y = 60 ksi ; $P/(f'_c bh)$ = 0.20	58
4.14	Curvature Ductility versus Steel Percentages f'_c = 3 ksi ; f_y = 60 ksi ; $P/(f'_c bh)$ = 0.30	59
4.15	Curvature Ductility versus Steel Percentages f'_c = 3 ksi ; f_y = 60 ksi ; $P/(f'_c bh)$ = 0.40	60
4.16	Curvature Ductility versus Steel Percentages f'_c = 3 ksi ; f_y = 40 ksi ; $P/(f'_c bh)$ = 0.00	61

FIGURE		PAGE

CHAPTER 1

INTRODUCTION

1.1 Object and Scope

The response of reinforced concrete building frames subjected to strong earthquake motion is a complex phenomemon. Its analysis is particularly difficult when it is necessary to consider the nonlinear behavior of materials associated with hysteretic loops of reinforced concrete behavior under cyclic loading.

As will be discussed later, satisfactory models and computer programs such as DRAIN-2D [24] and NACRS-II [28], are available for the analysis. However these programs were developed to run on large computers and are rarely employed in ordinary building design practice for several reasons. Generally they are too costly to use. The procedure for data input are often rigid and inconvenient. Finally, the design engineer often lacks the background in the specialties of structural dynamics to interpret the results.
Therefore, their use generally has been limited to research, and in recent years to special structures such as nuclear

power plant structures where sufficient resources are available for use of such specialized programs.

For ordinary buildings, which represent thousands and thousands of plane frames, the design has been based on only the so-called equivalent static seismic loading as prescribed by building codes. More realistic dynamic analysis is not carried out for reasons mentioned. Yet, these "ordinary" buildings are also important because their failure would also spell disaster. During the last earthquake that occurred in Mexico City (1985) [17], 285 multistory buildings collapsed, 143 of which were made of reinforced concrete. Approximately 10000 people died.

It would seem highly desirable to extend the availability of the special tools for dynamic analysis to ordinary reinforced concrete frames. With the rapid advances in microcomputer technology this seems a distinct possibility. The main objective of this thesis is to bring the sophisticated method of analysis to the design process of the "everyday structure". This is done in the context of a proposed design procedure for reinforced concrete frames subjected to strong earthquakes. This design procedure is illustrated in a flow-chart, Fig. (1.1). It essentially introduces a "proof test" or damage assessment process in the design iteration loop. In order to discuss the design process there is a need to consider the background for seismic design which is presented in the next section.

The design procedure itself is described in detail in Chapter 2 and illustrated by a numerical example in Chapter 5.

In Chapter 3, a microcomputer program for the nonlinear dynamic analysis of reinforced concrete frames subject to strong earthquakes is presented. The software was designed to have:

a) file-oriented input data;

b) nonlinear dynamic analysis;

c) capability of plotting two dimensional views displaying any of the following :

- -undeformed structural geometry;
- -ground acceleration input;
- -nodal displacement;
- -deformed structure;
- -finite plastic regions.

The main portion of the software ((b) nonlinear dynamic analysis) is essentially a translation of the mainframe computer program NACRS-II [28] to run on microcomputers. The mathematical model for the nonlinear analysis of plane reinforced concrete members is given in Appendix A. Some features have been added for design use. The most important are the interactive data input mode and plotting capability. Damage factor and ductility ratio have been incorporated in the output and plotting options. They are to be used in the decision whether a redesign of the structure is necessary. An example of the use of the program is given in Appendix B.

The translation from a program for mainframe computers to one for microcomputers requires the execution of the following two tasks:

a) Re-compilation of the source program, NACRS-II.

b) Re-design of the core of the program to be compatible with the interactive mode of input data and the plot programs.

The translation is a challenging task, in the sense that it requires a sufficient background on the subject of earthquake engineering in order to analyze all the subroutines that are parts of the program but also familiarity with the software and hardware of mainframe computers and microcomputers (FORTRAN compilers for mainframe computers are different from FORTRAN compilers for microcomputers). Because of the complexity of the topic and the size of the program these tasks were very time consuming.

In order to facilitate the redesign process, certain graphs are presented in Chapter 4 as design aids. As alluded to previously, a numerical example of the design procedure is presented in Chapter 5. Chapter 6, presents a summary and conclusions of this study.

1.2 Background for Seismic Building Design

1.2.1 Design Philosophy

During an earthquake structures are set into a complex motion in response to the ground motion. In structural engineering terms, the structure may be regarded as being subjected to inertia loads which vary with time.
For economical reasons, both in cost of computing and of training of personnel, the equivalent load (in lieu of dynamic load) approach has been adopted by building codes. For structural modeling, the assumptions of linear elastic behavior is usually made.

Regarding the representation of earthquake ground motions, seismic design of buildings is generally based on inputs reflecting a range of possible earthquake ground motions. The response spectrum method is the most widely used approach for representing earthquake excitations in dynamic analysis. The generation of a response spectrum curve can be carried out by subjecting a series of damped single degree-of-freedom mass-spring systems with varying natural periods to a given ground excitation. It is appropriate to consider a number of response spectra in order to arrive at a design spectrum. Many design spectra are presented at a 1-g level. Once a design spectra has been chosen, several factors must be considered in determining the level of earthquake excitation that a structure is to be designed to resist.

Generally, the objectives of earthquake-resistant design are as follows:

a) to insure that no significant structural damage would result from a moderate earthquake having a reasonable likehood of not being exceeded during the life of the structure;

b) to insure against collapse or major structural failure for a severe earthquake, with even lower probability of occurrence.

The severe earthquake is often called "maximum credible earthquake" and the moderate level motion is referred to as "maximum probable earthquake". The maximum credible earthquake spectrum reflects an envelope of response values that have a 85-95 % probability of not being exceeded during the lifetime of the structure. The maximum probable earthquake spectrum has a 50-60 % probability of not being exceeded. Design codes for buildings aim to incorporate the above two-level design philosophy in their provisions. The first design objective can be met by sizing the structural members such that their yield strengths will not be exceeded in a moderate earthquake. At this design level the behavior is essentially elastic. The building should be proportioned to resist this intensity of ground motion without discernible or measurable damage to the basic structure.

In meeting the second design objective, because this level of earthquake is unlikely to occur within the life of the structure, the designer is justified in permitting it to cause structural damage; however, collapse and loss of life must be avoided. Thus, the inelastic deformation capacity of the structure need be considered. At this design level, adequate energy absorption in the form of ductility must be provided for. Such information may be obtained only through a nonlinear dynamic analysis.

1.2.2 Models for Nonlinear Dynamic Analysis

A considerable amount of work has been carried out on the development of analytical models and numerical methods for the analysis of the behavior of buildings during strong earthquakes. The review in this section is limited to those works which are related to nonlinear dynamic analysis with special emphasis on the inelastic analysis of plane reinforced concrete frame structures subject to earthquake motions. The studies will be cited in two sections; simple models and complex models.

Among the earlier work was the shear beam representation of structures (simple models). The stiffness of each story was assigned to a shear spring which included nonlinear deformations. Aziz [6] used this shear-beam model in the study of a ten-story frame. A modified shear-beam model was introduced by Aoyama [1] for reinforced concrete structures.

Tansikirikongkol and Pecknold [40] used a bilinear shear model for approximate modal analysis of structures.

While the basic element of the simple shear beam model is a story of a frame, the basic element of a complex model is a frame member, that is, a column or a beam. In a complex model the choice of the model to represent a structural member is a crucial one in terms of computational effort and ease of formulating stiffness changes. A number of empirical models have been proposed in the past to represent the hysteretic behavior of reinforced concrete members:

Simple bilinear formulations models, Fig. (1.2) were the first ones employed by researchers. Heidebrecht, et al (1963, 1964) [20] developed a numerical method to analyze plane frame structures with elasto-plastic members under dynamic excitations. During the Third World Conference of Earthquake Engineering in New Zealand, two mathematical models were presented for the inelastic analysis of frame structures. In the first model by Wen and Janssen (1965) [41], a frame member was divided into short segments along its length. A bilinear moment-curvature relationship was assigned to each short segment, (Fig. (1.3)). The application of this model was limited to simple structures because a large size computer memory was required for the analysis. In the second model by Clough, Benuska and Wilson (1965) [12], a frame member was divided into two imaginary parallel elements: an elasto-plastic element to represent a yielding characteristic, and fully elastic element to represent strain hardening behavior.

An equivalent spring model was proposed by Giberson (1967) [18]; the model consisted of a linearly elastic member with two equivalent nonlinear springs at the member ends (Fig. (1.4)). Due to its relative simplicity, this model attracted considerable attention.

It was realized that the behavior of a reinforced concrete member under cyclic loading is more complicated than the above described bilinear models. The concept of a degrading bilinear model was introduced by Clough [13] and represents an improvement over the simple bilinear formulations, Fig. (1.5). Takeda [39] examined the experimental results for cyclic loading of a series of reinforced concrete members, and proposed a general trilinear model, which was in substantial agreement with the test results. This model is known as the "Takeda Model" and has been adopted by a number of investigators. The degrading stiffness model and the Takeda models have been included in the general inelastic dynamic analysis programs DRAIN-2D[24] and SAKE [31].

The original Takeda model did not include the so-called "pinching effects", which represents the effect of shear stress on the hysteresis behavior. Meyer and Roufaiel [28] used a modified Takeda-type model (see Fig. A.5 of Appendix A) that includes this consideration. Furthermore, none of the above models considered the effect of the finite size of the plastic regions. All of them were based on the spring model. Ih the model proposed by Meyer and Roufaiel the finite size of the plastic regions is taken into account (see Fig. A. 7 of Appendix A).

The accuracy of the model was demonstrated in [28] by analyzing numerous members and structures for which test results are available in the literature, leading to the conclusion that the model is effective in predicting the nonlinear behavior of reinforced concrete members in plane frames. Based on this mathematical model, the computer program NACRS-II was developed using a modification of the member stiffness portion of the general-purpose nonlinear analysis computer program, DRAIN-2D. This study has, in turn, adopted NACRS-II for the analysis part of the computer software.

1.2.3 Ductility

It is commonly accepted that it would be economically unrealistic to design a structure with sufficient strength to remain elastic during strong earthquakes, and inelastic behavior should be allowed to occur. To avoid collapse during a major earthquake, members must be ductile enough to absorb and dissipate energy by post-elastic deformations. A key parameter in the characterization of inelastic behavior is "ductility ratio". The "response ductility" of a member is the ratio of the maximum deformation to the deformation at first yield of the response history of a structure. The "design ductility" is the ductility attainable by the structure designed. The definition of ductility ratio may be applied at three levels:

a) structure or displacement ductility;

b) curvature ductility;

c) member or rotation ductility.

The displacement response ductility factor, μ_D, is defined as the ratio of Δ_m, the lateral displacement at the top of the structure at the end of the post-elastic range, to Δ_y, the lateral deflection at the same elevation when yield is first reached. The Commentary on the SEAOC code [38] indicates that the displacement ductility response factor required in design would range from 2 to 5. Since ductility in this range is associated with plastic deformations, permanent structural damage is expected.

The ductility of reinforced concrete sections, μ_ϕ, can be expressed by the curvature ductility ratio, ϕ_x/ϕ_y, where ϕ_x is the curvature at the end of the post-elastic range and ϕ_y is the curvature at first yield. For given section properties this curvature relationship can be derived using conventional reinforced concrete theory [33]. Data for such relationship are presented in Chapter 4 for some usual combinations of material properties.

The member or rotation ductility is defined as the ratio of the maximum end rotation to the end rotation at first yield. Since this ductility is associated with the spring models, it will not be used in this study.

In nonlinear response of multistory structures, the relationship between curvature ductility of the members and displacement ductility response of the structure is very complex. In Chapter 4, this relationship is calculated for one-bay frames from 3 to 10 stories, for a particular ground acceleration input. The data is intended to be used as a reference guide for redesign.

1.2.4 Damage Factor

In the context of design that allows inelastic response, the degree of "damage" needs to be defined. In past studies, it has generally been treated subjectively. Damage can be looked at from an economical viewpoint, using a scale from zero for no damage to a maximum of unity, representing total failure of the member.

Wiggins and Moran [42] have proposed to grade existing buildings on a scale from 0 to 180 points to measure its structural reliability. Culver et al [14] have proposed another field evaluation method but its use has been limited because of the subjective nature of the judgement required. Shibata and Sozen [17] have introduced a damage parameter, to be used for design, as the ratio between the initial stiffness and the reduced secant stiffness at maximum deformation. This damage ratio can be easily computed when the moment-curvature relationship is determined. But as damage indicator this factor is of limited use because "failure" is not defined.

Meyer and Roufaiel [28] introduced a modified damage factor described in Appendix A. This damage factor represents the ratio between the secant stiffness at the onset failure and the minimum secant stiffness during the response. The value of zero, indicates no damage. The value of 1.0 indicates a total failure of the member. This damage factor is defined as (see also Fig. A. 8 of Appendix A):

$$\text{damage factor} = \frac{M_m}{M_x} \cdot \frac{\phi_x}{\phi_m} \qquad \text{for } M_x > M_y$$

$$\text{damage factor} = 0 \qquad \text{for } M_x < M_y$$

where M_y is the yield moment of the section of the member and M_x is the maximum moment that has been attained at the section; and M_m is the ultimate moment capacity of the section as described in Chapter 4.

According to a recommendation from [28], the damage factor may be used to classify damage as follows:

<u>Damage Factor</u>	<u>Damage Class</u>
0.00 to 0.25	low damage;
0.25 to 0.50	moderate damage;
0.50 or greater	high damage(redesign the member);

This classification will be used herein to decide whether the damage level of a member is acceptable when responding to a presumed maximum credible earthquake motion.

1.3 Notation

The following notation was used in this work:

b = section width of the member,

d' = section depth of the member,

f'_c = uniaxial strength of plain(unconfined) concrete,

f_y = steel yield stress,

f_{sy} = f_y,

M_m = ultimate moment capacity of the section,

M_x = maximum moment that has been attained at the section,

M_y = yield moment of the section,

Δ_m = lateral displacement at the top of the structure at the end of the post-elastic range,

Δ_y = lateral deflection at the same elevation when yield is first reached,

ϵ_o = strain at f'_c,

ϵ_m = concrete crushing strain,

μ_D = displacement response ductility factor,

μ_ϕ = ductility of reinforced concrete sections,

ρ = longitudinal steel percentage,

ρ'' = confinement steel percentage,

ϕ_m = ultimate curvature of the section,

ϕ_x = curvature at the end of the post-elastic range,

ϕ_y = curvature at first yield.

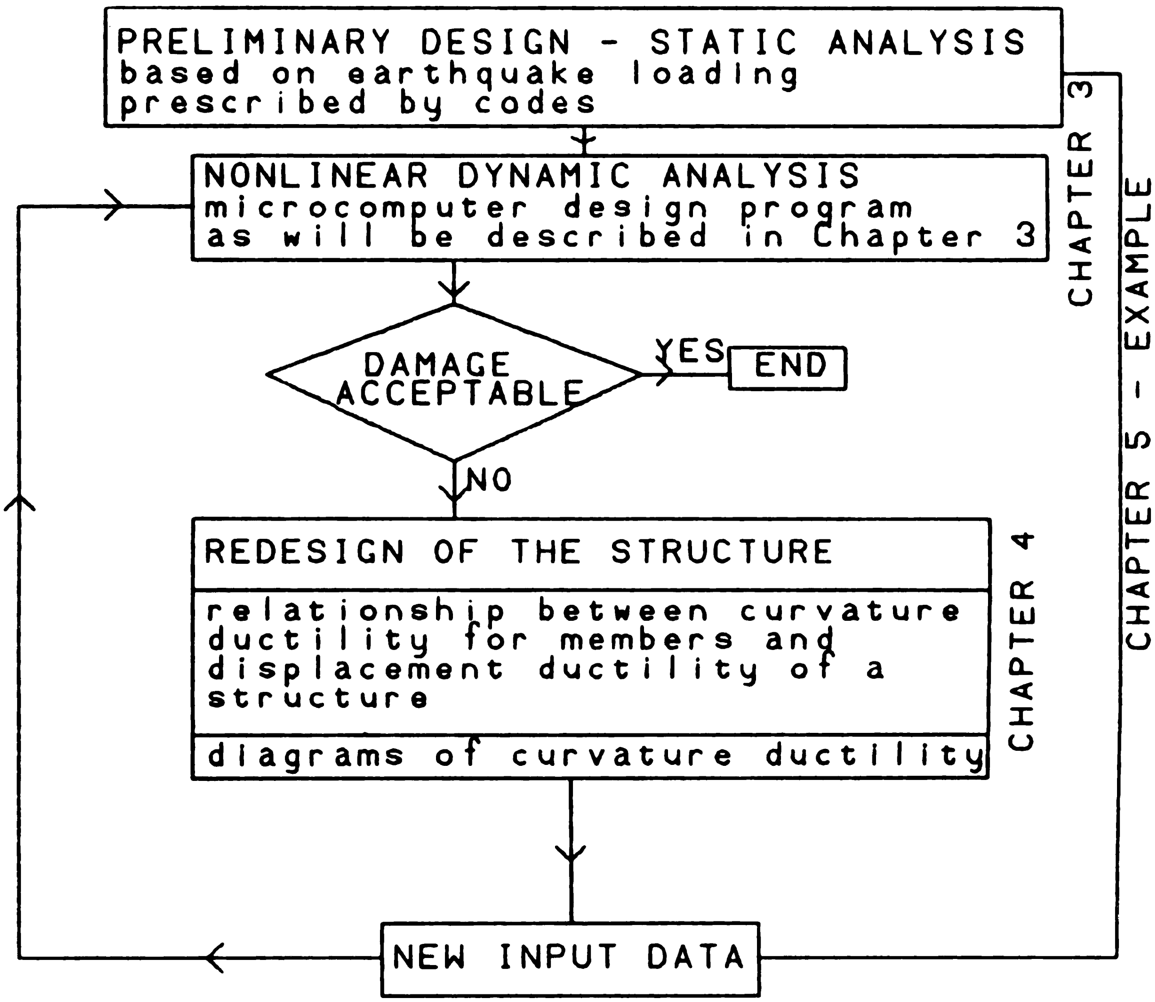

Fig.(1.1) - Flow-chart - Design Procedure

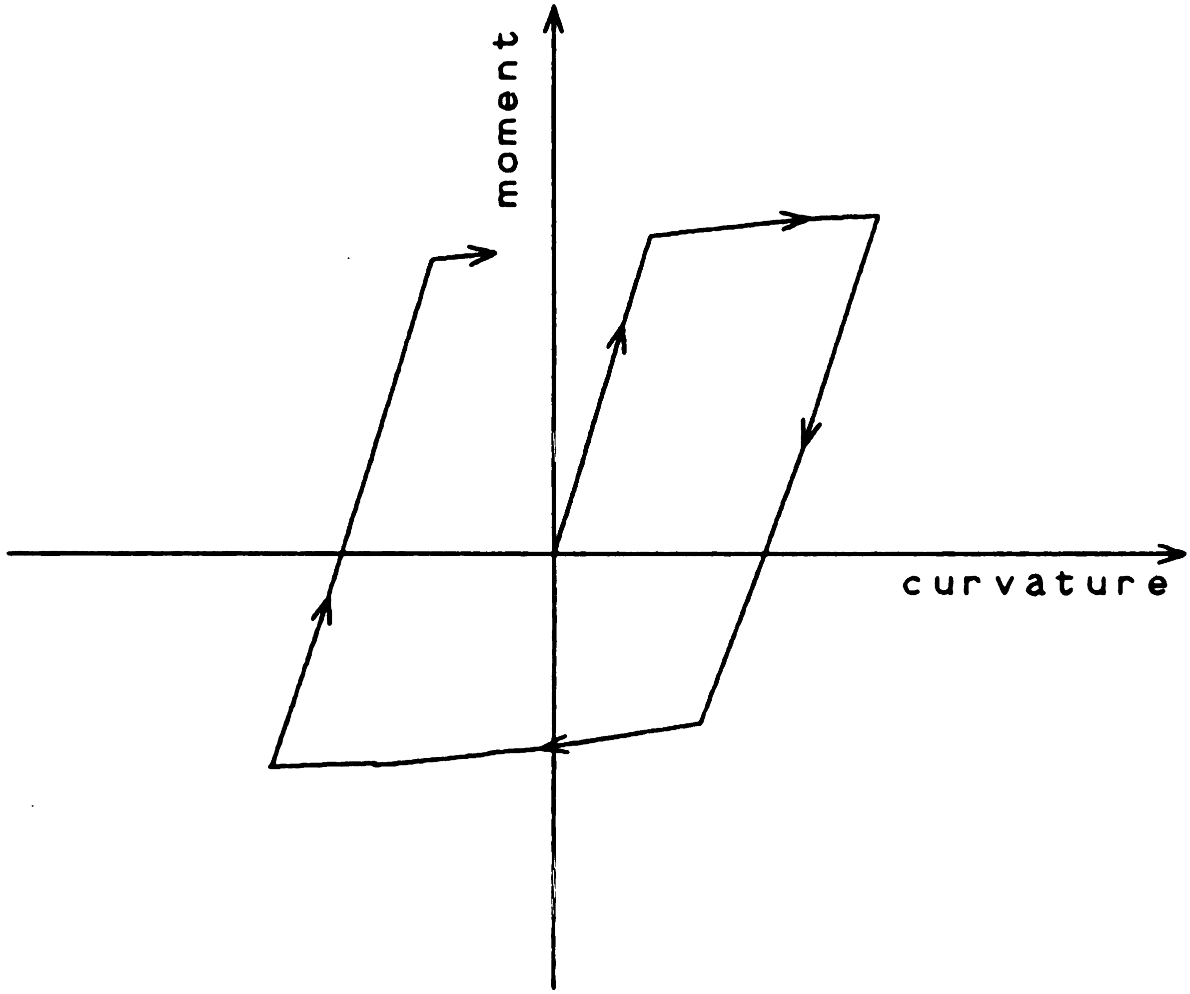

Fig.(1.2)-Bilinear Hysteretic Model

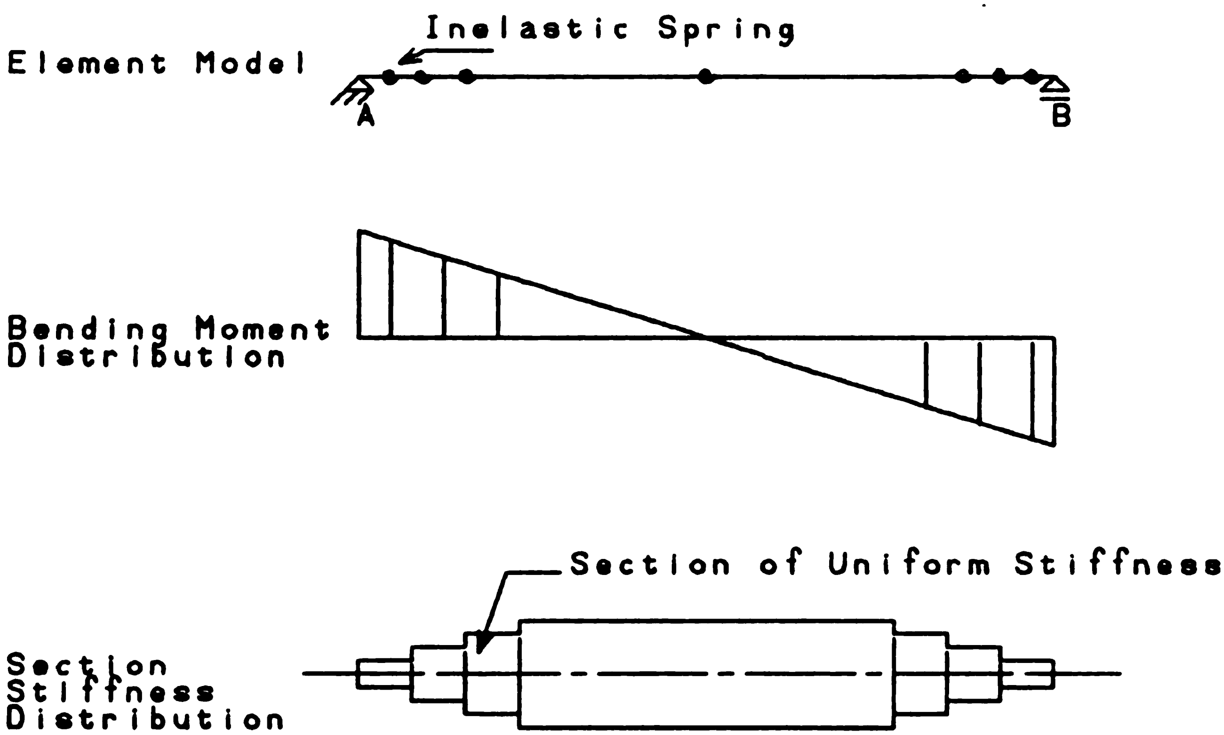

Fig.(1.3) - Frame Member Model

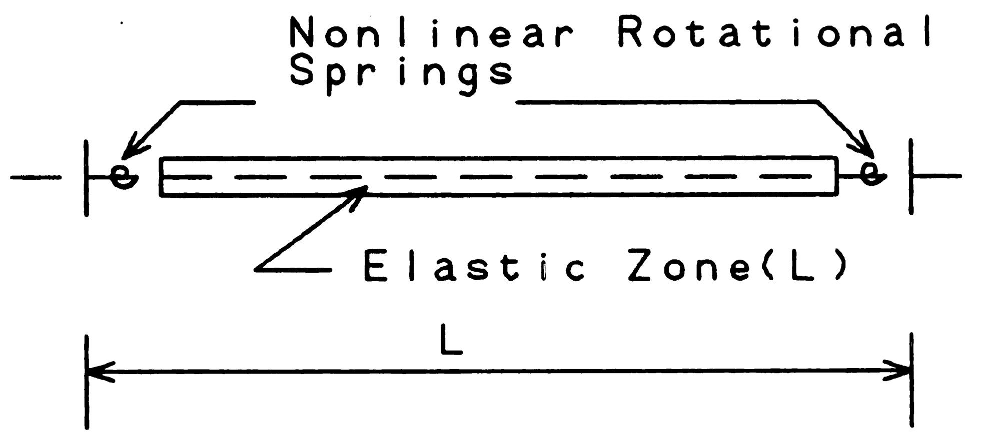

Fig.(1.4)- Equivalent Spring Model

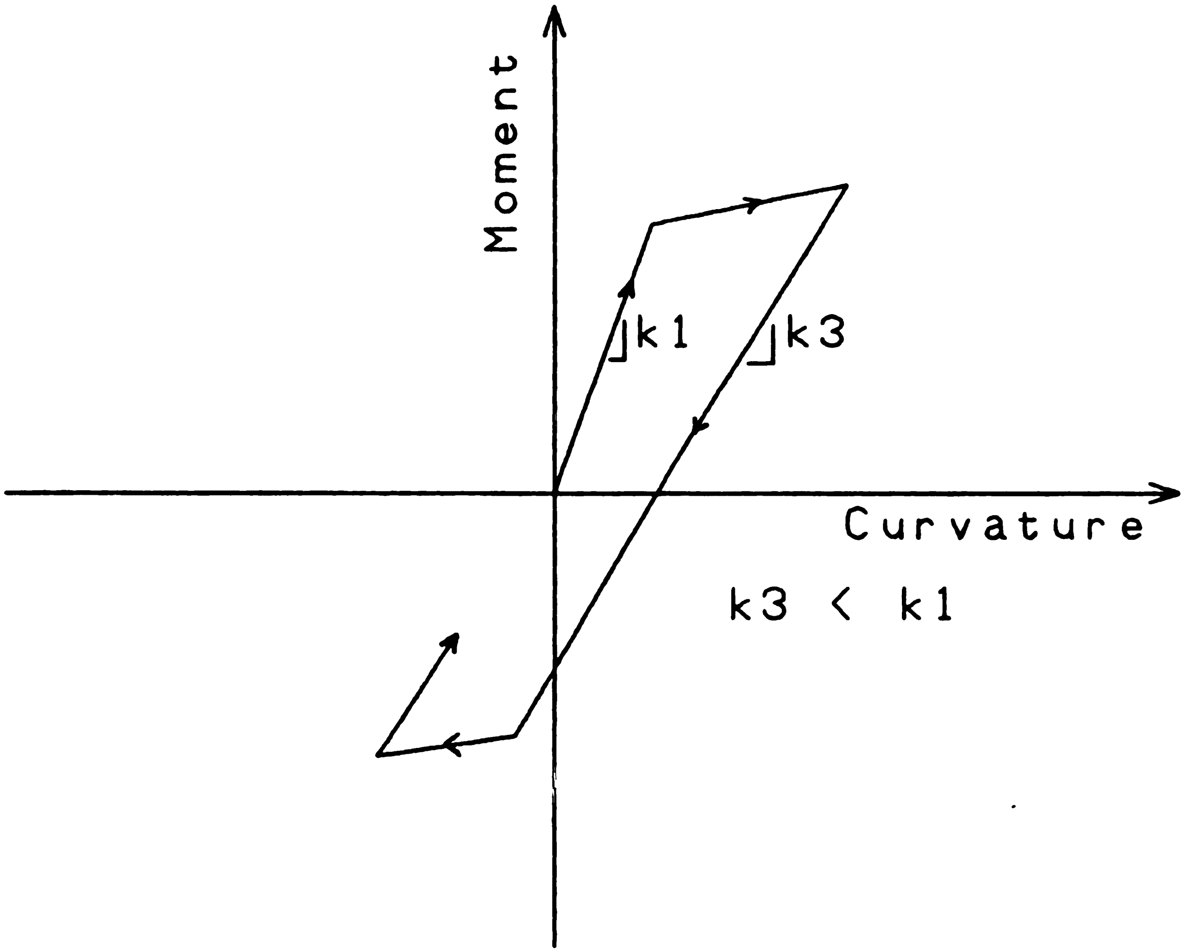

Fig.(1.5)- Bilinear Degrading Hysteretic Model

CHAPTER 2

A DESIGN PROCEDURE

The objective of this chapter is to present a design procedure for reinforced concrete frames subjected to seismic loads. As illustrated in Fig. (1.1), the procedure consists of four basic steps:

a) preliminary design based on member forces obtained by a linear elastic analysis for equivalent static loading as prescribed by design codes, such as the UBC[21];

b) nonlinear dynamic analysis, using the microcomputer program presented in Chapter 3;

c) checking of damage of the structure;

d) redesign of the structure if necessary.

The complete design procedure is illustrated by a numerical example in Chapter 5.

2.1 Preliminary Design

The design seismic loading recommended by building codes, for example the UBC Code is in the form of statically applied loading. It is presumed that these static loadingswould be equivalent in their effects on the structure to the earthquake motions. However, dynamic analysis of structureshave shown that the response inertia loads may be much greater than the static lateral loads recommended by such codes. It is well known that structures designed based on loads prescribed by such codes have survived substantial earthquakes. This has been attributed to the ability of ductile structures to dissipate energy by post-elastic deformations, damping, and soil-structure interation. In recent years, special detailing practices for seismic loads(see for example, Appendix A of ACI Building Code 318-83) have been codified to enhance the performance of the structure.

Therefore, the first phase of the proposed design procedure is to use the usual design process a current practice such as the UBC code. It is also presumed that minimum detailing requirements for seismic design in the ACI Code are also followed.

2.2 Nonlinear Dynamic Analysis

This phase essentially represents a "proof test" of the structure obtained in the first phase. A dynamic analysis would be carried out by use of the microcomputer program MC-QUAKE to be described in Chapter 3. A major decision for this phase is the choice of earthquake (recorded or artificially generated) to be applied to the structure. General considerations for this choice are briefly presented in Section 2.4. For the purpose of this study, it is presumed a choice has been made. It may be, for example, the E-W component of the 1940 El Centro earthquake with the amplitudes amplified by a factor of 1.2.

The analysis consists of three phases a) input data, b) computations , c) output of the results.

The initial sizing of structural members determined in the preliminary design described before is used in the first run of the analysis. The consideration of inelastic behavior requires input of additional parameters over those needed for regular linear elastic analysis. The input data include:

a) material properties:

a.1) concrete: concrete strength, f'_c, strain at maximum stress, ϵ_o, confinement ratio, ρ'' (see Equation A.3);

Although the confinement ratio ρ'', as precisely defined in Equation A.3, is a function of the confinement steel, it affects the stress-strain relationship of concrete. Therefore, it is considered a concrete parameter.

a.2) steel: modulus of elasticity, E_s, strain hardening ratio, p_s, yield stress, f_y, and ultimate strain, ϵ_s;

b) member properties: magnitude of axial load, P, longitudinal reinforcement, ρ , and transversal reinforcement.

The following assumptions are made for the nonlinear dynamic analysis:

a) The analysis is limited to plane framed structures.

b) Every member in the structure may be treated as a massless line member, that is, the conventional beam theory applies.

c) All mass at a floor level including rotary inertia is concentrated at the joints.

d) The structure is fixed to an infinitely rigid foundation.

e) The inelastic deformation of each constituent member follows the modified Takeda's hysteresis model.

f) The ground motion takes place in the plane of the structure and in the horizontal direction only.

g) The deformations are small and the geometry of the structure retains the initial configuration. However, the effects of gravity loads (the P- Δ effects) are considered.

h) Axial load remains constant and equal to the gravity load effect present at the beginning of the ground motion.

The output results include the following:

a) maximum horizontal displacement at the top of the frame;

b) displacement ductility factor μ_D;

and for each end of every member:

c) finite size of the plastic regions;

d) damage factor;

e) maximum moment.

2.3 Redesigning of the Structure

The structure will be modified or redesigned if the damage factor of one or both ends of one or more members attains a certain prescribed level, for example, 0.5. The new design may be guided by the design aids presented in Chapter 4 in a manner described therein.

2.4 Selection of Design Earthquake

In order to establish the ground motion characteristics of the maximum probable earthquake for any given building site, it is necessary first to study the earthquake history of the region. It may be necessary to analyze seismological parameters, such as length of faults, fault distance from the site, etc, in order to estimate the intensity of the earthquake expected at the site. Such study could provide estimates for the maximum acceleration expected at the site concerned, predominant period for maximum acceleration and duration of shaking.

One approach to obtaining an appropriate ground motion record has been to modify and distort an actual earthquake record so that it represents an event of different magnitude and distance. The intensity of it may be adjusted by a amplitude-scaling factor, the frequency content may be modified by a change of time scale, and the duration of the earthquake may be changed by truncating or duplicating portions of the record.

It is more desirable to have a statistically significant sample of earthquake motions appropriate to the given magnitude and distance, rather than one. It is also quite feasible to derive artificial earthquake motions which represent the desired design earthquake. These ground motions are generated by random process that may be repeated to provide the samples needed for the desirable input motion.

It is noted again, however, that for the purpose of this study, the designer is presumed to have been provided with this input ground record.

CHAPTER 3

COMPUTER PROGRAM

A computer program (code name : MC-QUAKE) for the nonlinear dynamic analysis of plane reinforced concrete frames subject to strong motion earthquakes is presented. The program is written using FORTRAN 77 and has been developed to run on microcomputers.

In order to make the computer program more convenient to use the input and output data were handled in a file-oriented format, as illustrated in the flow-chart , Fig. (3.1).

MC-QUAKE is composed of three sub-programs:

a) Sub-program DATA - for input data;

b) Sub-Program MAIN - the main program;

c) Sub-Program PLOT - for plotting input data and output results

A complete example illustrating the use of MC-QUAKE is presented in Appendix B.

3.1 Sub-program DATA

Even for a relatively small structure, a nonlinear dynamic analysis requires a set of data of almost a hundred items. In order to handle these large data sets, DATA was designed to be a file-oriented program, in the sense that data can be created, saved, retrieved, edited and listed.

DATA creates the input data file for subsequent use in Sub-program MAIN. The input is carried out in an interactive mode with the user responding to prompts. When one selects DATA (by typing "DATA") the following sub-system menu appears as prompts:

0 : NEW DATA

1 : CORRECT DATA

2 : LIST DATA

3 : END>

The system waits for a number corresponding to the desired option. For example, if one wishes to enter new data, respond with 0, etc.

a) NEW DATA -- before data can be entered into the system, a data file must be opened to accept data. When a data file is opened for the first time, the user specifies the name of the data file. When the data file is first opened, all data values are initially equal to zero. The system automatically prompts for the information needed.

b) CORRECT DATA -- to change data values from existing input data files, select the CORRECT DATA option of the DATA menu. Once this option is selected, the user specifies the variable to be corrected, followed by the new data value.

c) LIST -- the data file may be displayed on the screen and/or have them printed out as a hard copy.

The input data for the analysis of a structure can be grouped into the following sets;

1) Control Information;

2) Control Joint Coordinates;

3) Node Generation Data;

4) Boundary Condition Code;

5) Mass Generation Data;

6) Load Control Data;

7) Earthquake Acceleration Control-Damping Factors;

8) Output Control;

9) Beam Element Control;

10) Reinforcement Steel Properties;

11) Concrete Properties;

12) Member Properties.

A set of data within each group is needed for the overall analysis.

3.1.1 Control Information

To supply the "control information", responses to the following prompts are needed.

- number of nodal points;
- number of control nodes for x and y coordinates generation (number of nodes to be used as input data);
- number of sets of nodes to be generated ;
- number of sets of boundary condition code to be generated;
- number of sets of mass data to be generated;
- data checking or run;
  - 0 : run
  - 1 : check only.

3.1.2 Control Joint Coordinates

The prompts are:

- node number;
- x-coordinate;
- y-coordinate.

3.1.3 Node Generation Data

The prompts are:

- first node number;
- last node number;

- number of nodes to be generated;
- difference between two successive nodes.

3.1.4 Boundary Condition Code

1 : for fixed D.O.F.

0 : for free D.O.F.

- first node in the group;
- code for x-displacement;
- code for y-displacement;
- code for rotation;
- last node in the group;
- difference between two successive nodes.

For example, if first node = 1, last node = 6 and difference between two successive nodes = 1, all nodes 1, 2, 3, 4 ,5 and 6 will have the same boundary conditions.

3.1.5 Mass Generation Data

- first node number in the group;
- x-mass;
- y-mass;
- rotary mass;
- last node number in the group;
- difference between two successive nodes.

3.1.6 Load Control Data

- gravity acceleration (consistent with the basic units chosen);
- number of integration steps for dynamic analysis;
- integration time step size;
- amplification factor, c_1, to be applied to the amplitudes of ground acceleration ;
- scaling factor, c_2, to be applied to the time coordinate of the ground motion record;
- absolute value of maximum permissible displacement, Δ_F.

Factors c_1 and c_2 are used to modify and distort the earthquake record used so that it may represent an event of different magnitude and distance, as described in Section 2.4. The quantity Δ_F is used to end the computation when the horizontal displacement at the top of the frame becomes too large (signifying failure). It is taken as 0.06 H, where H is the total building height.

3.1.7 Earthquake Acceleration Control-Damping Factors

- number of time-ground acceleration pairs defining the ground motion input;
- code for printing acceleration input;
    0 : do not print acceleration record;

1 : print acceleration record;

- number of steps between printing;
- title to identify record;

1 : 1940 El Centro NS;

2 : Caltec A-1[23];

3 : Caltec A-2;

4 : Caltec B-1;

5 : Caltec B-2;

6 : Caltec C-1;

7 : Taft;

8 : San Fernando 1941;

9 : Other;

- mass proportional damping factor, α;
- tangent stiffness proportional damping factor, β.

3.1.8 Output Control

- number of nodes for x-displacement;
- list of nodes for x-displacement.

3.1.9 Beam Element Control

- number of members in the structure;
- number of reinforcing steel property types;
- number of concrete property types.

3.1.10 Reinforcing Steel Properties

- modulus of elasticity, E_s;
- strain hardening ratio, p_s;
- yield stress, f_y;
- ultimate stress, ϵ_s.

3.1.11 Concrete Properties

- concrete strength, f'_c;
- strain at maximum stress, ϵ_0;
- confinement steel ratio, ρ''.

3.1.12 Member Properties

- member number (negative value for symmetric section so that the last three items of the following data list need not be input);
- node number at end of the member, node I;
- node number at the other end, node J;
- steel type number (if zero, all the information except node number will be the same as the preceding element and the following parameters could be omitted);
- concrete type number;
- magnitude of axial load due to gravity;

- height of the section;
- width of the section at the bottom;
- cover of the section at the bottom;
- reinforcement at the bottom;
- width at the top;
- cover at the top;
- reinforcement at the top.

3.2 Sub-program MAIN

MAIN computes the nonlinear response of reinforced concrete frames subject to dynamic loading following the mathematical model proposed by Meyer and Roufaiel [28] as described in Appendix A. It is a modified version of the program NACRS-II (Nonlinear Analysis of Reinforced Concrete Structures)[28] developed to run in the IBM/370 Computer Systems. The NACRS-II itself is a modified version of the program DRAIN-2D by A. E. Kannan and G. H. Powell[24].
MAIN also prints input and output information, which includes the following:

a) input data information;

b) nodal displacement envelopes;

c) for each frame member of the structure, the inelastic response:

1. curvature ductility provided;
2. plastic length at nodes I and J;

3. damage factor;

4. maximum moment.

d) first yield displacement, Δ_y;

e) maximum horizontal displacement at the top of the structure;

f) displacement ductility , μ_D.

3.3 Sub-program PLOT

PLOT is developed to plot two dimensional views for:

a) the undeformed structure : this is provided as a visual aid to confirm the correctness of input data node coordinates and member incidences;

b) ground acceleration input: plotting this input data is useful to viewing the acceleration record data;

c) displacement at the top of the frame as a function of the time ;

d) the deformed structure: after each time increment the deformed structure could be plotted;

e) the finite plastic regions and the damage factor: this is the most important output in terms of plastic analysis.

Two quantities are presented for each member end the length of the inelastic region and the damage factor. The inelastic length could indicate possible collapse mechanism and the damage factor will indicate how far or close the member is from total failure. A damage factor equal 1.0 is considered as total failure.

3.4 Hardware Requirements

The following is a list of the minimum hardware requirements for running MC-QUAKE:

1. One IBM or IBM compatible microcomputer;
2. 512 kilobytes of random access memory;
3. One 360K floppy disk drive and one 10 megabyte hard disk;
4. A 8087 math coprocessor. (Its use greatly reduces run time, to one third or less of the time required without the 8087.)
5. Graphics Card (for the PLOT Sub-program) - the Hercules Graphics Card is required; the program can be easily adapted for running with another graphic card;
6. One dot matrix printer - program output can be printed on any printer which is compatible with the computer. A hard copy of the screen graphics produced by PLOT may be obtained.

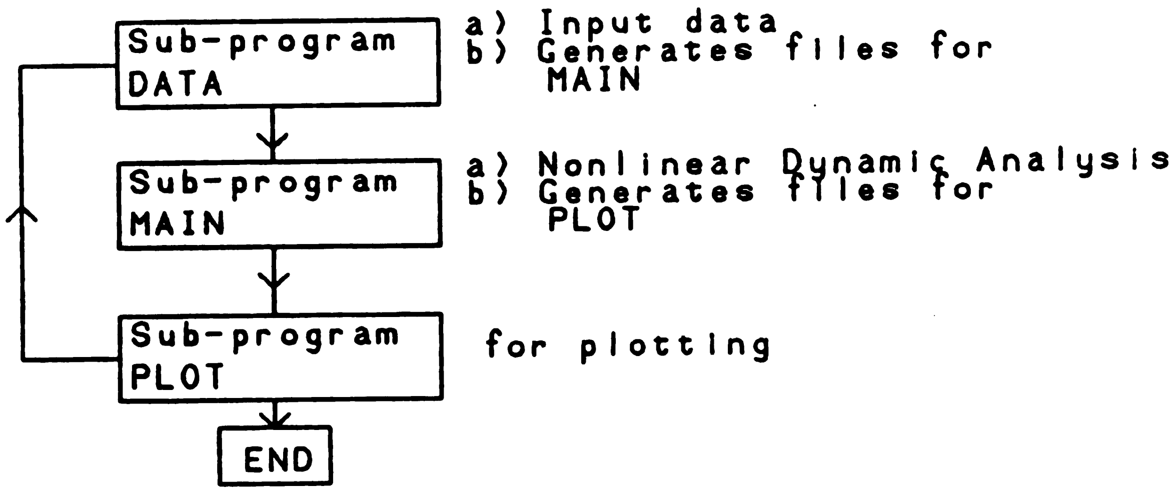

Fig.(3.1) - Flow-chart of MC-QUAKE

CHAPTER 4

DESIGN AIDS

As explained in Chapter 2, if any member of the structure reaches a level of damage, as defined in Chapter 1, with a damage factor greater than 0.5, the structure should be redesigned (although such redesign may involve a modification of only a single member). For example, one may modify one or more of the basic design parameters (b, d, ρ, ρ'', and so forth) for the columns and/or for the beams and then reanalyze and check the structure again. In general this would be "too loose" (too much freedom), and in consequence, take too many trials in order to arrive at an appropriate design.

Design aids are provided herein to give some guidance for the redesign. Instead of dealing with the basic parameters(such as, b, d, ρ, and ρ''.) directly, it is convenient to deal with the maximum curvature ductility factor as an intermediary. First note that there are specific relationships between the basic material parameters and the maximum curvature ductility of a section. The primary moment-curvature relationship of a section can be derived using conventional reinforced concrete theory only.

Details are given, for example, in Park's book[33]. These primary moment-curvature relationships were used to compute the data presented in Fig. (4.1) to Fig. (4.20) for some practical ranges of the parameters.

For a damage factor less than 0.5, the response curvature ductility must be less than the maximum (member section maximum curvature ductility). Obviously, the maximum curvature ductility needed depends on the structure geometry, such as, number of stories, material properties and the response displacement ductility factor and the ground motion. It is impossible to cover all cases.

In order to provide a guide, Figures (4.22) to (4.29) present a relationship between the response displacement ductility and the maximum curvature ductility of a section. Note that each point of the graph represents a design with damage factor generally less than 0.5 (or with negligible exceedance), as indicated in Table (4.1). Thus one may begin the modification by first choosing a response ductility factor, in general ranging from 2 to 5, and go to the appropriate graph for the number of stories in order to determine the maximum curvature ductility for the columns and beams. Then, go to the first group of graphs to choose the design parameters.

4.1 Curvature Ductility Diagrams

The yield curvature ϕ_y is defined as the curvature at which the strain in the steel equals its yield strain. The ultimate curvature ϕ_m is defined as the curvature at which the strain of the concrete cover is equal to ϵ_m. The ratio ϕ_m/ϕ_y is called the curvature ductility, μ_ϕ, of the section.

Curves for μ_ϕ are plotted in Figures (4.1) to (4.20) for a range of practical combinations of f_y and f_c' for normal weight concrete and for $\epsilon_0 = 0.002$. As design aids the diagrams are constructed that give values of curvature ductility as a function of the longitudinal reinforcement percentage and the confinement steel percentage.

It may be noted from Fig. (4.1) to Fig. (4.20), that:

a) when the longitudinal reinforcement increases, the curvature ductility decreases;

b) when the confinement steel increases, the curvature ductility increases;

c) when the steel yield stress , f_y, increases or the uniaxial strength of the concrete, f_c', decreases, the curvature ductility decreases; and

d) when the level of axial load on the section increases, the curvature ductility decreases.

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4.2 Relationship between Displacement Ductility and Curvature Ductility

Some attempts at relating μ_D and μ_ϕ for multistory frames have been made. In general they are based on static collapse mechanisms [33] which would involve such simplified assumptions as :

a) frame reaches yield at all the plastic hinges sections simultaneously;

b) bending moment patterns are obtained from the equivalent static lateral loading recommended by code;

c) this code static loading corresponds predominantly to the first mode of vibration response.

Therefore this static approach can be taken only as a guide. It is the goal here to determine this relationship on the basis of nonlinear dynamic analysis. Framed structures subjected to both gravity and seismic loads will be analyzed.

4.2.1 Structure Types

Reinforced concrete frames of one-bay ranging from 3 to 10 stories were selected. The dimensions are shown in Fig.(4.21), a "typical" one-bay frame.

The following additional data are assumed for the analysis:

a) concrete properties:

f'_c = 3 ksi

ϵ_0 = 0.002

ρ'' = 1.0 % for all members

b) reinforcement steel properties:

f_y = 60 ksi

E_s = 29,000 ksi

p_s = 0.01

ϵ_s = 0.015

c) dead load per floor = 50 kips

4.2.2 Ground Motions

The 1940 El Centro (NS) earthquake is used. From that record a segment of 12 seconds duration with a peak acceleration approximately 0.33 g is applied to the frames.

4.2.3 Response Parameters

The parameters used to characterize the inelastic response were as follows:

a) maximum horizontal displacement at the top;

b) finite size of plastic regions;

c) damage factor;

d) displacement ductility factor μ_D;

e) maximum moment of the member.

4.2.4 Presentation of Results

A series of dynamic analysis (120 cases), using the software presented in Chapter 3, were carried out to find the relationship between curvature ductility for members and the displacement ductility of a structure.
A review of the results for these eight frames indicates that the displacement ductility factor ranges from 1.2 to 4.8 as the curvature ductility of the beam members ranges from 28 to 34 and the column steel percentage ranges from 2% to 6%.

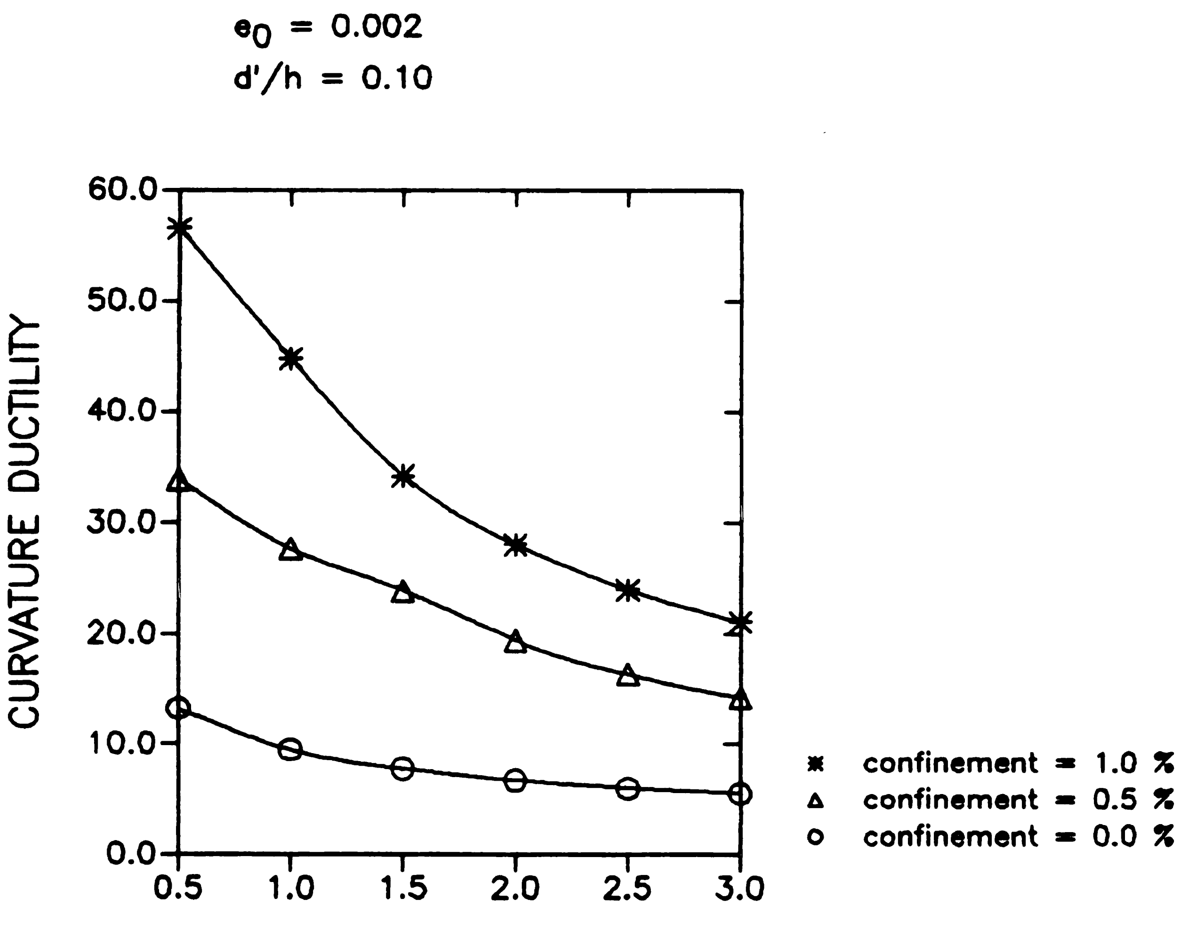


Fig.(4.1)—Curvature Ductility versus Steel Percentages
f'_c= 5 ksi f_y= 60 ksi $P/(f'_c\ b\ h)$= 0.0

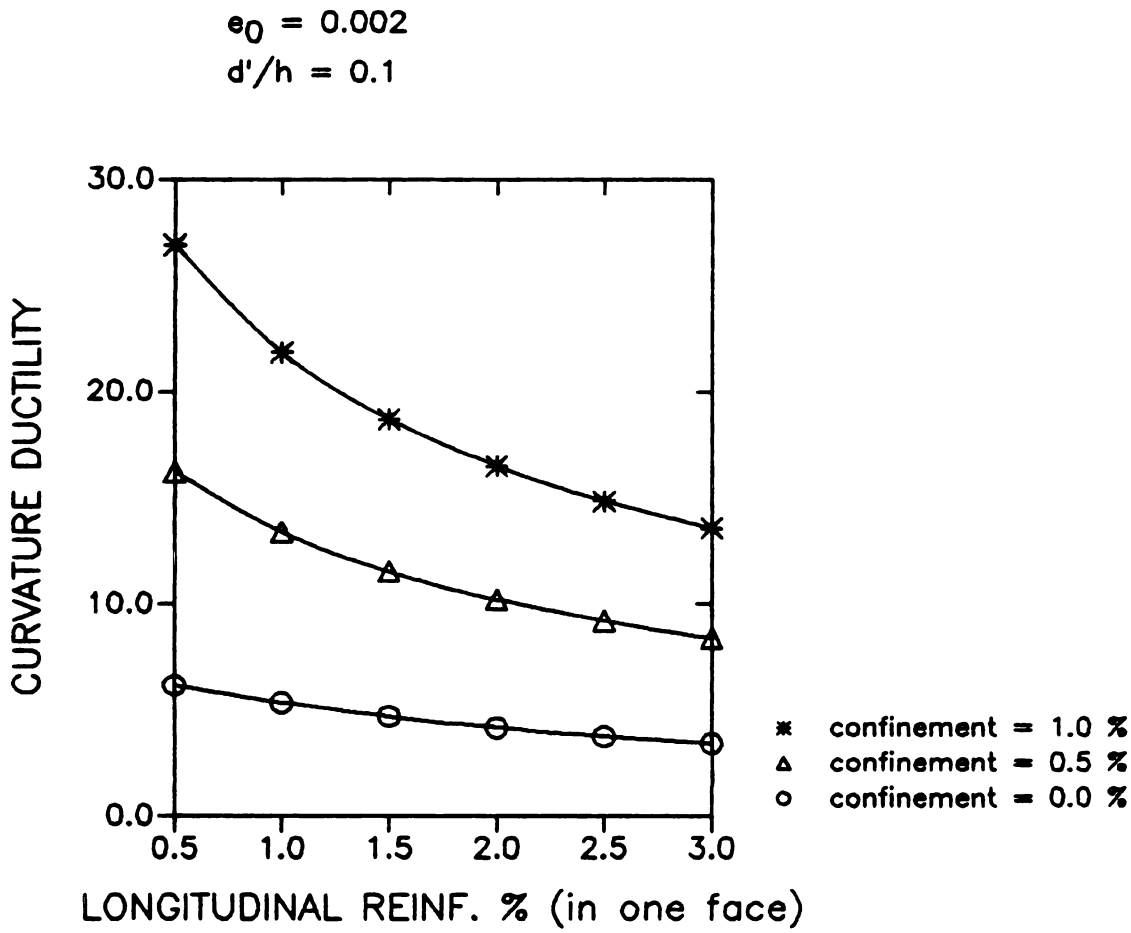


Fig.(4.2)—Curvature Ductility versus Steel Percentages
f'_c= 5ksi f_y= 60 ksi $P/(f'_c b\ h)$ = 0.10

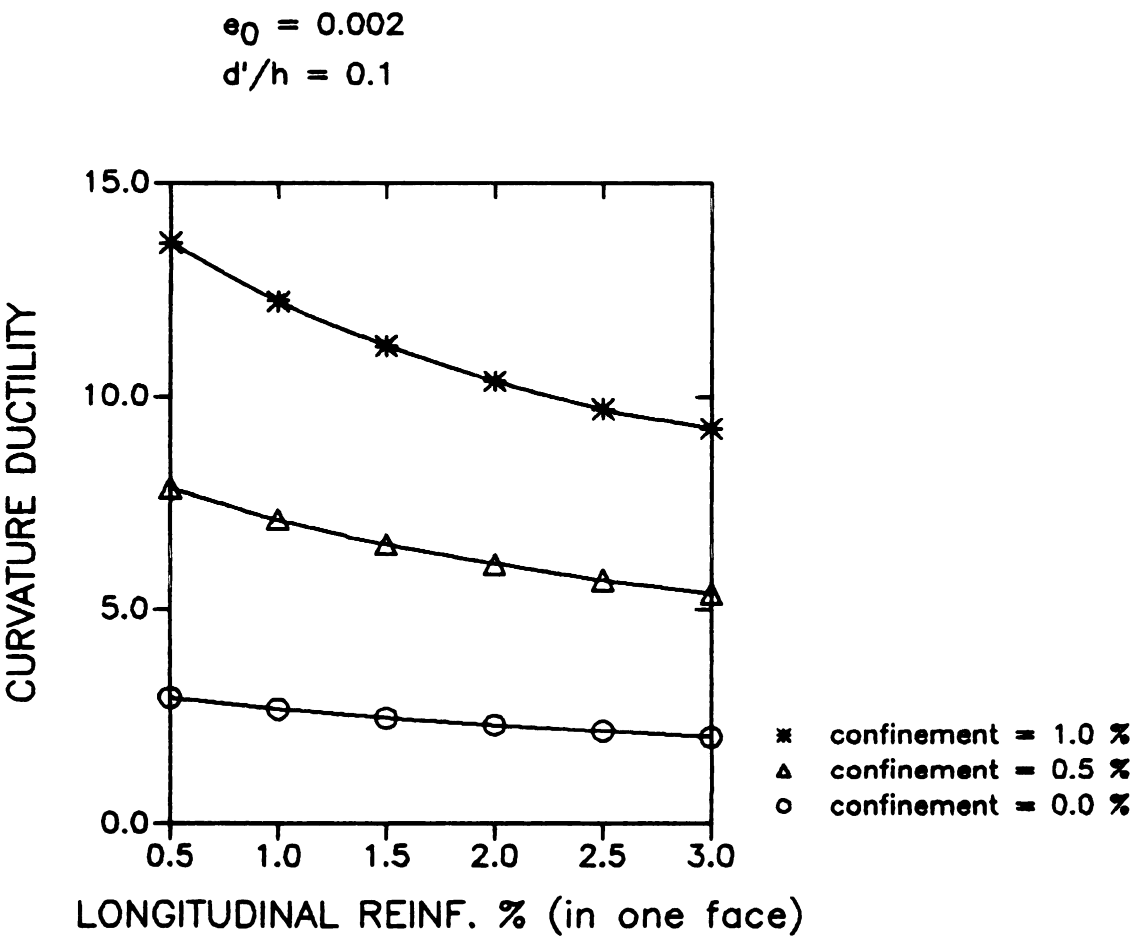


Fig.(4.3)—Curvature Ductility versus Steel Percentages
f'_c= 5 ksi f_y= 60 ksi $P/(f'_c\ b\ h)$= 0.20

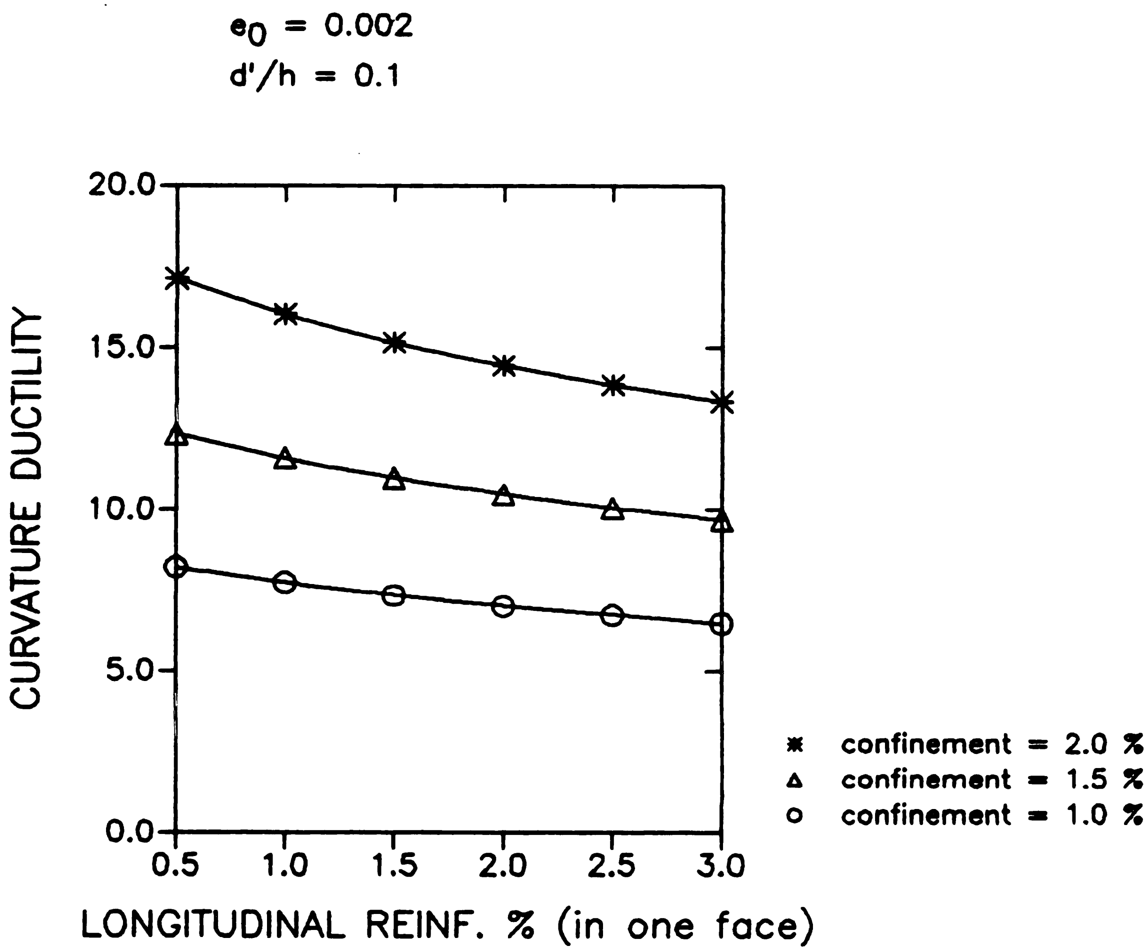


Fig.(4.4)—Curvature Ductility versus Steel Percentages

f'_c = 5 ksi f_y = 60 ksi $P/(f'_c\ b\ h)$ = 0.30

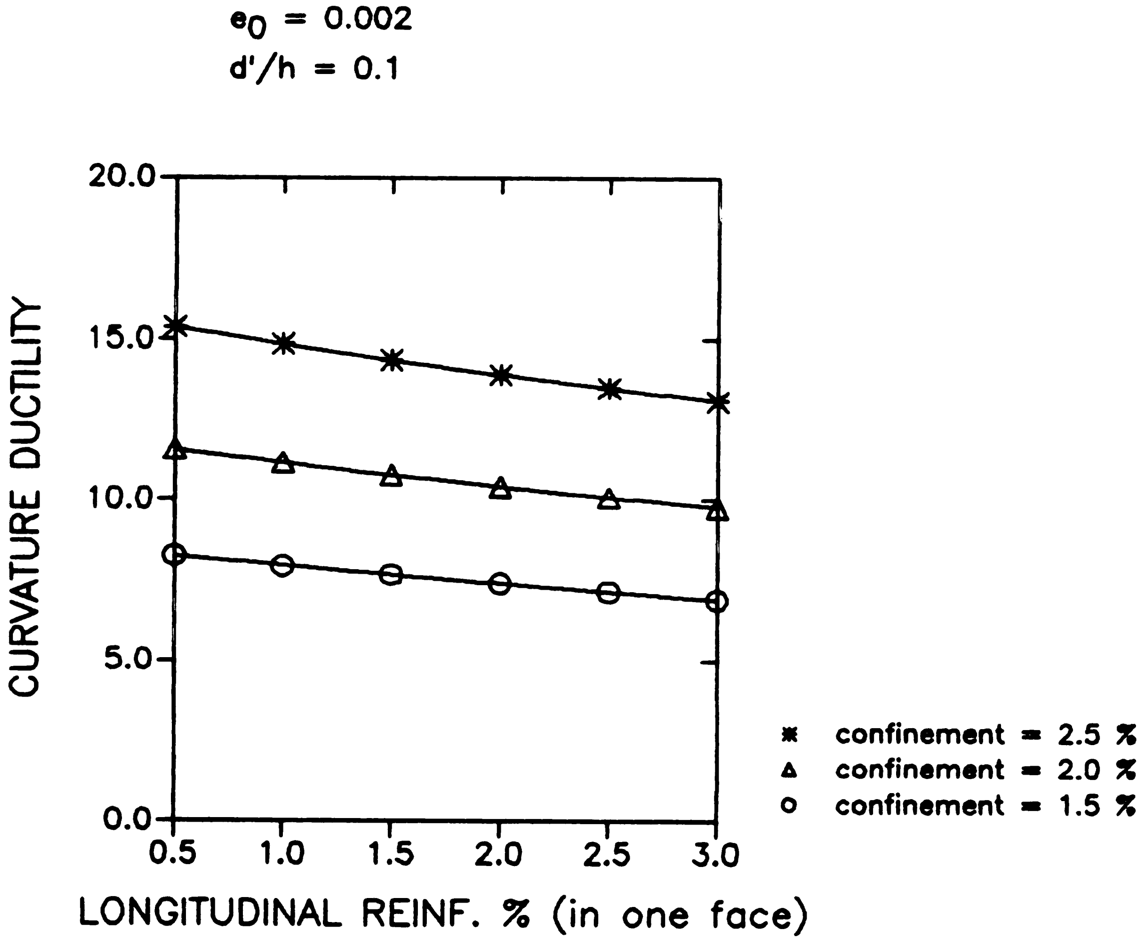


Fig.(4.5)—Curvature Ductility versus Steel Percentages
f'_c= 5 ksi f_y= 60 ksi $P/(f'_c\ b\ h)$= 0.40

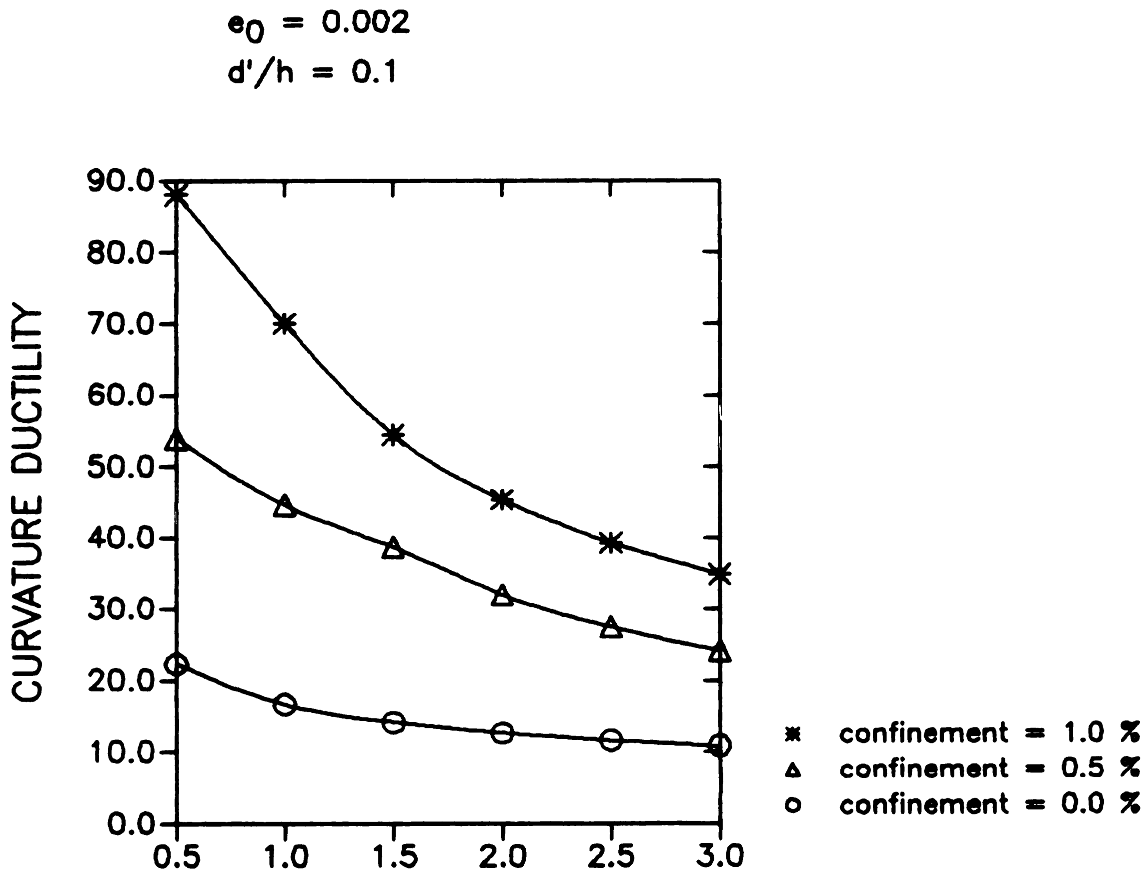


Fig.(4.6)—Curvature Ductility versus Steel Percentages

f'_c = 5 ksi f_y = 40 ksi $P/(f'_c\ bh) = 0.0$

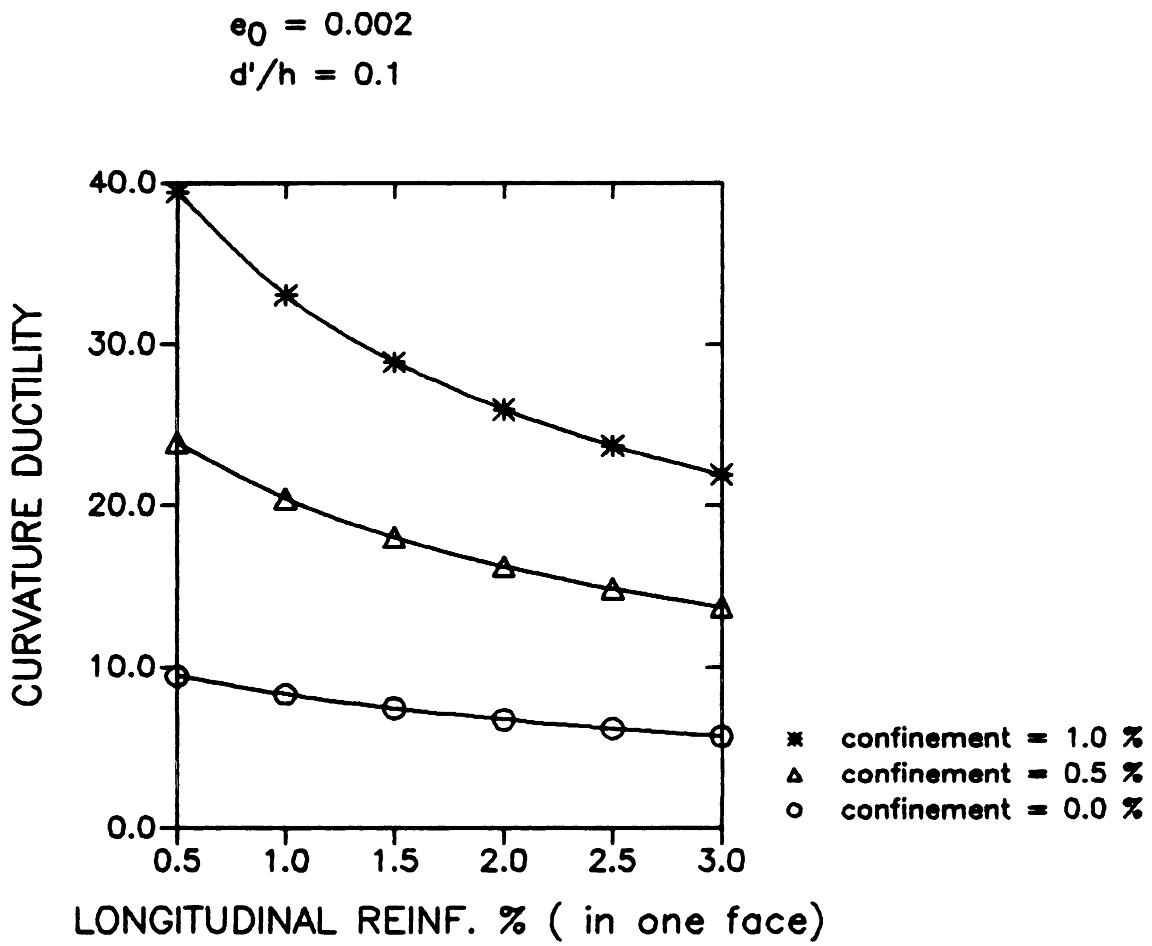


Fig.(4.7)—Curvature Ductility versus Steel Percentages

f'_c= 5 ksi f_y= 40 ksi $P/(f'_c\ b\ h)$= 0.10

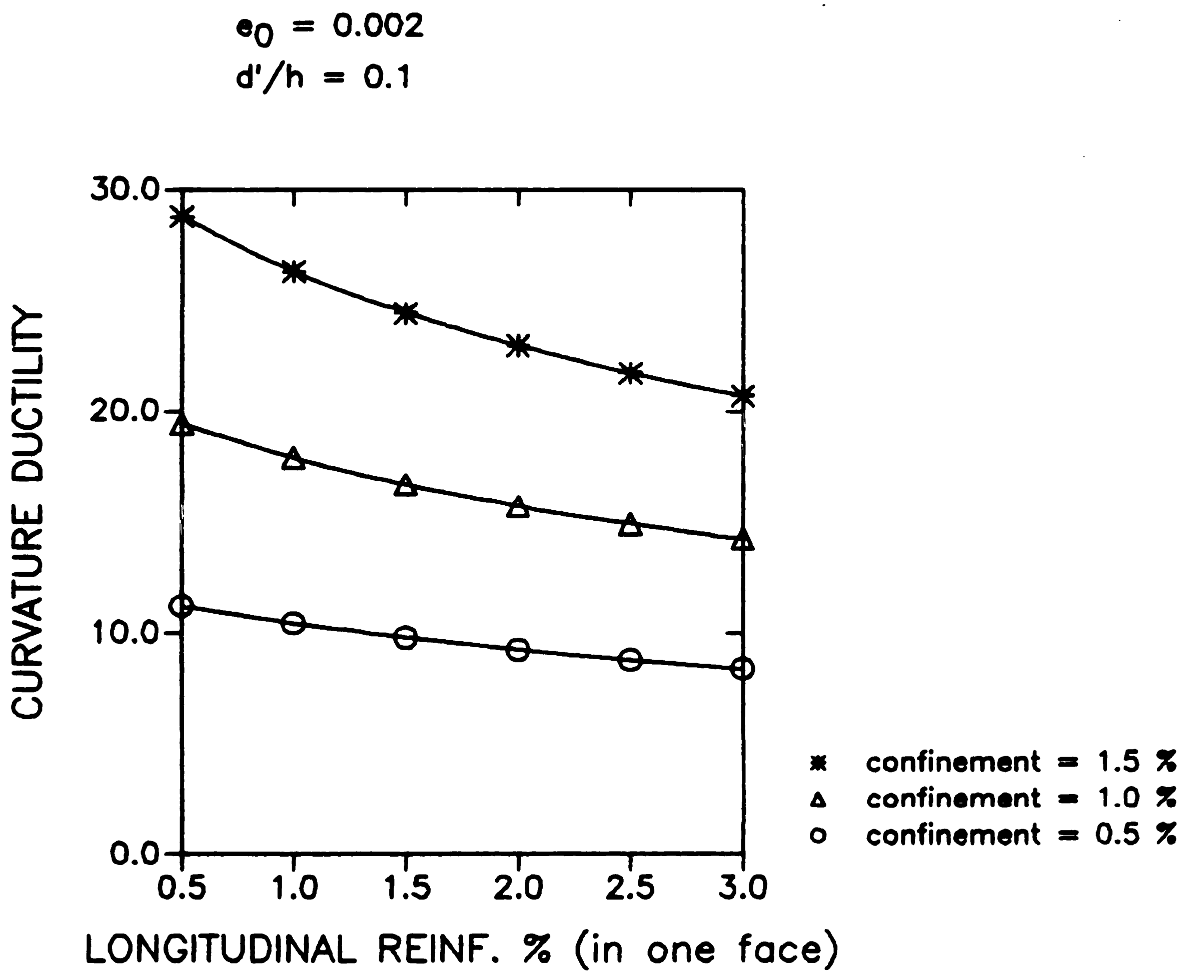


Fig.(4.8)—Curvature Ductility versus Steel Percentages

f'_c= 5 ksi f_y= 40 ksi $P/(f'_c\ b\ h)$= 0.20

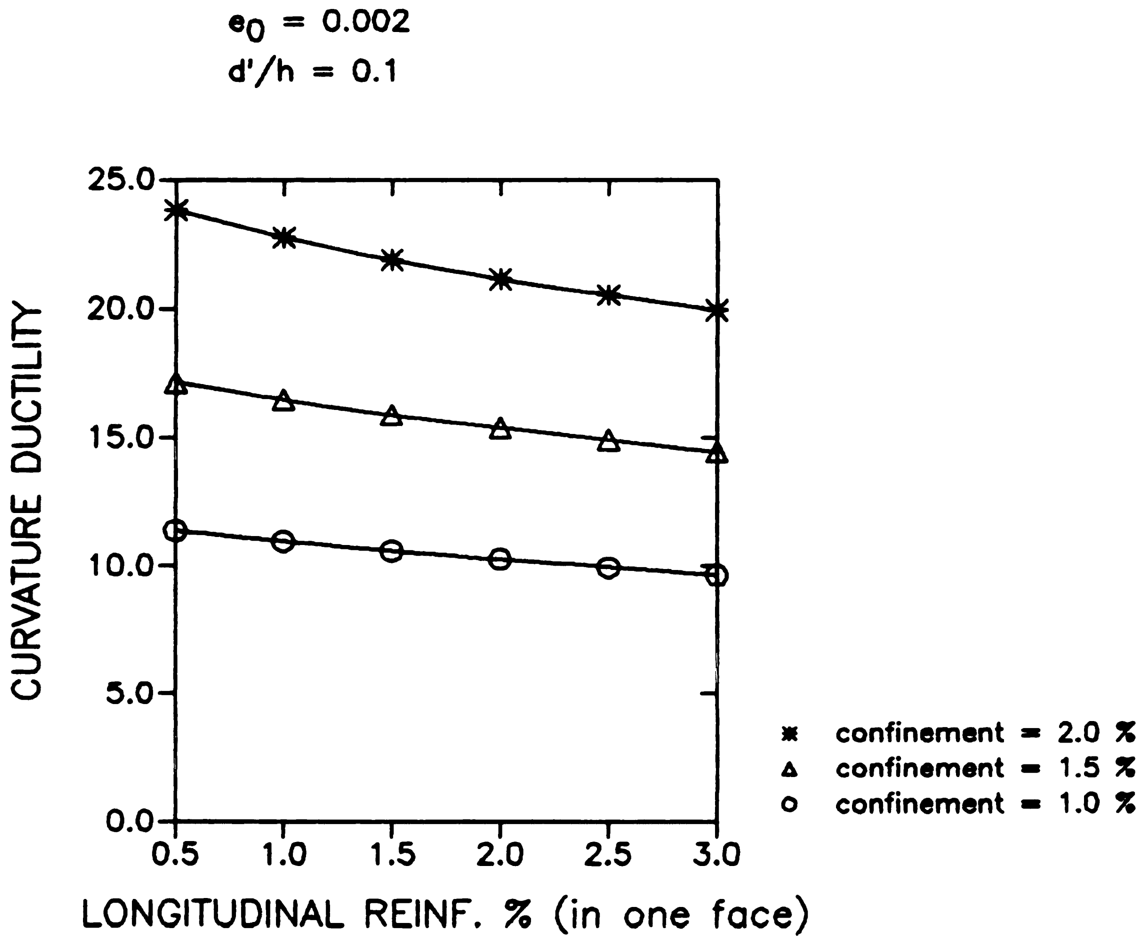


Fig.(4.9)—Curvature Ductility versus Steel Percentages
f'_c= 5 ksi f_y= 40 ksi $P/(f'_c\ b\ h)$= 0.30

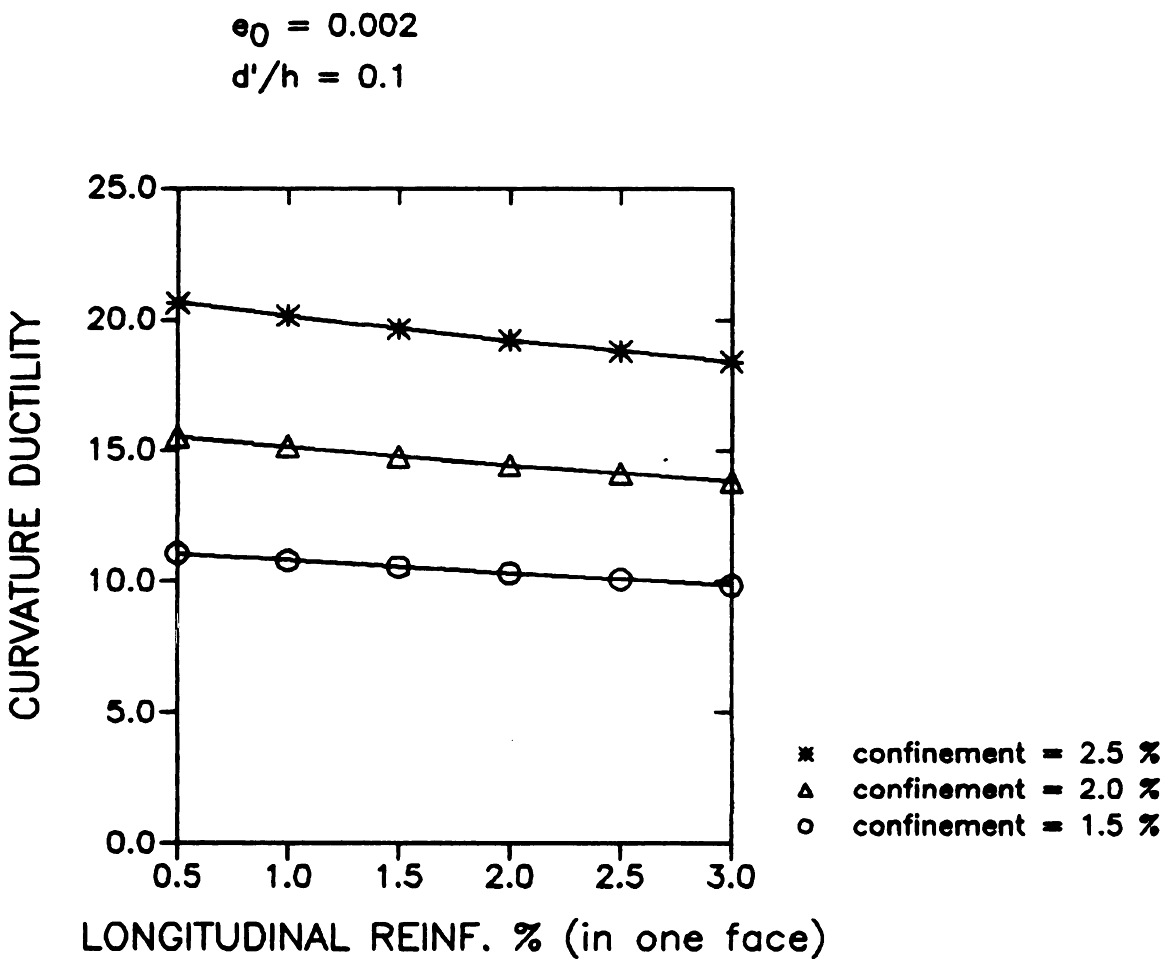


Fig.(4.10)—Curvature Ductility versus Steel Percentages
f'_c= 5 ksi f_y= 40 ksi $P/(f'_c\ b\ h)$= 0.40

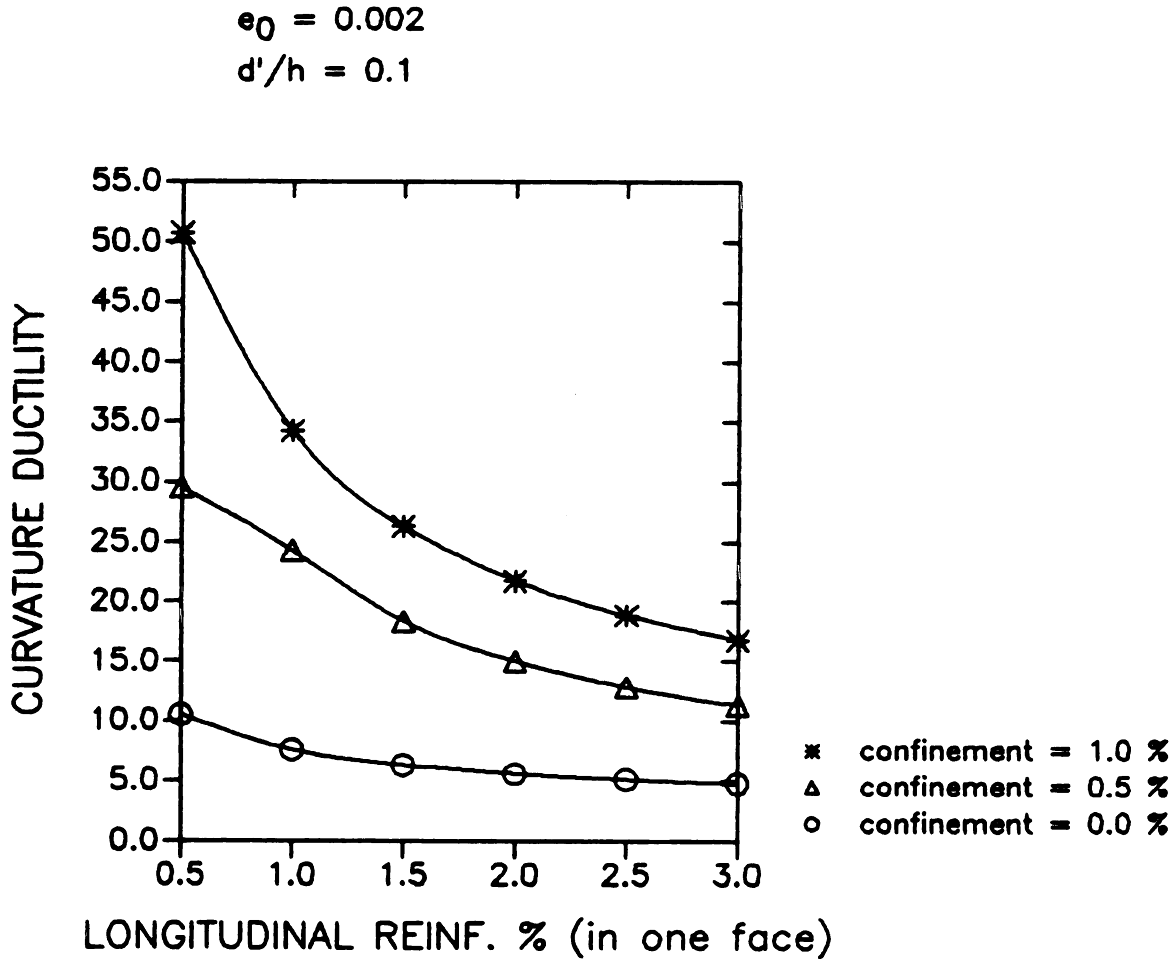


Fig.(4.11)—Curvature Ductility versus Steel Percentages
f'_c= 3 ksi f_y= 60 ksi $P/(f'_c\ b\ h)$ = 0.0

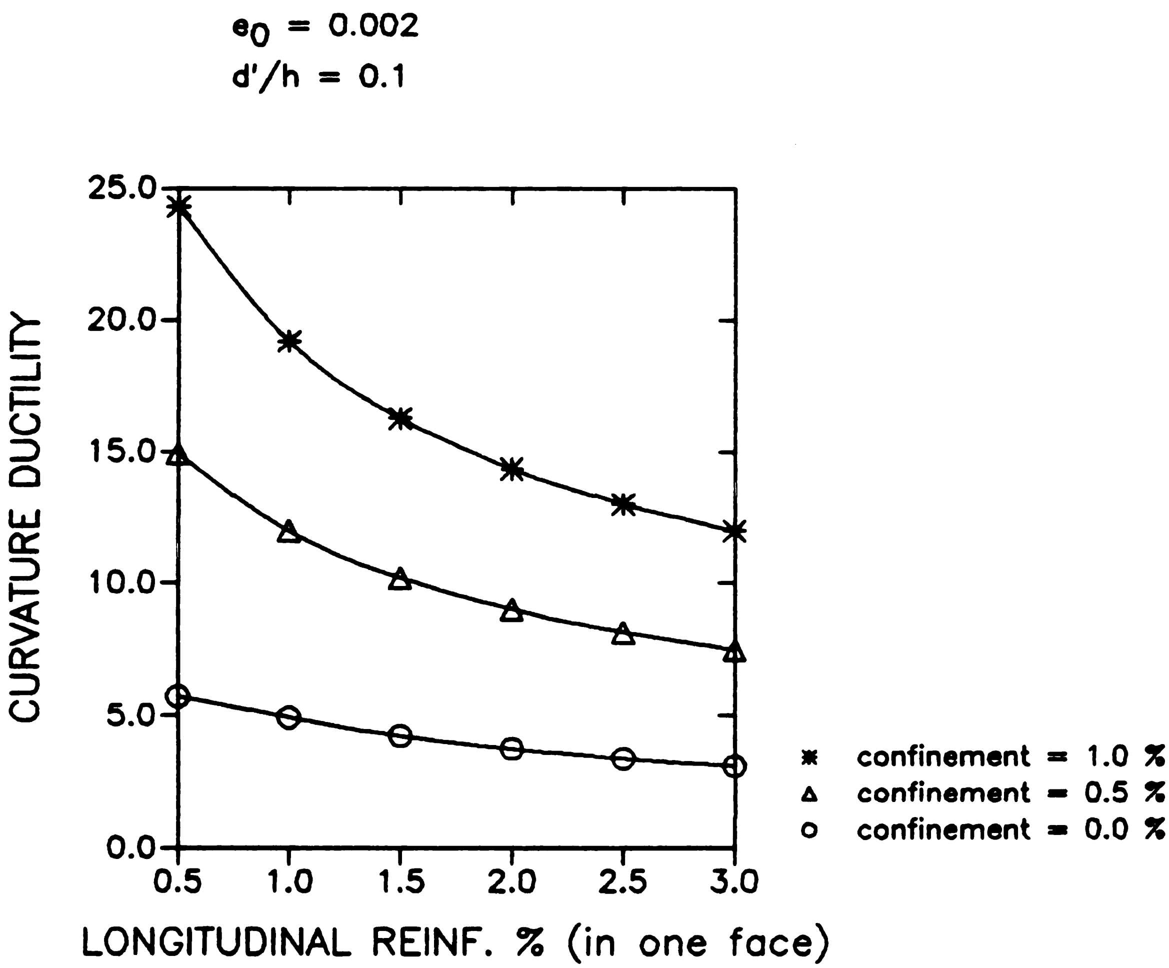


Fig.(4.12)—Curvature Ductility versus Steel Percentages
f'_c= 3 ksi f_y= 60 ksi $P/(f'_c\ b\ h)$= 0.10

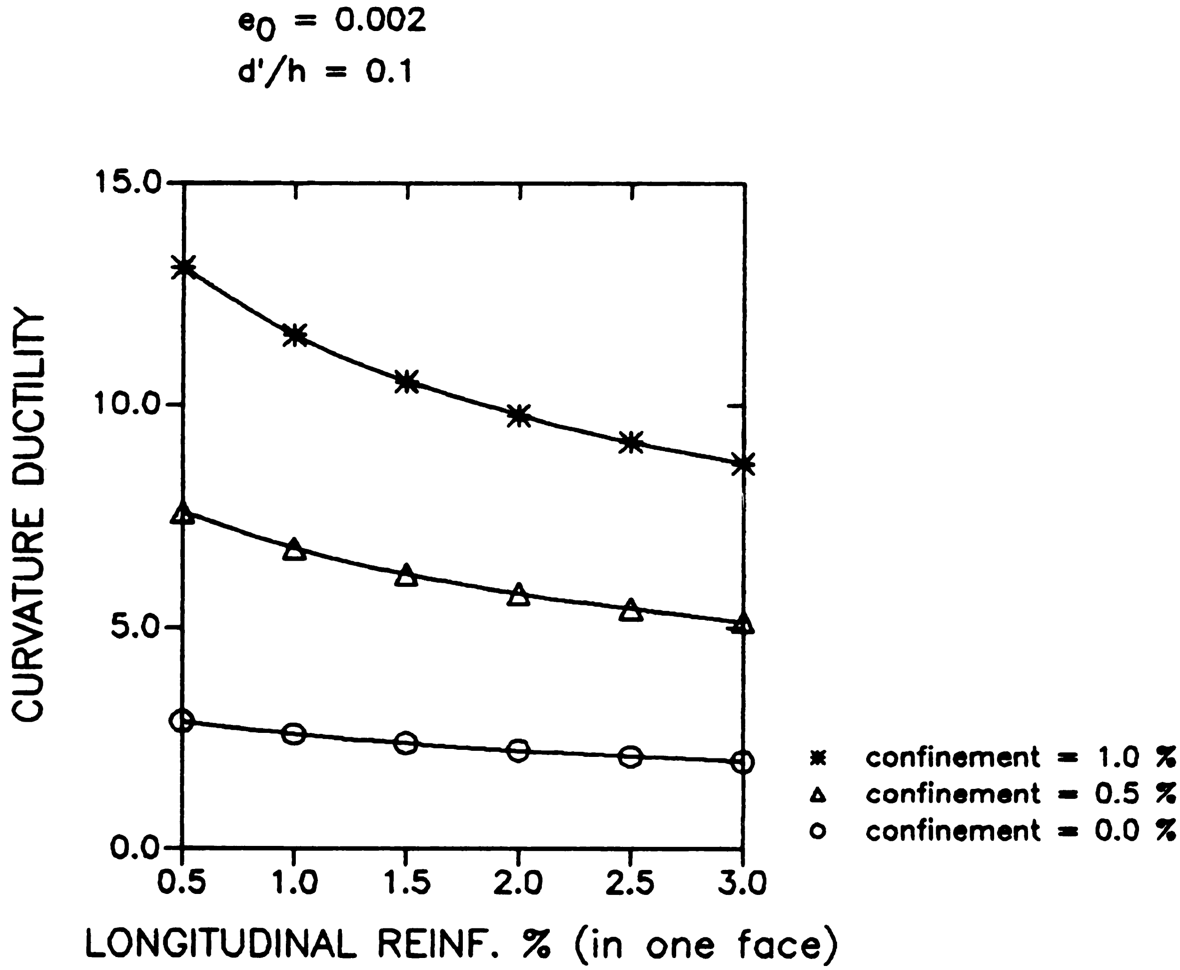


Fig.(4.13)–Curvature Ductility versus Steel Percentages
f'_c= 3 ksi f_y= 60 ksi $P/(f'_c\ b\ h)$= 0.20

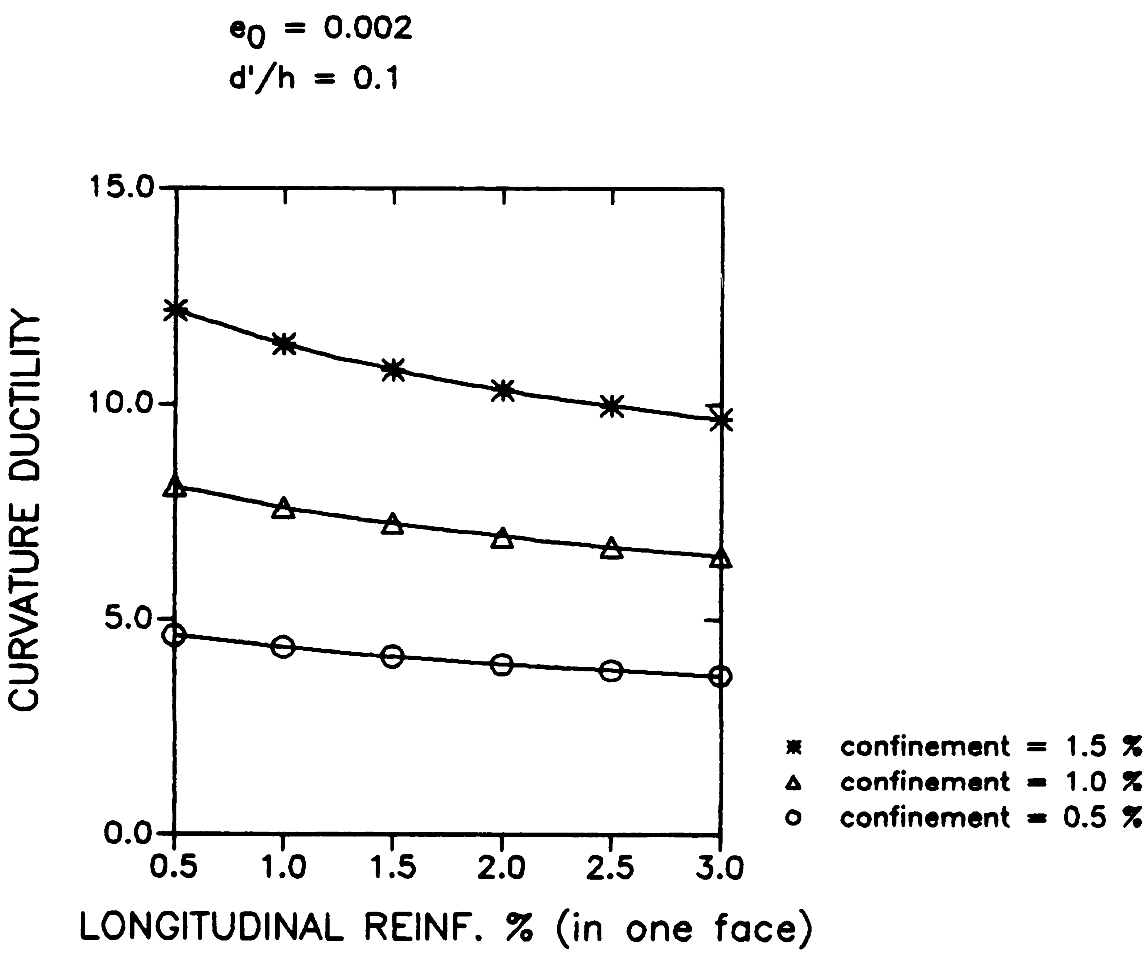


Fig.(4.14)—Curvature Ductility versus Steel Percentages
f'_c= 3 ksi f_y= 60 ksi $P/(f'_c\ b\ h)$= 0.30


CURVATURE DUCTILITY
f'c

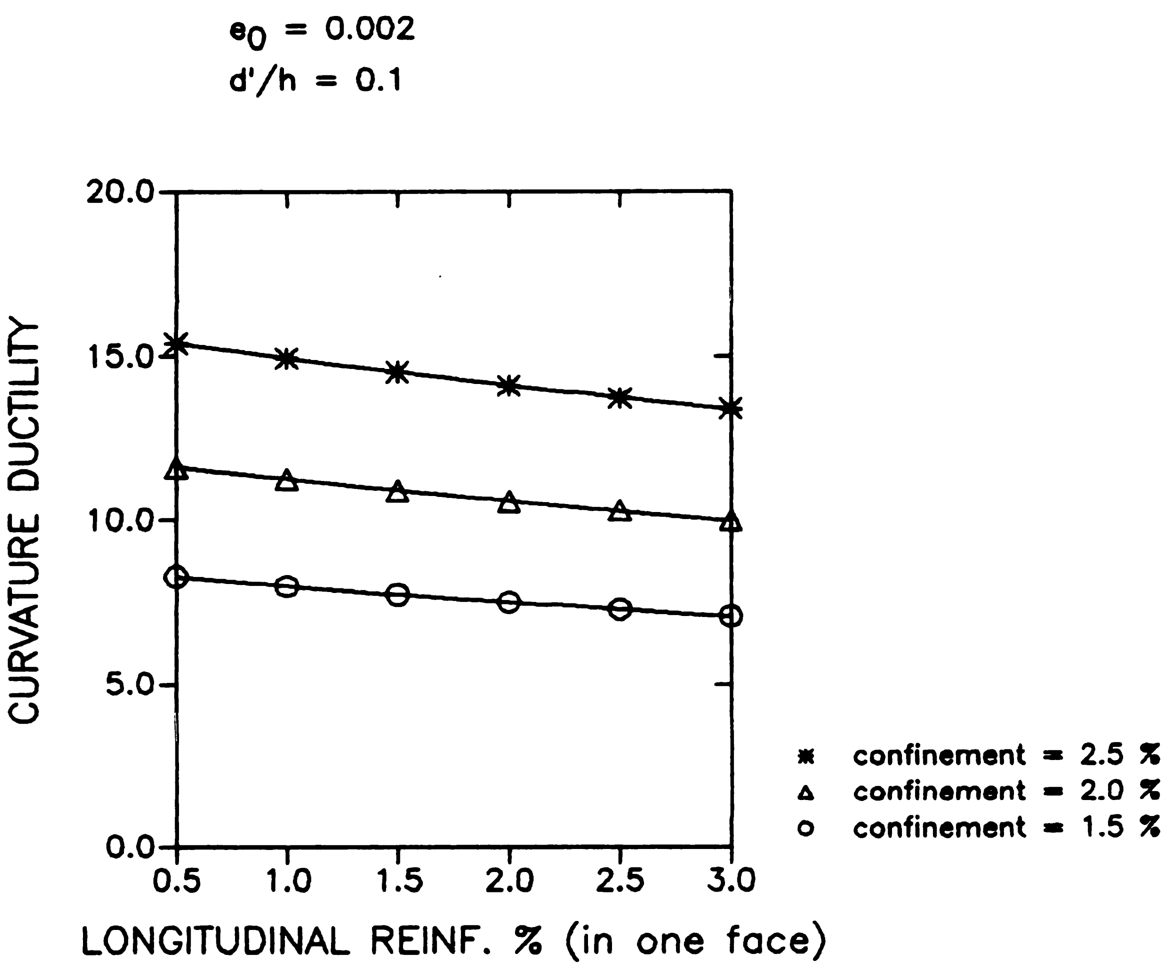


Fig.(4.15)—Curvature Ductility versus Steel Percentages
f'_c= 3 ksi f_y= 60 ksi $P/(f'_c\ b\ h)$= 0.40

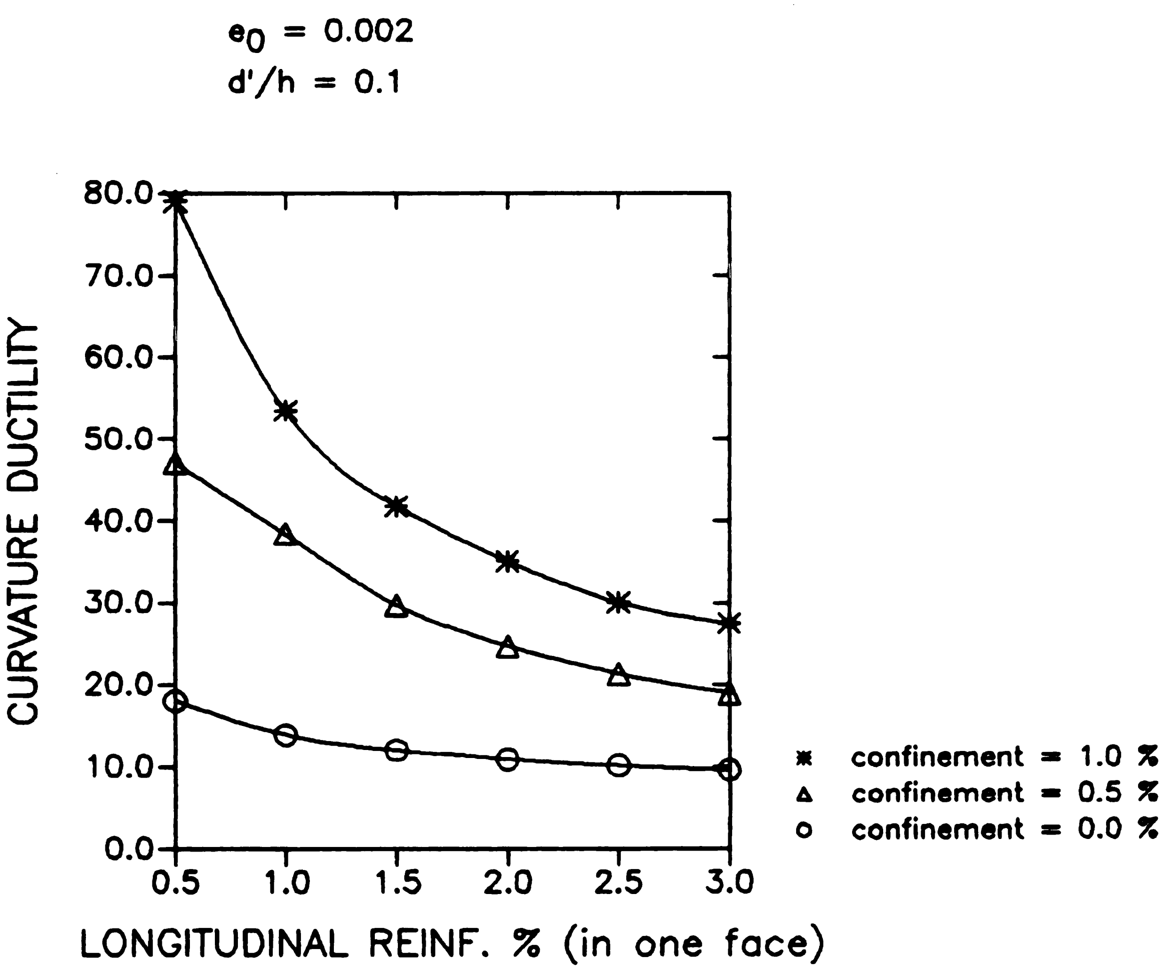


Fig.(4.16)–Curvature Ductility versus Steel Percentages
f'_c= 3 ksi f_y= 40 ksi $P/(f'_c\ b\ h)$= 0.0

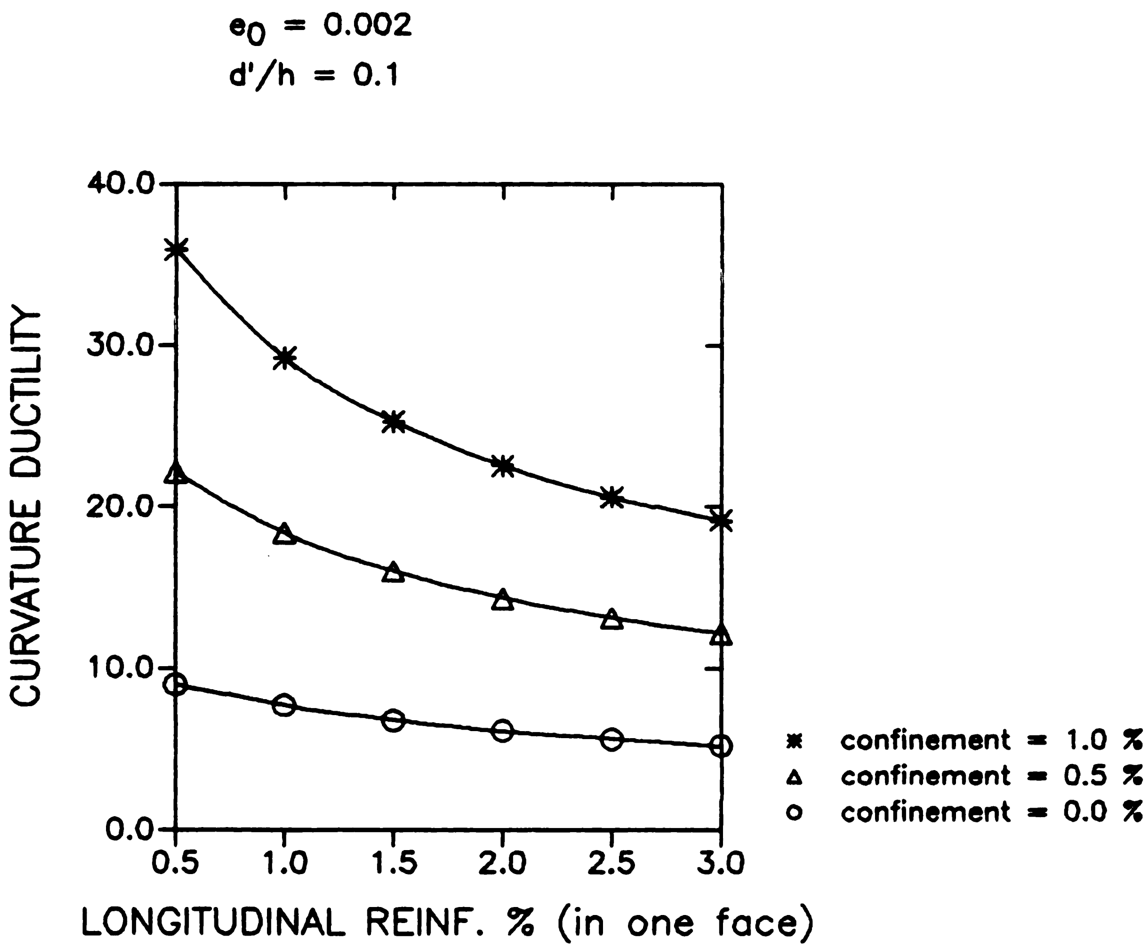


Fig.(4.17)–Curvature Ductility versus Steel Percentages
f'_c= 3 ksi f_y= 40 ksi $P/(f'_c\ b\ h)$= 0.10

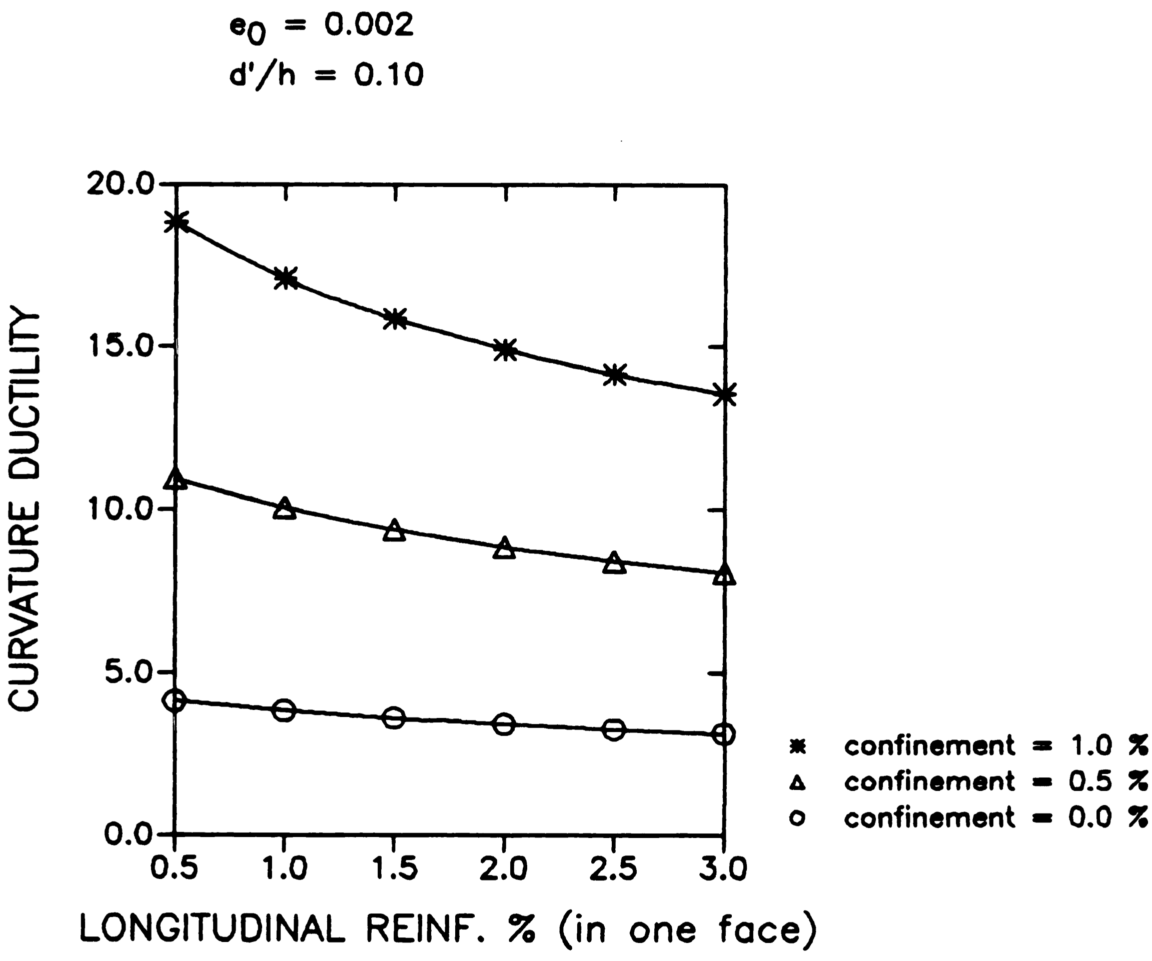


Fig.(4.18)–Curvature Ductility versus Steel Percentages
f'_c= 3 ksi f_y= 40 ksi $P/(f'_c\ b\ h)$= 0.20

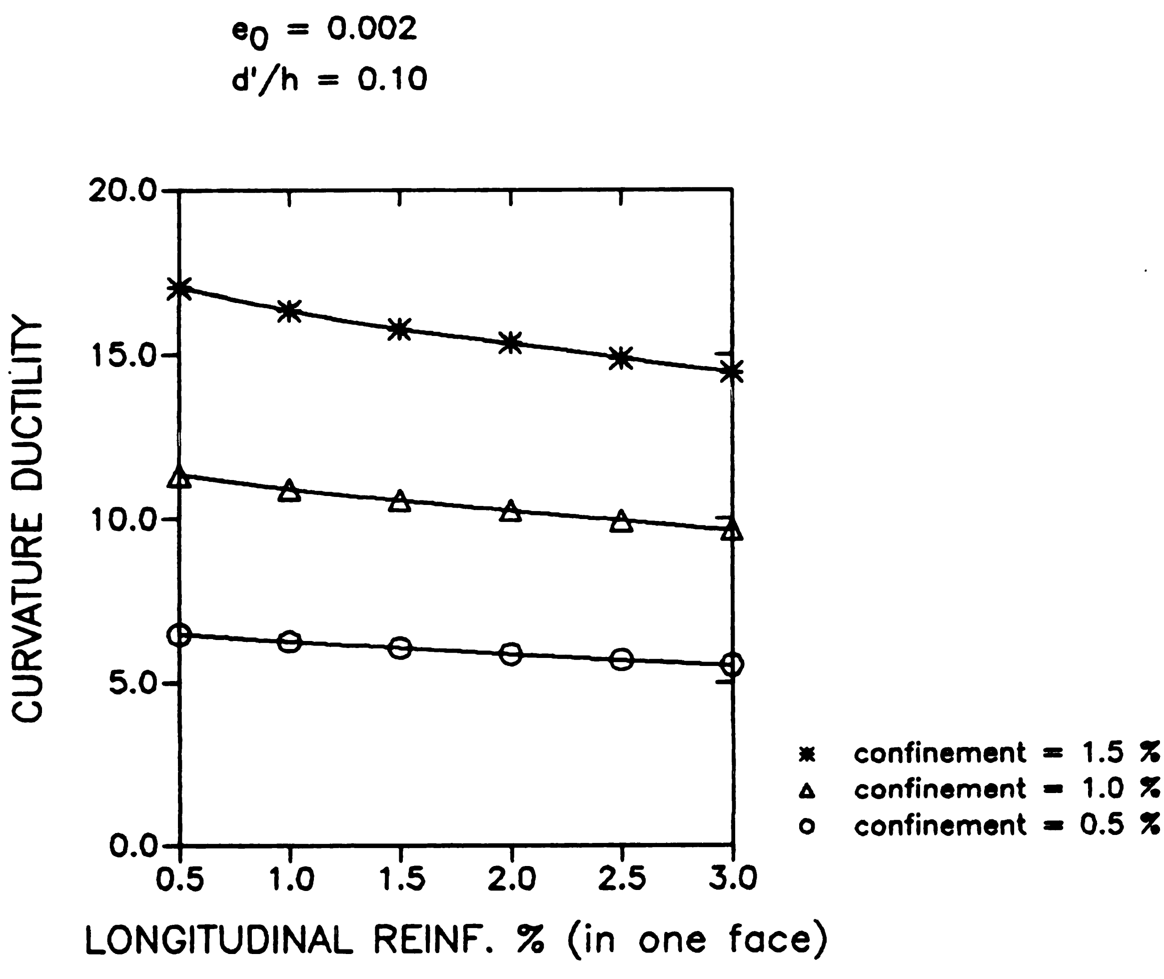


Fig.(4.19)—Curvature Ductility versus Steel Percentages

f'_c= 3 ksi f_y= 40 ksi $P/(f'_c\ b\ h)$= 0.30

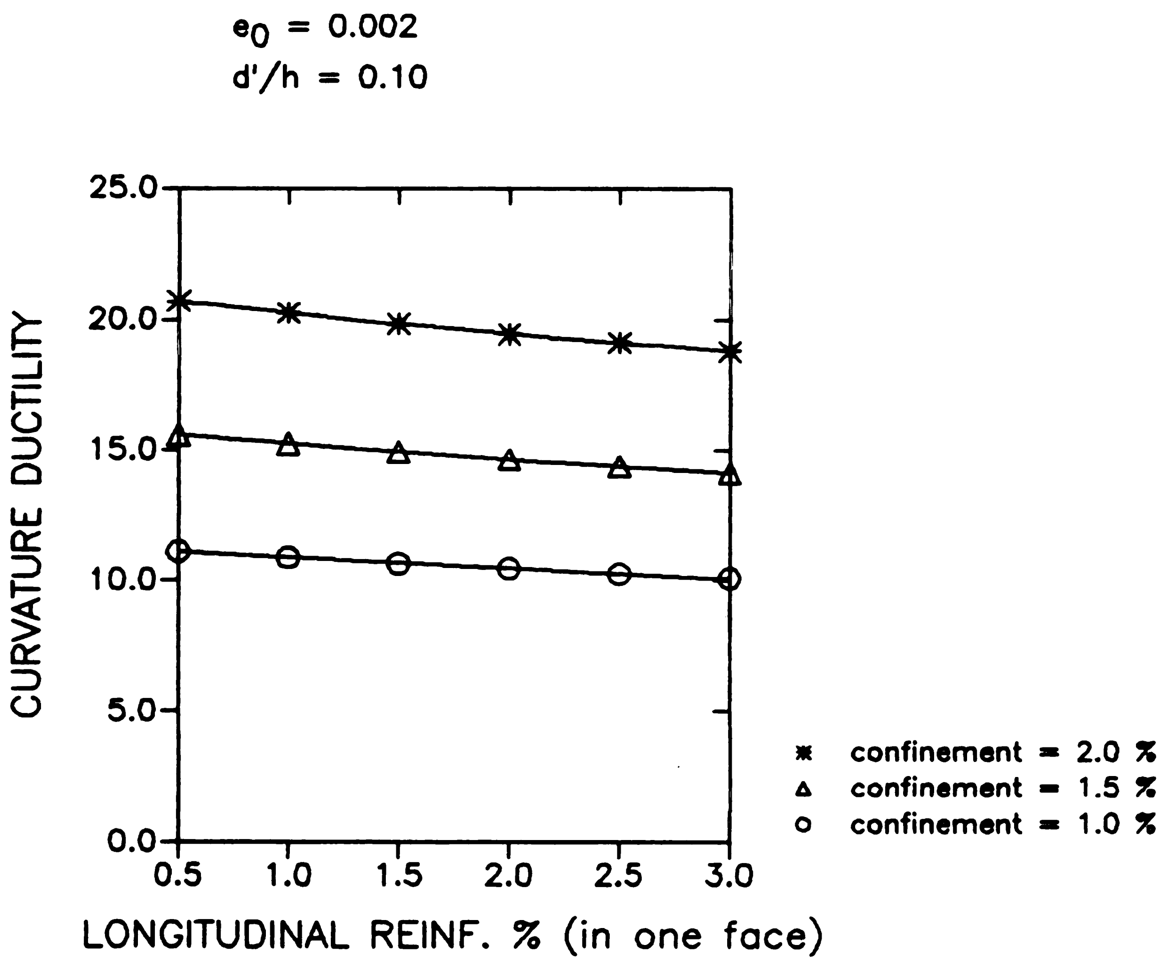
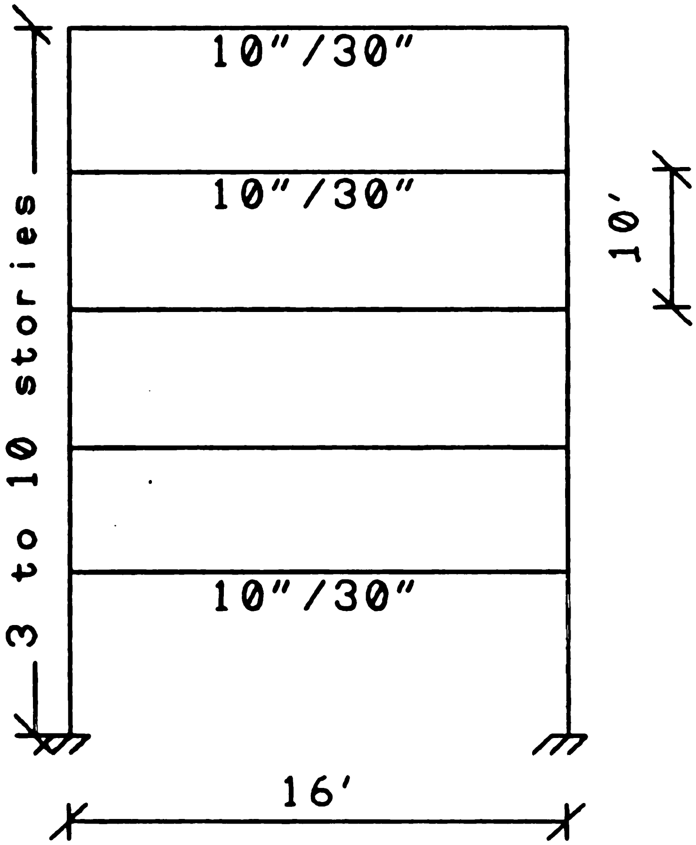


Fig.(4.20)—Curvature Ductility versus Steel Percentages
f'_c= 3 ksi f_y= 40 ksi $P/(f'_c\ b\ h)$= 0.40



Columns Size:

3 to 10 stories - 10"/10"
6 to 8 stories - 15"/15"
9 and 10 stories - 20"/20"

Fig.(4.21) - Basic Structure Properties

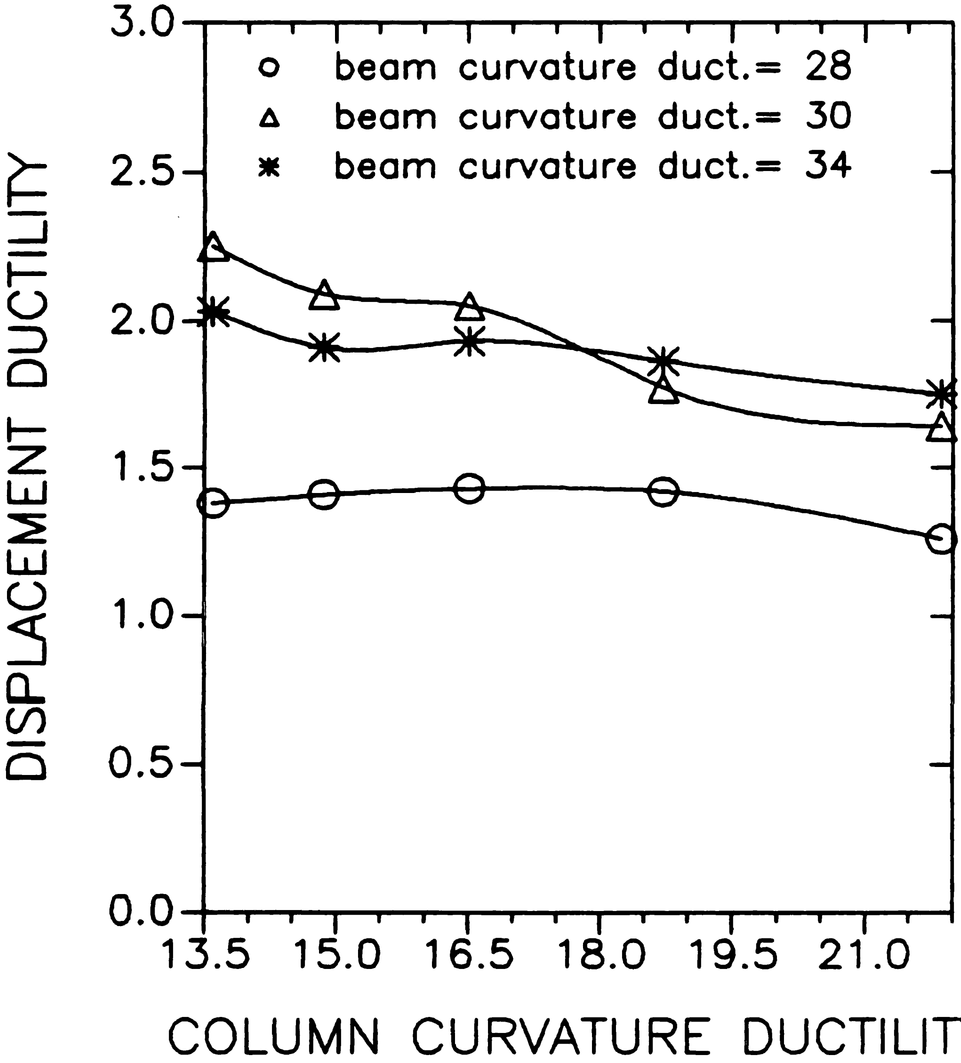


Fig.(4.22)–Displacement Ductility versus Curvature Ductility THREE STORIES FRAME

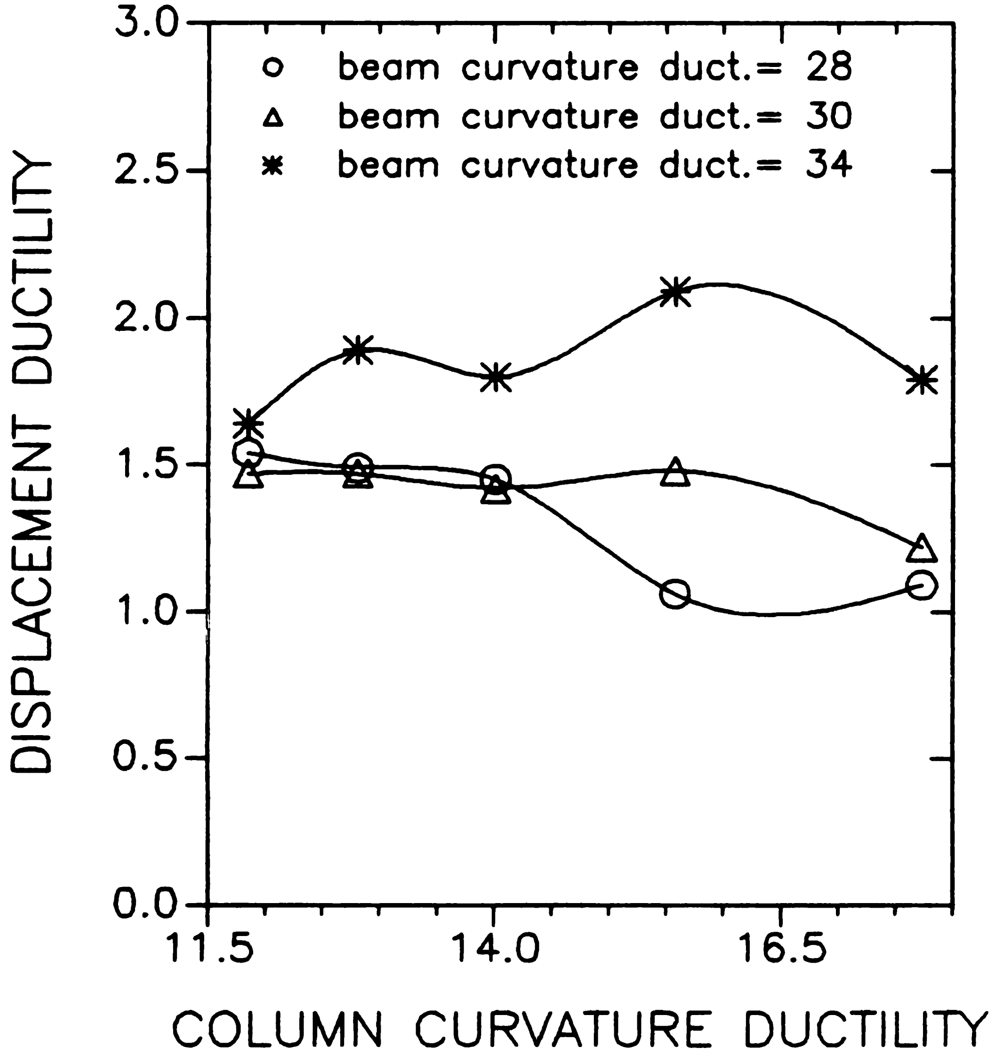


Fig.(4.23)–Displacement Ductility versus Curvature Ductility
FOUR STORIES FRAME

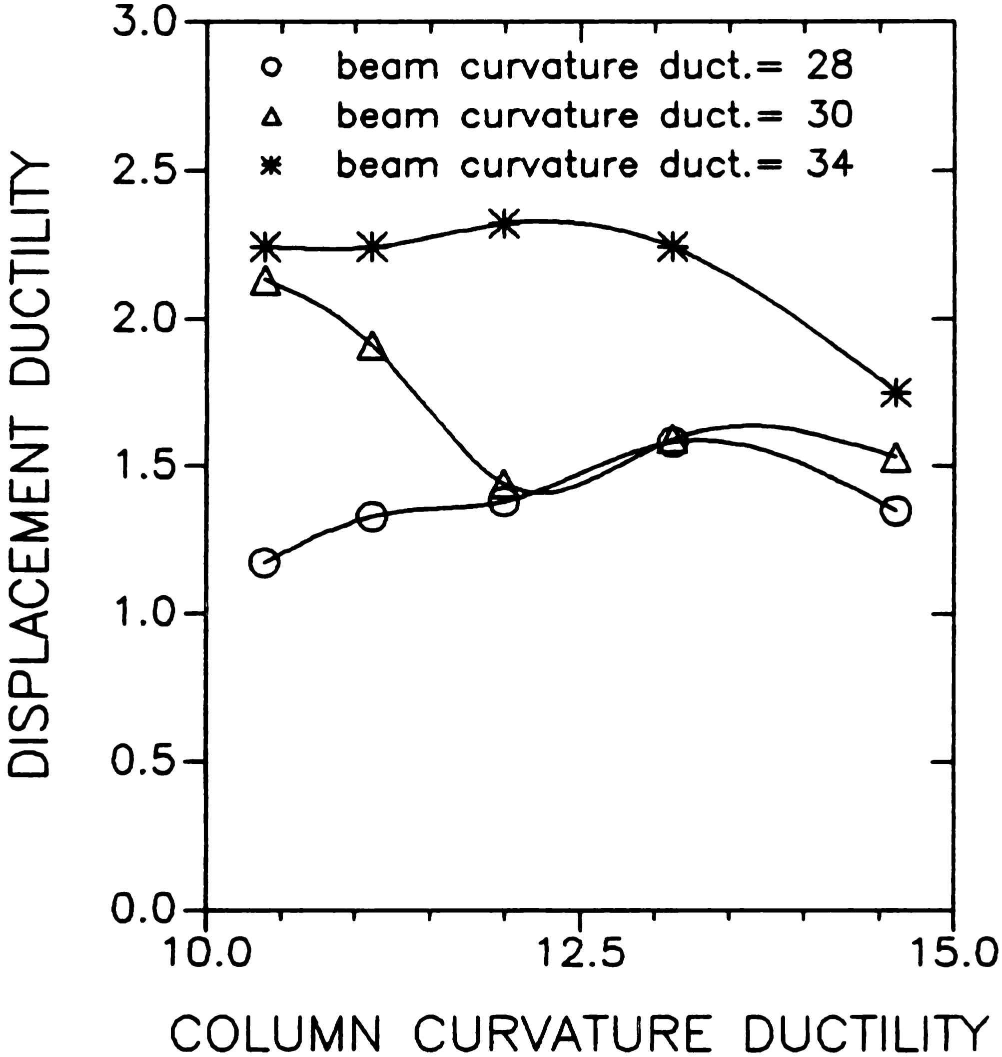


Fig.(4.24)—Displacement Ductility versus Curvature Ductility FIVE STORIES FRAME

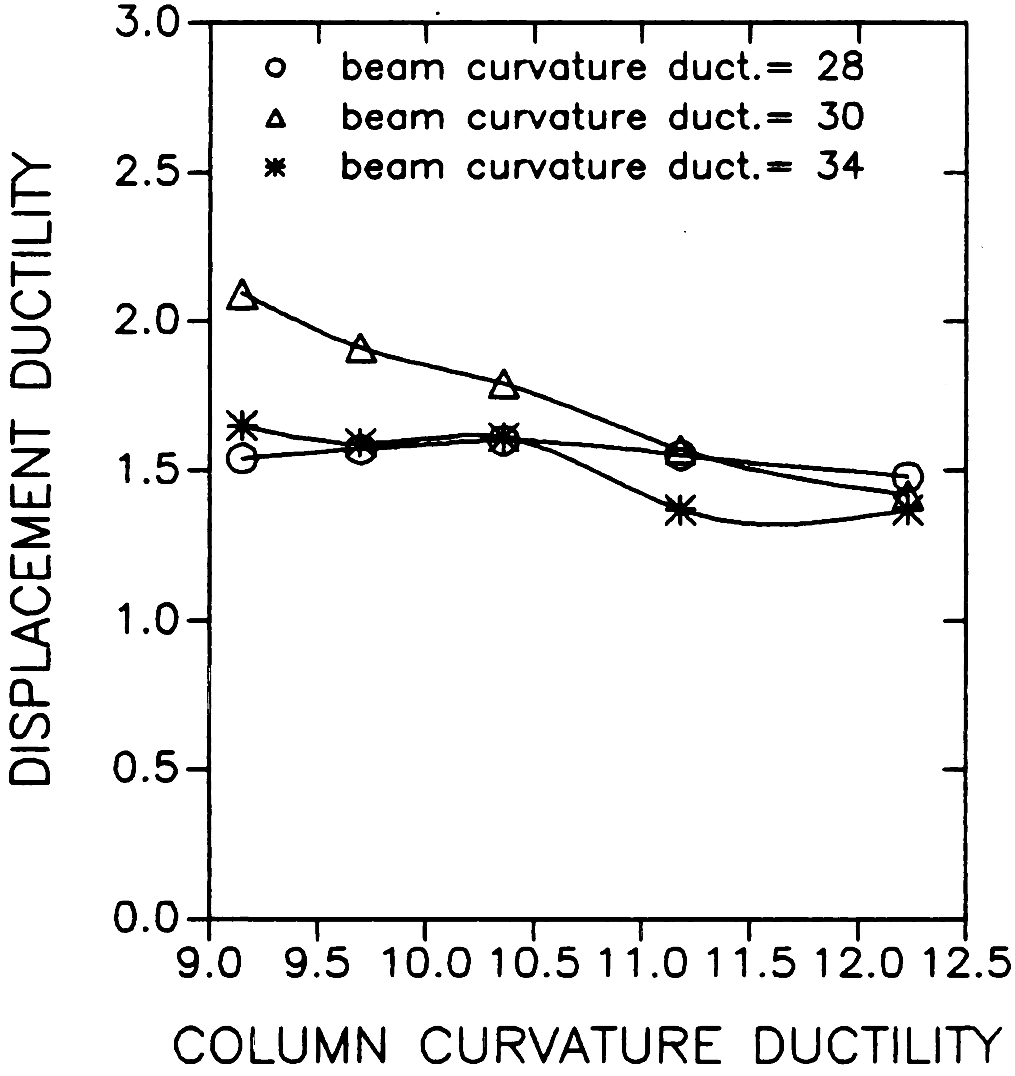


Fig.(4.25)—Displacement Ductility versus Curvature Ductility SIX STORIES FRAME

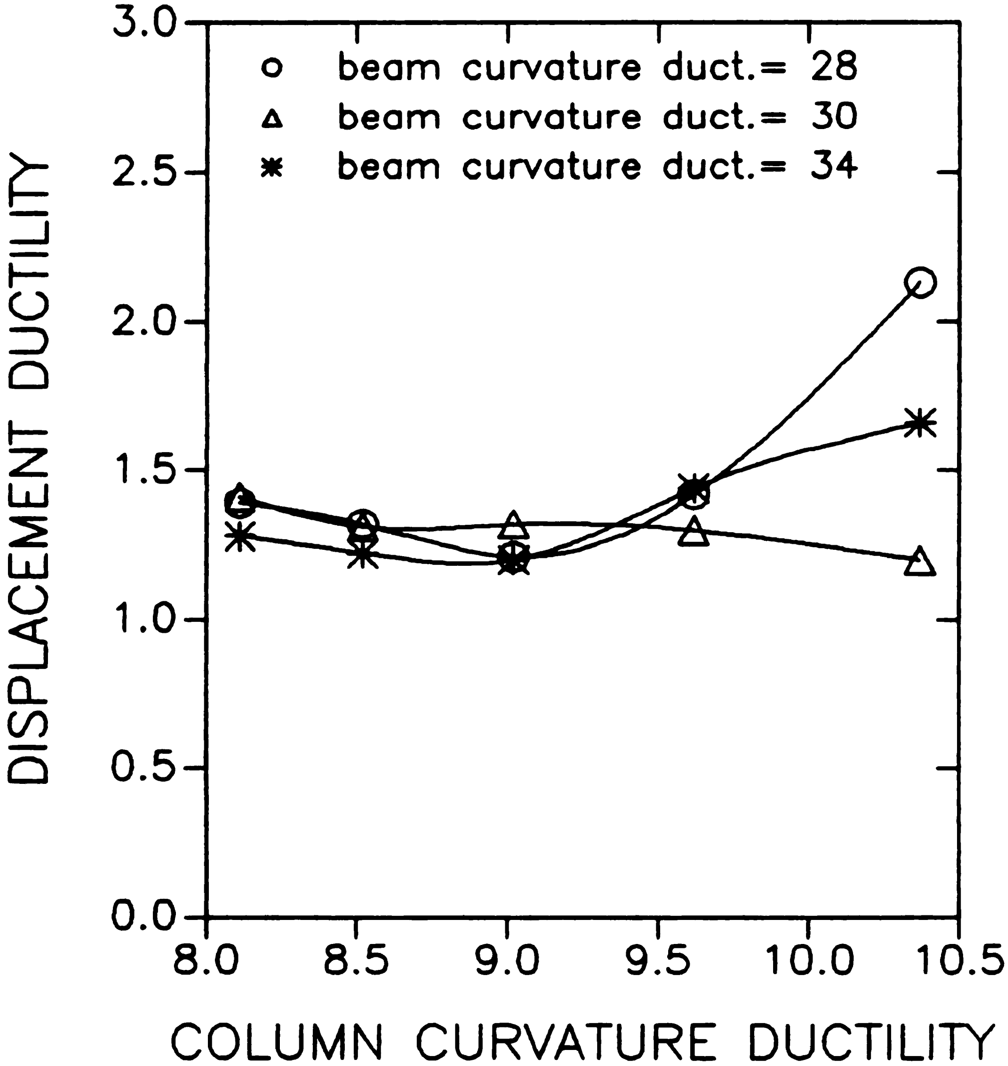


Fig.(4.26)—Displacement Ductility versus
Curvature Ductility
SEVEN STORIES FRAME

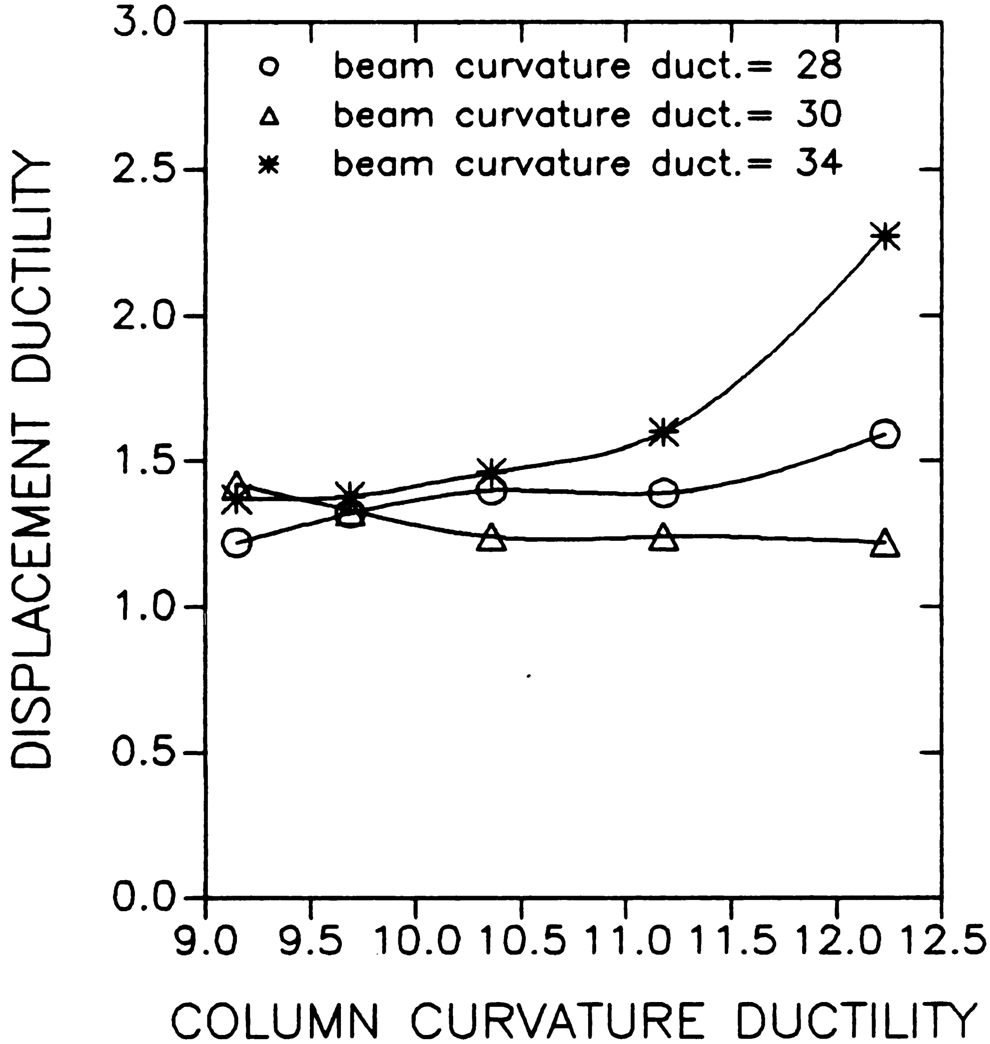


Fig.(4.27)–Displacement Ductility versus Curvature Ductility
EIGTH STORIES FRAME

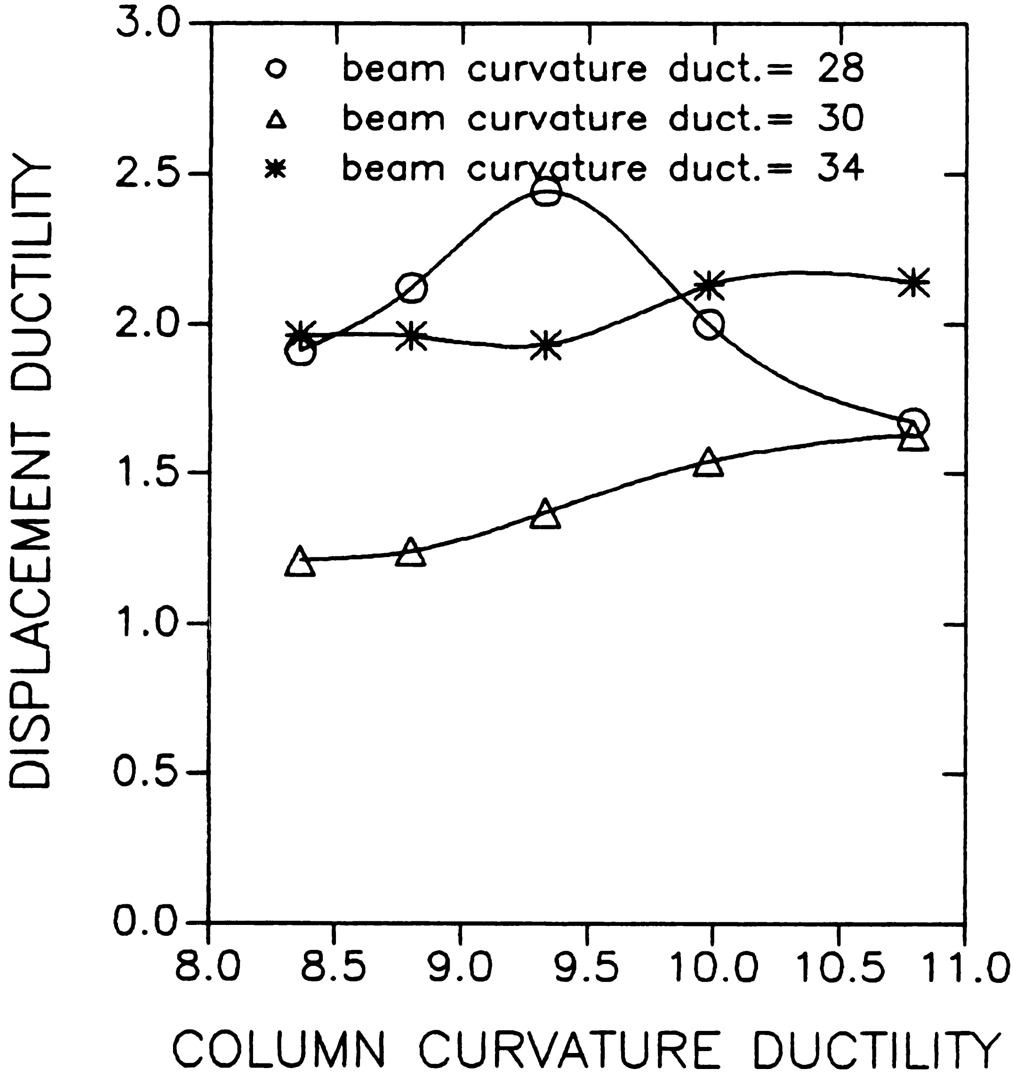


Fig.(4.28)—Displacement Ductility versus Curvature Ductility
NINE STORIES FRAME

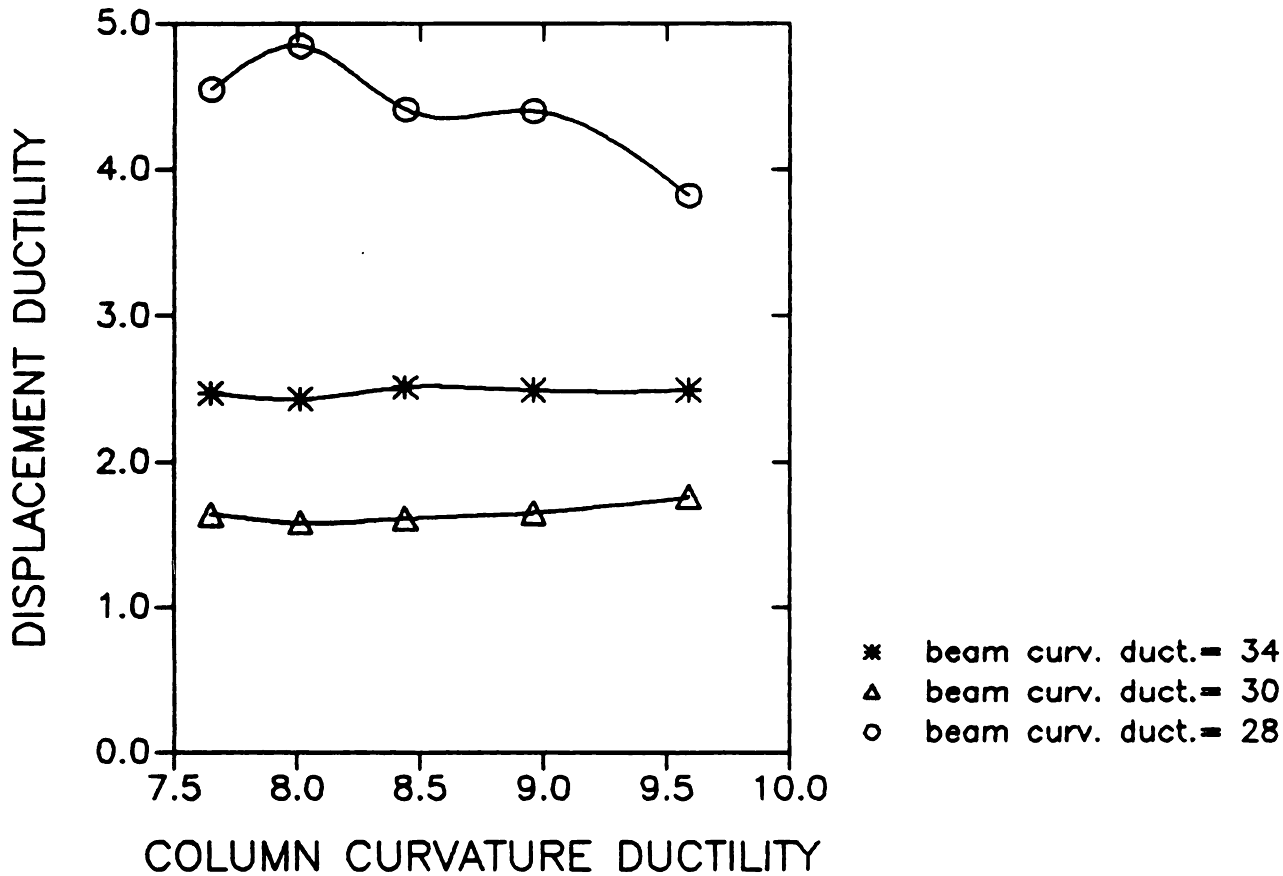


Fig.(4.29)—Displacement Ductility versus Curvature Ductility
TEN STORIES FRAME

Damage Factors(*)						
Number of Stories	Beam Curvature	ρ Values for Columns Used				
		2%	3%	4%	5%	6%
3	28	0.47C	0.42C	0.49C	0.43	0.48
	30	0.46C	0.47C	0.26C	0.41	0.43
	34	0.46C	0.44	0.42	0.38	0.41
4	28	0.38	0.48	0.52	0.35	0.50
	30	0.47C	0.44	0.44	0.47	0.49
	34	0.46C	0.41	0.47	0.52	0.43
5	28	0.50C	0.49C	0.49	0.35	0.32
	30	0.50C	0.49	0.50	0.50	0.45
	34	0.48	0.44	0.43	0.53	0.45
6	28	0.50C	0.47C	0.50C	0.52	0.48
	30	0.44	0.47	0.50	0.45	0.50
	34	0.41	0.52	0.51	0.47	0.53
7	28	0.27	0.45	0.30	0.49	0.53
	30	0.32	0.29	0.38	0.52	0.49
	34	0.52	0.50	0.36	0.40	0.50
8	28	0.37	0.52	0.49	0.53	0.49
	30	0.26	0.36	0.49	0.46	0.50
	34	0.50	0.53	0.51	0.41	0.47
9	28	0.34	0.26	0.24	0.40	0.39
	30	0.51	0.45	0.51	0.35	0.34
	34	0.50	0.50	0.54	0.50	0.40

Table 4.1 Damage Factors for One-bay Frames

Damage Factors(*)						
Number of Stories	Beam Curvature	ρ Values for Columns Used				
		2%	3%	4%	5%	6%
10	28	0.35	0.40	0.43	0.50	0.35
	30	0.46	0.52	0.53	0.51	0.52
	34	0.50	0.47	0.50	0.52	0.50

* - Factors listed are for beams unless followed by the letter C, in which case, the factor is associated with a column.

Table 4.1 Damage Factors for One-bay Frames (continued) an

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CHAPTER 5

DESIGN EXAMPLE

In Chapter 2, a design procedure was presented involving an explicit consideration of damage due to seismic loads. The computer software has been described in Chapter 3. Certain design aids to help guide necessary redesign were presented in Chapter 4.

In order to illustrate the material presented in the preceding chapters, the design of a three story frame is considered in this chapter. The geometry of the structure is shown in Fig. (5.1). For simplicity, all beams will have the same properties and so will all columns. Since an example on the use of the program for dynamic analysis has already been given in Appendix B, it will not be repeated here.

The following basic steps, as described before, will be used to illustrate the design procedure.

a) Preliminary design - applying prescribed dead load and live load and earthquake loading in the form of static lateral loading as prescribed by the UBC Code, the initial sizing of structural members and steel reinforcement are determined based on a linear elastic analysis and the ACI Building Code.

b) Nonlinear dynamic analysis - it is performed on the structure using the MC-QUAKE computer program presented in Chapter 3.

c) Check damage - in case of occurrence of high damage to one or more members(according to the damage scale factor, as presented in Chapter 1) the frame should be redesigned. This will be guided by what was presented in Chapter 4 .

d) Redesign - using Figures (4.22) to (4.29) in Chapter 4, for a particular number of stories, according to the value of the displacement ductility chosen for the frame, the curvature of the beams and the columns may be selected from design aids such as Figures (4.1) to (4.20), for a particular combination of material properties, and using the curvature ductility chosen for the members, new values for the confinement steel or for the longitudinal reinforcement may be determined. Using these new values, another nonlinear dynamic analysis is performed as in b), and so on.

5.1 Preliminary Design

5.1.1 Design Basic Data and Specifications

The following data are given for the design:

a) concrete properties:

$f'_c = 3$ ksi

$\epsilon_0 = 0.002$

b) reinforcement steel properties:

$f_y = 60$ ksi

$E_s = 29,000$ ksi

$p_s = 0.01$

$\epsilon_s = 0.015$

Dimensions of the members have been assumed as illustrated in Fig. (5.1).

In accordance with ACI Code,

$U = 1.4\ D + 1.7\ L$

When wind load W is considered in the design the required strength U provided should also be no less than

$U = 0.75\ (\ 1.4\ D + 1.7\ L + 1.7\ W\ )$

If an earthquake load E is to be included, W will be replaced by 1.1 E.

Then for the earthquake load one has

$U = 0.75\ (1.4\ D + 1.7\ L + 1.7 \cdot 1.1\ E\ )$

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For seismic loads the UBC Code is adopted.

5.1.2 Loads

a) Dead Load + Live Load

Let the dead load intensity w_D and live load intensity w_L be given as:

w_D = 0.2 kips/in

w_L = 0.2 kips/in(floor) ; w_L = 0.04 kips/in(roof)

Then,

$w_U = 0.75(1.4 \cdot 0.2 + 1.7 \cdot 0.2)$; w_U = 0.465 kips/in(floor)

$w_U = 0.75(1.4 \cdot 0.2 + 1.7 \cdot 0.04)$; w_U = 0.261 kips/in(roof)

b) Seismic Loads(in accordance with the UBC Code)

The structure is designed to resist minimum total lateral seismic forces in accordance with the following formula:

$V = Z \cdot I \cdot K \cdot C \cdot S \cdot W$

The value of K is not less than that set forth in Table No. 23-I of the UBC Code.

Considering that the frame will be a ductile moment-resisting frame,

$K = 0.67$

The value of C is determined in accordance with the following formula:

$$C = \frac{1}{15\sqrt{T}}$$

The value of C need not exceed 0.12.

The period T may be determined by the following formula:

$$T = \frac{0.05\ h_n}{\sqrt{D}}$$

where

h_n = height in feet above the base to level n, the highest level.

D = the dimension of the structure, in feet, in a direction parallel to the applied forces

Then,

D = 250/12 = 20.83 ft

h_n = 480/12 = 40 ft

$$T = \frac{0.05 \cdot 40.00}{\sqrt{20.83}} = 0.44 \text{ sec}$$

$$C = \frac{1}{15\sqrt{0.44}} = 0.10 < 0.12$$

The value of S (numerical coefficient for site-structure resonance) will be considered as equal to 1.5 , based on the fact that there is no substantiated geotechnical data for determining the characteristic site period T_s.

The product of $C \cdot S$ need not exceed 0.14.

$C \cdot S = 0.10 \cdot 1.50 = 0.15 > 0.14$

$C \cdot S$ will be considered to be equal to 0.14.

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The value of I (occupancy importance factor) is indicated in Table No. 23-K(UBC Code).

$I = 1.25$

Z = numerical coefficient dependent upon the zone as determined by Figures 1, 2 and 3 in the UBC Code (Chapter 23).

Z will be considered as equal 1.0 (Zone 4).

W = the total dead load

$W = 0.2 \cdot 250 \cdot 3 = 150$ kips

Then, the total lateral seismic load is:

$V = 1.0 \cdot 1.25 \cdot 0.67 \cdot 0.14 \cdot 150 = 17.59$ kips

The force V is distributed over the height of the structure in accordance with

$$V = F_t + \sum_{i=1}^{n} F_i$$

where

F_t = lateral concentrated load at the top of the frame;

F_i = lateral concentrated load at level "i".

The concentrated load at the top is determined according to the following formula:

$$F_t = 0.07 \cdot T \cdot V$$

(F_t shall not exceed 0.25 V and may be considered as 0 where T is 0.7 sec or less.) The remaining portion of V is distributed over the height of the structure according to the following formula:

$$F_x = \frac{(V - F_t) \cdot w_x \cdot h_x}{\sum_{i=1}^{n} w_i \cdot h_i}$$

$h_{x,i}$= height above the base to the level x or i, respectively,

$w_{i,x}$= portion of W which is located at level i or x respectively.

Then,

$T = 0.44 < 0.70$ $\qquad F_t = 0.0$

h_1= 160 in

h_2= 320 in

h_3= 480 in

$w_1 = w_2 = w_3 = 0.33$

$$F_1 = \frac{17.59 \cdot 0.33 \cdot 480}{0.33 \cdot (160+320+480)} = 8.79 \text{ kips}$$

F_2 = 5.85 kips

F_3 = 2.93 kips

Corresponding to the load factors for seismic loads the lateral loads at the floor levels are calculated as:

$F_{1U} = 0.75 \cdot 1.7 \cdot 1.1 \cdot 8.79 = 12.33$ kips

F_{2U} = 8.23 kips

F_{3U} = 4.10 kips

5.1.3 Linear Analysis

Using the loads as given by Section 5.1.2 the structure is given a linear static analysis. The results needed for the design of the controlling reinforced members are listed below:

a) Dead Load + Live Load

- beam 2: M_{Umax} = 2061.1 kips.in
- column 4: M_U = 419.22 kips.in; N_U = 148.88 kips ;

  V_U = 7.81 kips
- column 6: M_U = 1147.0 kips.in; N_U = 90.75 kips

  V_U = 13.86 kips
- column 8: M_U = 991.36 kips.in; N_U = 32.63 kips

  V_U = 12.41 kips

in which M_U, V_U and N_U denote moment, shear and axial force respectively.

b) Dead Load + Live Load + Seismic Load

- beam 3: M_{Umax} = 3504.2 kips.in ; V_{Umax} = 70.35 kips
- column 4: M_U = 1646.8 kips.in; N_U = 175.87 kips

$V_U = 20.13$ kips

- column 6: $M_U = 1930.9$ kips.in; $N_U = 105.53$ kips

  $V_U = 24.13$ kips

- column 8: $M_U = 1383.5$ kips.in; $N_U = 37.38$ kips

  $V_U = 18.57$ kips

5.1.4 Design of Beams and Columns

Using conventional reinforced concrete theory and following the ACI Code for designing beams and columns subjected to the moments and axial loads given above, the following percentages of steel reinforcement were determined :

a) for the beams:

- longitudinal reinforcement = 0.80 %
- transverse reinforcement(confinement) = 0.80 %

b) for the columns:

- longitudinal reinforcement in one face = 0.50 %
- transverse reinforcement(confinement) = 0.80 %

The percentage of transverse reinforcement(confinement) was calculated from the amount of shear reinforcement needed for the loads defined.

5.2 Nonlinear Dynamic Analysis

With the steel reinforcements determined, the frame as shown in Fig. (5.1) can now be analyzed, using MC-QUAKE program. Assume that the first 12 seconds of the N-S 1940 El Centro earthquake has been chosen as the maximum credible earthquake(the ground motion for the "proof test" of the structure). (It has a peak acceleration of approximately 0.3g, as shown in the plot output for ground acceleration record, Appendix B.)

The results of the analysis are given in the Appendix B as was first presented in Chapter 3. The inelastic responses are as follows:

a) maximum damage factor at member 5 = 0.45;

b) first yield displacement = 3.12 in;

c) max. displacement at the top = 4.13 in;

d) displacement ductility of the frame = 1.33.

For this example , only moderate damage has occurred (damage factor = 0.45 < 0.50). The structure designed has performed well when subjected to the test ground motion chosen.

5.3 Damage - Redesign of the frame

In order to illustrate the redesign process, assume the test ground motion is the same as the preceding one except that

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that the amplitudes are amplified by a factor of 1.4. The computed inelastic responses obtained by use of MC-QUAKE are:

a) maximum damage factor at member 5 = 0.98;

b) first yield displacement = 2.93 in;

c) max. displacement at the top = 5.76 in;

d) displacement ductility = 1.97.

The damage factor in member 5 being greater than 0.50 (high damage), the frame needs to be redesigned. In order to simplify the presentation of the example, it is assumed that the steel percentages of the members are the only design variables to be modified.

5.3.1 Displacement Ductility of the Frame

Commentary on the SEAOC Code [38] indicates that the displacement ductility factor required in design should range from 2 to 5. For this example a displacement ductility of the frame equal 2.25 is chosen(the maximum value). For a three-story frame the following curvature ductility factors are obtained from Fig. (4.22):

for beams = 30,

for columns = 13.5.

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5.3.2 Curvature Ductility Diagrams

For the combination of material properties, f'_c = 3 ksi and f_y = 60 ksi and

a) for the beams, $P/f'_c bh$ = 0.0 Fig.(4.11) for a curvature ductility of 30 and a confinement steel of 0.8 % which is the same as before, a new value of the longitudinal reinforcement of 1.0 % (versus 0.8 % previously) is indicated.

b) for the columns $P/f'_c bh$ = 0.10 Fig.(4.12) for a curvature ductility of 13.5 and a confinement steel of 0.8% which is also unchanged. Similarly a new value of the longitudinal reinforcement of 2.25 % (versus 0.5 % previously) is indicated.

Using these new values for reinforcement of the members, another nonlinear dynamic analysis is performed with the following results:

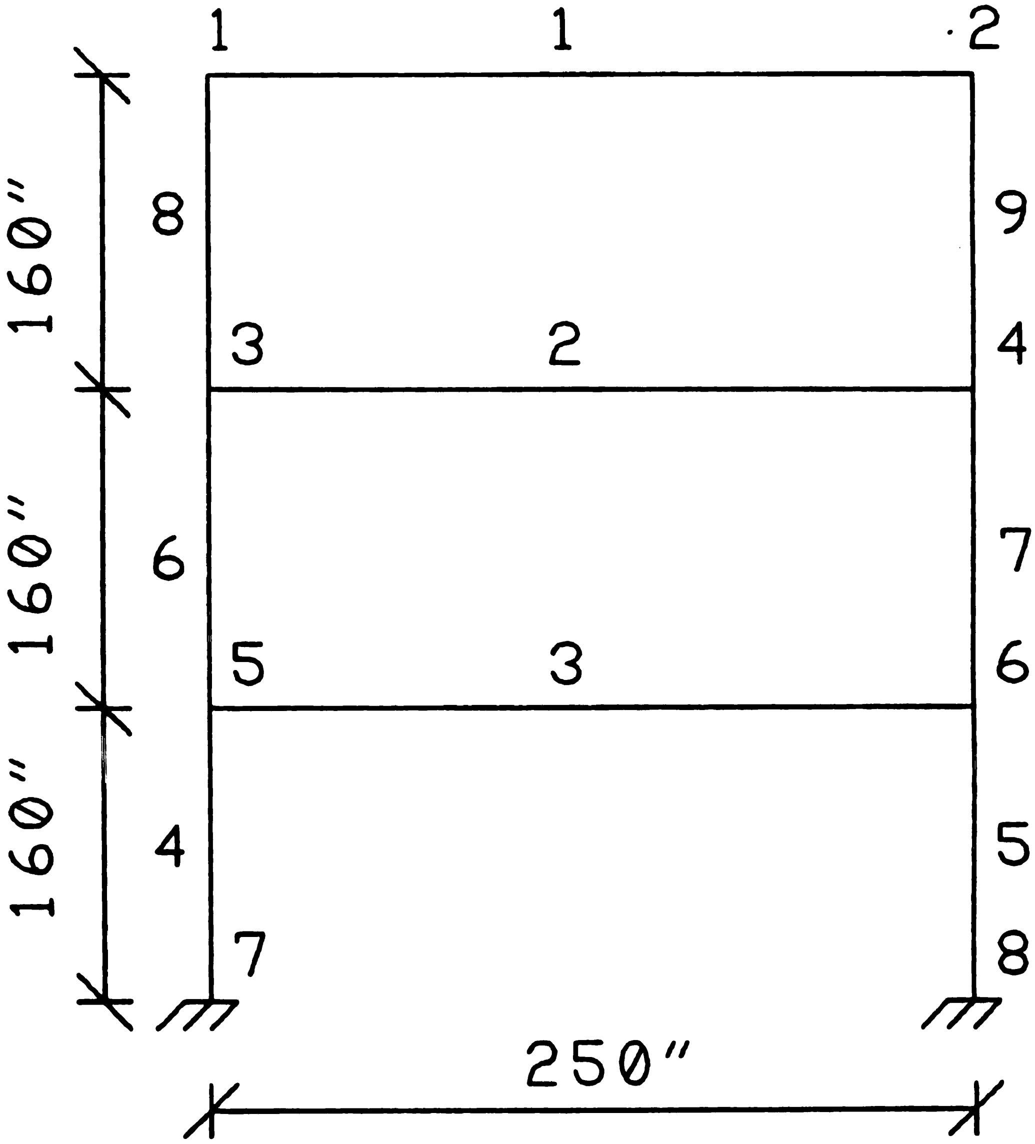
a) maximum damage factor at member 5 = 0.35;

b) first yield displacement = 2.28 in;

c) max. displacement at the top = 2.92 in;

d) displacement ductility = 1.28.

Since the damage is only "moderate" (damage factor = 0.35 < 0.50) under the maximum credible earthquake, the redesign may be regarded as satisfactory.



beams: 10"/30"
columns: 20"/20"

Fig.(5.1) - Example

CHAPTER 6

SUMMARY AND CONCLUSION

6.1 Summary

The behavior of reinforced concrete frames during earthquakes is complex and influenced by a variety of material and structural parameters, in addition to the ground motion characteristics. Considerations of economics require the composite material be used beyond their linearly elastic ranges. Thus a reasonably accurate analysis needs to consider the nonlinear behavior of the structure. Although nonlinear analysis computer programs are available, they are rarely used for design of ordinary buildings because they are too costly to run.

The purpose of this study was to present a design procedure for reinforced concrete frames subjected to strong earthquakes based on a microcomputer program for nonlinear dynamic analysis. The design procedure consisted of four basic steps:

a) preliminary design based on member forces obtained by a linear elastic analysis for equivalent static loading as prescribed by design codes, e.g., the UBC Code [21];

b) nonlinear dynamic analysis;

c) checking of structural damage;

d) if necessary, redesign of the structure and repeat of the design cycle.

The design procedure was described in Chapter 2 and illustrated by a numerical example in Chapter 5.

The establishment of this design procedure required development of the following:

a) a microcomputer program for the nonlinear dynamic analysis of reinforced concrete frames subject to strong earthquakes, as presented in Chapter 3;

b) design aids to be used in the redesign of the structure if it reached a damage factor, as defined in Chapter 1, greater than 0.5.

The microcomputer program (code name: MC-QUAKE) is composed of three sub-programs:

a) Sub-program DATA - for input data;

b) Sub-program MAIN - for nonlinear dynamic analysis;

c) Sub-program PLOT - for plotting input and output results.

Sub-program MAIN is an adaptation of Program NACRS-II [28] prepared for use on mainframe computers.

The design aids provided herein are to give guidance for redesign of the structure in order to avoid taking too many trials in order to arrive at an appropriate design. In the first group, Figures (4.1) to (4.20), diagrams are constructed that give values of curvature ductility of a section as a function of the longitudinal reinforcement percentage and the confinement steel percentage. In the second group, Figures (4.22) to (4.29), the relationship between the displacement ductility of a structure , μ_D, and curvature ductility of a section, μ_ϕ, was determined for reinforced concrete frames of one-bay ranging from 3 to 10 stories subjected to the 1940 El Centro (NS) earthquake.

6.2 Concluding Remarks

This study has demonstrated that microcomputers can make available to the designer of "ordinary" reinforced concrete structures highly sophisticated tools of analysis that has heretofore been limited, because of economic reasons, to "special" structures. This study has also attempted to extend the use of the analytical tool for the design procedure by introducing a "proof test" step in the process.

A key point in the design procedure is the choice of the "proof test" ground motion. Several questions may be raised such as: what type of motion (El Centro , Taft, or generated artificial earthquakes), what amplification factor if different than 1.0, what duration of the ground motion. Of course, such questions are also faced now by designers of "special" structures. The kinds of considerations that may be needed for choice of test ground motion have been pointed out in Section 2.5 - Selection of Design Earthquake. Obviously some standardization at the local or regional level (microzonation) would be highly desirable. This is perhaps the most important "link" that needs to be strengthened in the proposed design procedure. Studies of this type need a synthesis of expertise from seismology, geology, geotechnical and structural engineering.

The first group of design aids are based on material properties and structural mechanics, and therefore are "exact". Their usefulness as design aids are direct in limiting the choices of variations of parameters in the design procedure. As regards the second group, they were based on only one-bay frames and one ground motion(1940 El Centro NS). Their usefulness is limited to providing a reference or a rough guide for choosing the curvature ductilities. Of course the "soundness" of a design will always be checked by the analysis.

From the structural engineering point of view, further studies may include:

a) shear walls - many realistic reinforced concrete buildings contain shear walls in addition to a ductile moment frame. It is not appropriate to model such structural walls using the mathematical model presented in Appendix A. Therefore, it would be very useful if a reinforced concrete shear wall element, which could accurately simulate the behavior under seismic loads, be developed and included in the microcomputer program.

b) variation of axial force - in the present analysis, member axial forces were considered to remain constant throughout the response, leading to a constant geometric stiffness matrix to approximate the P-Δ effect. A more realistic analysis that considers the effects of variable axial forces would enable the analysis to include the possibly important effects of vertical ground motion.

c) the structures considered herein were limited to plane frames. Although they have a wide range of applicability, there are circumstances for example, asymmetric structural framing, for which the three dimensional space behavior beyond the linear range of the members needs to be considered. This would be a most challenging task for further research.

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APPENDIX A

THE MATHEMATICAL MODEL

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THE MATHEMATICAL MODEL

A.1 Introduction

The purpose of this appendix is to present the essential features of the mathematical model for the inelastichysteretic behavior of reinforced concrete members used in the analysis program. As mentioned previously the model was that proposed by Meyer and Roufaiel[28] which is a modified version of the Takeda model. The modification includes accounts of the effects of the finite size of the plastic regions as well as the axial and shear forces. The accuracy of the model was demonstrated in [28] by analyzing numerous members and structures. The results showed that the model is very effective in predicting the cyclic nonlinear behavior of reinforced concrete frame members. The model requires the specification of only a few material properties of steel and concrete in order to simulate the complete behavior under cyclic loads.

A. 2 Material Constitutive Laws

A. 2. 1 Reinforced Steel

The stress-strain curve for steel is shown in Fig. (A. 1). The bilinear curve consists of an elastic part where

$$E_s = \frac{f_{sy}}{\epsilon_{sy}} \tag{A. 1}$$

and an inelastic part with strain hardening ratio

$$p_s = \frac{f_{su} - f_{sy}}{\epsilon_{su} - \epsilon_{sy}} \cdot \frac{1}{E_s} \tag{A. 2}$$

where f_{sy} and ϵ_{sy} are the yield stress and strain respectively, f_{su} is the ultimate stress, and ϵ_{su} is the strain at ultimate stress.

A. 2. 2 Concrete

The proposed stress-strain curve for confined concrete (confined by transverse or hoop steel), Fig. (A. 2) may be completely determined by specifying the following three parameters:

1) uniaxial strength of plain (unconfined) concrete, f_c';

2) strain at f'_c, ϵ_0;

3) volumetric ratio of confinement steel given by

$$\rho'' = \frac{2(b'' + d'')}{b'' \cdot d'' \cdot s} A''_s \qquad (A.3)$$

where b'' and d'' are the width and depth of the confined core, A''_s is the hoop steel cross-sectional area, and s is the hoop steel spacing, Fig. (A.3). Initially A''_s is equal to the shear reinforcement (stirrups) determined for the member in the preliminary design phase.

Referring to Fig. (A.2), the parameters that define the stress-strain curve are related to the preceding three basic parameters as follows:

$$f_{cu} = \alpha_c \, f'_c \qquad (A.4)$$

$$\epsilon_{cu} = \alpha_c \, \epsilon_0 \qquad (A.5)$$

$$f_{cy} = \frac{2}{3} \, f_{cu} \qquad (A.6)$$

$$\epsilon_{cy} = \frac{1}{3} \, \epsilon_{cu} \qquad (A.7)$$

$$\epsilon_m = \beta_c \, \epsilon_{cu} \qquad (A.8)$$

where α_c and β_c are factors which account for the confinement steel effect on concrete strength and ultimate strain, respectively. These factors are determined from the empirical expressions,

$$\alpha_c = 1 + 10\ \rho'' \tag{A.9}$$

$$\beta_c = 2 + 600\ \rho'' \tag{A.10}$$

The cut-off point of the unloading branch, $0.2\ f_{cu}$ is chosen empirically.

A.3 Primary Moment-Curvature Relationship

The primary moment-curvature relationship of a beam section can be derived using conventional reinforced theory only. A typical primary moment-curvature curve is shown in Fig. (A.4). The bending moment M_m is associated with the curvature ϕ_m at which the concrete strain reaches the crushing strain ϵ_m. Thus, the primary moment-curvature relationship is valid up to the crushing of the concrete cover at which point the section is regarded as having failed.

A.4 Hysteretic Behavior of Reinforced Concrete

Under load reversals, the stiffness of a reinforced concrete member changes due to the cracking of the concrete and debonding effects at the steel-concrete interface. In the computer program, a modified Takeda-type model is used to represent hysteretic moment-curvature relationships, Fig. (A. 5).

The model has five basic branch types.

1- Elastic loading and unloading;

2- Inelastic loading;

3- Inelastic unloading;

4- Inelastic reloading during closing of cracks in which the "pinching effect" is considered. (to be described in section A. 5)

5- Inelastic reloading after closing of cracks.

In branches 1 and 2 the stiffnesses are:

$(EI)_1 = (EI)_e$ (elastic range)

and

$(EI)_2 = p\ (EI)_e$

In order to compute $(EI)_3$ for branch 3 define,

M_1^+, ϕ_1^+ - maximum moment and curvature reached in the positive direction during the current load cycle.

M_x^-, ϕ_x^- - maximum moment and curvature reached in the previous load cycles in the negative direction.

It follows that,

$$M_0 = \frac{p}{1 - p} \cdot (\phi_1^+ (EI)_1 - M_1^+) \qquad (A. 11)$$

$$\phi_0 = \frac{1}{1 - p} \cdot (\phi_1^+ - \frac{M_1^+}{(EI)_1}) \qquad (A. 12)$$

in which the point (M_0, ϕ_0) is on a line drawn from (M_1^+, ϕ_1^+) parallel to branch 1.

$$\overline{EI} = \frac{M_x^- - M_0}{\phi_x^- - \phi_0} \qquad \text{(A. 13)}$$

$$\phi_r^+ = \phi_0 - \frac{M_0}{(\overline{EI})} \qquad \text{(A. 14)}$$

$$(EI)_3 = \frac{M_1^+}{\phi_1^+ - \phi_r^+} \qquad \text{(A. 15)}$$

The definitions of $(EI)_4$ and $(EI)_5$ are based on the pinching effect as described in the next section.

A. 5 Shear Effect on Hysteretic Behavior

In a reversed loading cycle, previously opened cracks tend to close, leading to an increase in stiffness and the characteristic "pinched" shape of the moment-curvature curve. Experimental work indicated that when the maximum nominal shear stress, v_{max}, is less than 3.5 $\sqrt{f'_c}$ the hysteresis loops would be stable. When v_{max} is greater than 3.5 $\sqrt{f'_c}$, stiffness degradation would result in "pinched" load-deformation curves.

In order to include this effect in the mathematical model, the stiffness of a section is assumed to be represented by two straight line segments, Fig. (A. 6), where:

$(0, \phi_r)$ - point of reloading

(M_p, ϕ_p) - "pinching point"

(M_x, ϕ_x) - point of resumed inelastic loading

(M_n, ϕ_n) - point of no "pinching"

The line segment 4 represents reloading before the closing of shear cracks. The line segment 5 represents reloading after the closing of shear cracks.

From Fig. (A.6), we have:

$$\phi_n = \phi_r \frac{\overline{EI}}{(\overline{EI}) - (EI)_e} \qquad (A.16)$$

$$M_n = (EI)_e \cdot \phi_n \qquad (A.17)$$

where $(\overline{EI}) = \dfrac{M_x}{\phi_x - \phi_r}$

Introducing a pinching factor, α_p, which represents the effect of shear stress on the hysteresis behavior and writing,

$$M_p = \alpha_p \cdot M_n \qquad (A.18)$$

$$\phi_p = \alpha_p \cdot \phi_n \qquad (A.19)$$

with the empirical equation,

$$\alpha_p = 0.4 \frac{a}{d} - 0.6 \qquad (A.20)$$

where

$$\alpha_p = 0 \text{ if } \frac{a}{d} < 1.5$$

$$\alpha_p = 1 \text{ if } \frac{a}{d} > 4$$

a : shear span(in a typical building frame, $a = L/2$.)

d : effective depth of the beam

the pinching effect is now completely defined.

A.6 Tangent Stiffness Matrix

To compute the tangent stiffness of a general frame member, the element is subdivided into three regions, Fig. (A.7):

a) an inelastic region of length x_i at node i, having the average stiffness $(\overline{EI})_i$;

b) an inelastic region of length x_j at node j, having the average stiffness $(\overline{EI})_j$;

c) a central region of length $(L - x_i - x_j)$, having the initial elastic stiffness $(EI)_e$.

For the six planar degrees of freedom identified in Fig. (A.7), the tangent stiffness of this frame element can be written as

$$[K_e] = \begin{bmatrix} k_{11} & 0 & 0 & k_{14} & 0 & 0 \\ & k_{22} & k_{23} & 0 & k_{25} & k_{26} \\ & & k_{33} & 0 & k_{35} & k_{36} \\ \text{symm.} & & & k_{44} & 0 & 0 \\ & & & & k_{55} & k_{56} \\ & & & & & k_{66} \end{bmatrix}$$

the coefficients

$$k_{11} = k_{44} = -k_{14} = \frac{EA}{L}$$

are assumed to remain constant. k_{33}, k_{36}, k_{66} are obtained from their flexibility, which can be computed by integrating the moment-curvature expression over the entire length of the member.

Defining

$$Q_i = \frac{(EI)_e}{(\overline{EI})_i} \qquad \text{(A.21)}$$

$$Q_j = \frac{(EI)_e}{(\overline{EI})_j} \qquad \text{(A.22)}$$

the stiffness ratios for the end of the regions i and j, the flexibility coefficients are given by

$$f_{ii} = \frac{1}{3(EI)_e L^2}[(Q_j-1)\ x_j^3 - (Q_i-1)\ (L-x_i)^3 + Q_i L^3] \quad (A.23)$$

$$f_{jj} = \frac{1}{3(EI)_e L^2}[(Q_i-1)\ x_i^3 - (Q_j-1)\ (L-x_j)^3 + Q_j L^3] \quad (A.24)$$

$$f_{ij} = \frac{1}{3(EI)_e L^2}[(Q_j-1)\ x_j^2\ (1.5\ L - x_j) +$$

$$+ (Q_i-1)\ x_i^2\ (1.5\ L - x_i) + L^3/2] \quad (A.25)$$

Then for stiffness coefficients,

$$k_{33} = f_{jj} / (f_{ii} f_{jj} - f_{ij}^2) \quad (A.26)$$

$$k_{66} = f_{ii} / (f_{ii} f_{jj} - f_{ij}^2) \quad (A.27)$$

$$k_{36} = - f_{ij} / (f_{ii} f_{jj} - f_{ij}^2) \quad (A.28)$$

and the remaining coefficients follow from statics,

$$k_{23} = - k_{35} = (k_{33} + k_{36}) / L \quad (A.29)$$

$$k_{26} = - k_{56} = (k_{36} + k_{66}) / L \quad (A.30)$$

$$k_{22} = k_{55} = - k_{25} = (k_{33} + 2\ k_{36} + k_{66})/L^2 \quad (A.31)$$

The length x_i and stiffness ratio Q_i of the plastic region at node i depend on the current branch of the moment-curvature diagram.

For the initial elastic loading and unloading the member end which has not experienced any inelasticity, x_i is taken as zero and Q_i equal to one. For inelastic loading the length of the plastic region is determined by

$$x_i = \frac{M_i - M_y}{M_i + M_j} \cdot L \qquad (A.32)$$

assuming that moments vary linearly along the beam length. For inelastic unloading, x_i will be assumed to remain the maximum plastic region length reached in all the previous inelastic loading cycles.

A.7 Axial Load Effect

The effect of gravity loads on story shear and moment through the lateral story displacement was considered in the program. This phenomenon, known as the P-Δ effect, is considered using the usual geometric stiffness matrix for the beam member. As mentioned previously, the axial load is assumed to remain constant and equal to the gravity load effect present at the beginning of the ground motion.

A.8 Strength Degradation During Cyclic Loading

Under load reversals, not only the stiffness of a reinforced concrete member decreases, but also its strength deteriorates if the member is strained beyond a certain critical load level. Then, the maximum load required to deform the member to a given level of deformation decreases. Strength degradation and the corresponding critical load are functions of several variables, such as the degree of confinement and the value of axial force.

Experiments have indicated that [28] the strength degradation commences when the concrete cover reaches the failure strain ϵ_m, as defined in Equation (A.8). The terminal point (M_m, ϕ_m) of the moment-curvature curve, Fig. (A.4), had been defined as the point associated with the curvature at which the concrete strain reaches the value ϵ_m. It is also observed that once the concrete strain ϵ_m has been exceeded members would fail after only a very few cycles. Therefore, for practical purposes, the curvature associated with ϵ_m is defined as the point of failure. That is, the small amount of energy that would be dissipated in those few cycles is ignored in the model.

A.9 Damage Factor

The damage factor is defined as

$$\text{damage factor} = \frac{M_m}{M_x} \cdot \frac{\phi_x}{\phi_m} \qquad \text{for } M_x > M_y$$

$$= 0 \qquad \text{for } M_x < M_y$$

where M_y is the yield moment of the member section (Fig. (A.8)). This factor represents the ratio between the secant stiffness at the point of failure, and the minimum secant stiffness, reached during the response. The value of damage factor equal to zero indicates that deformations have remained elastic and there is no damage. The value of damage factor equal to 1.0 indicates that the member has been loaded to the point of failure.

A.10 Equations of Motions

The equation of motion of a structure can be written in the matrix form as:

$$M\ \Omega\ddot{U} + C\ \Omega\dot{U} + K\ \Omega U = -\ M\ \Omega\ddot{X}_g \qquad \text{(A.33)}$$

where M, C, K are respectively, the structure mass, damping, and tangent stiffness matrices. The symbol Ω is used here to denote increment instead of the usual symbol . Thus, ΩU, $\Omega\dot{U}$, $\Omega\ddot{U}$ are the incremental nodal displacement, velocity and acceleration vectors relative to the ground, and $\Omega\ddot{X}_g$ is the incremental ground acceleration.

Using the usual assembly procedure, the structure stiffness matrix is assembled from the element stiffness matrices described in previous sections. The numerical integration technique of solving the equation of motion used was Newmark's Beta method for " average acceleration (Beta=1/4)". It is unconditionally stable for linear systems and becomes unstable only when overly large time steps are used for the analysis of nonlinear problems. The incremental velocities and displacements over a short time step are calculated from the following equations,

$$\Omega\dot{U} = \ddot{U}_n\ \Omega t + 1/2\ \Omega\ddot{U}\ \Omega t \qquad (A.34)$$

$$\Omega U = \dot{U}_n\ \Omega t + 1/2\ \ddot{U}_n(\Omega t)^2 + 1/4\ (\Omega t)^2\Omega\ddot{U} \qquad (A.35)$$

in which ΩU, $\Omega\dot{U}$, $\Omega\ddot{U}$ are change of horizontal displacement, velocity, and acceleration vectors relative to the ground motion between time step "n" and "n+1" and $\dot{U}_n$, $\ddot{U}_n$ are velocity and acceleration vectors relative to the ground motion at the end of step "n".

Equation (A.35) can be solved to calculate the corresponding incremental acceleration:

$$\Omega\ddot{U} = \frac{4}{(\Omega t)^2}\cdot[\Omega U - \dot{U}_n\ \Omega t - 1/2\ \ddot{U}_n\ (\Omega t)^2] \qquad (A.36)$$

Substituting, equation (A.36) into Equation (A.34) gives:

$$\Omega\dot{U} = \frac{2}{\Omega t}\cdot[\Omega U - \dot{U}_n\ \Omega t + 3/2\ \ddot{U}_n\ (\Omega t)^2 \qquad (A.37)$$

Substituting, equation (A.36) and Equation (A.37) into the equation of motion and re-arranging the terms yields:

$$\{\ \Omega U\ \} = [\ A\ ]^{-1}\ \{\ B\ \} \qquad (A.38)$$

in which,

$$A = \frac{4\ M}{(\Omega t)^2} + \frac{2\ C}{\Omega t} + K \qquad (A.39)$$

and

$$B = M\ (\frac{4}{\Omega t}\cdot\dot{U}_n + 2\ \ddot{U}_n - \Omega\ddot{X}_g) +$$

$$+ 2\ C\ (\dot{U}_n - 3/2\ \ddot{U}_n\ \Omega t) \qquad (A.40)$$

From Equation (A. 38) the incremental displacement can be obtained. Then the corresponding incremental velocity and acceleration vectors are given by Equation (A. 37) and Equation (A. 36).

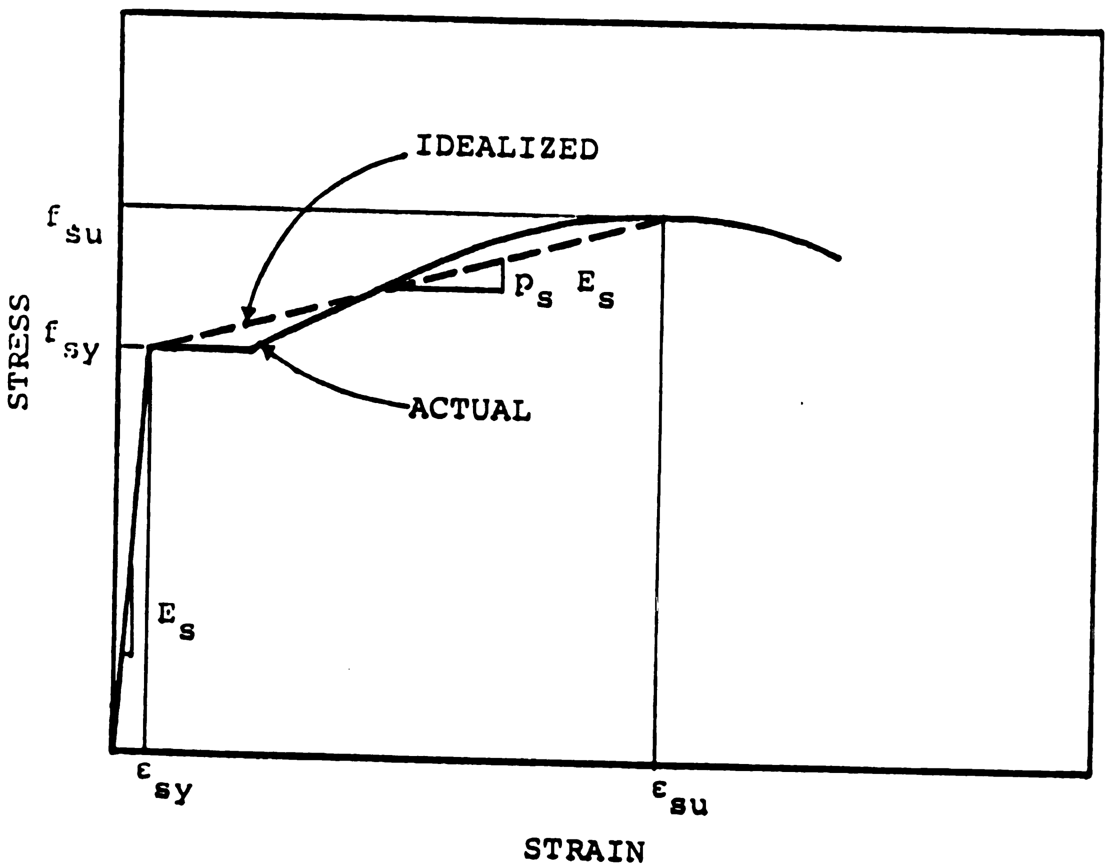

Fig. (A. 1) - Steel stress-strain curve

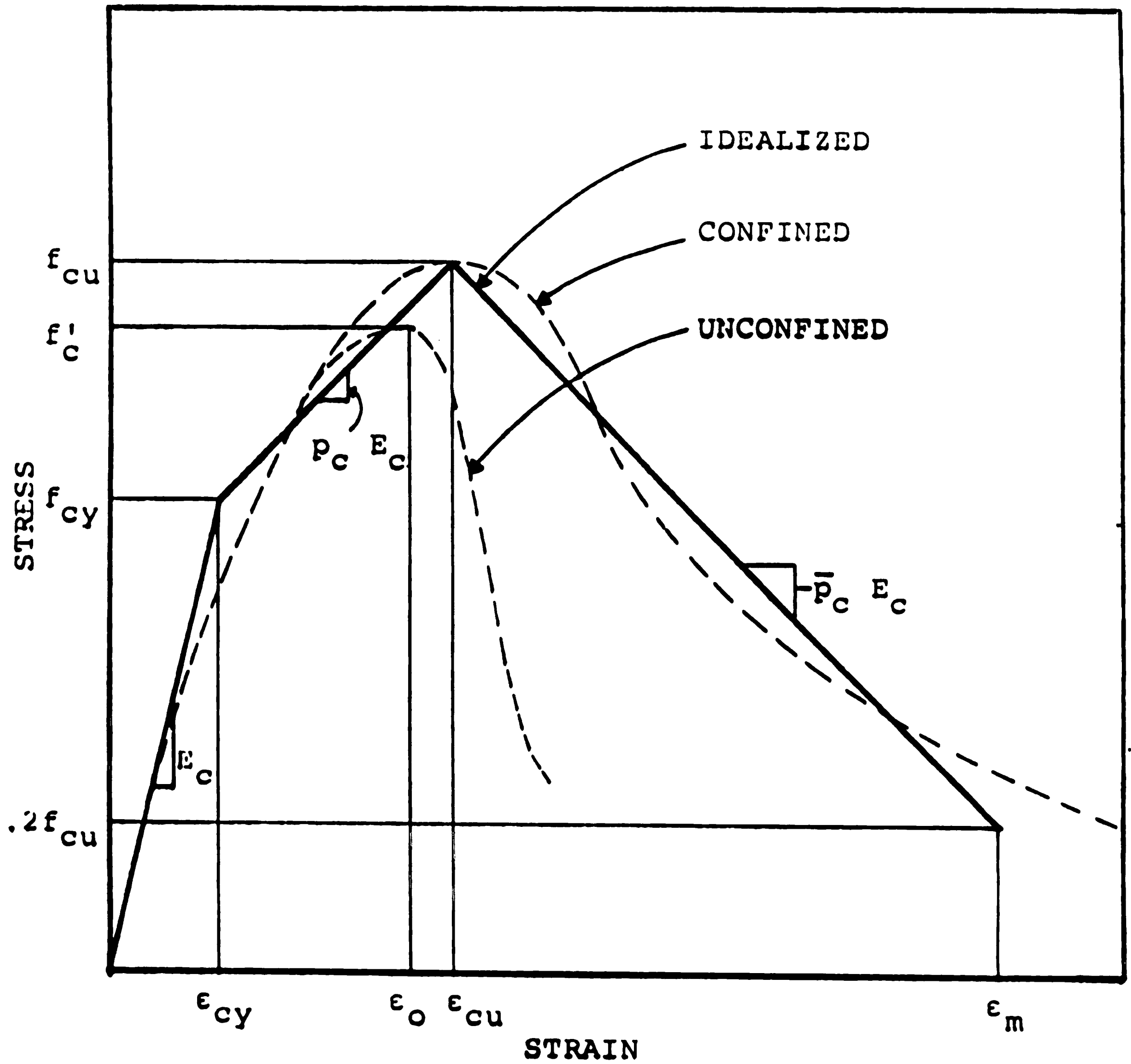

Fig. (A. 2)- Adopted stress-strain curve for concrete

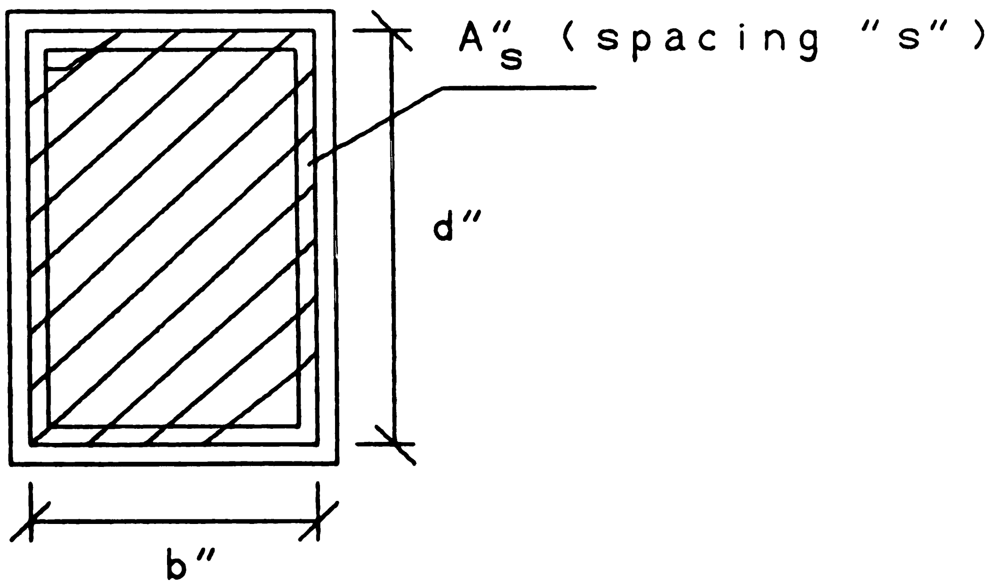

Fig.(A.3) - Cross-Sectional Confined Area

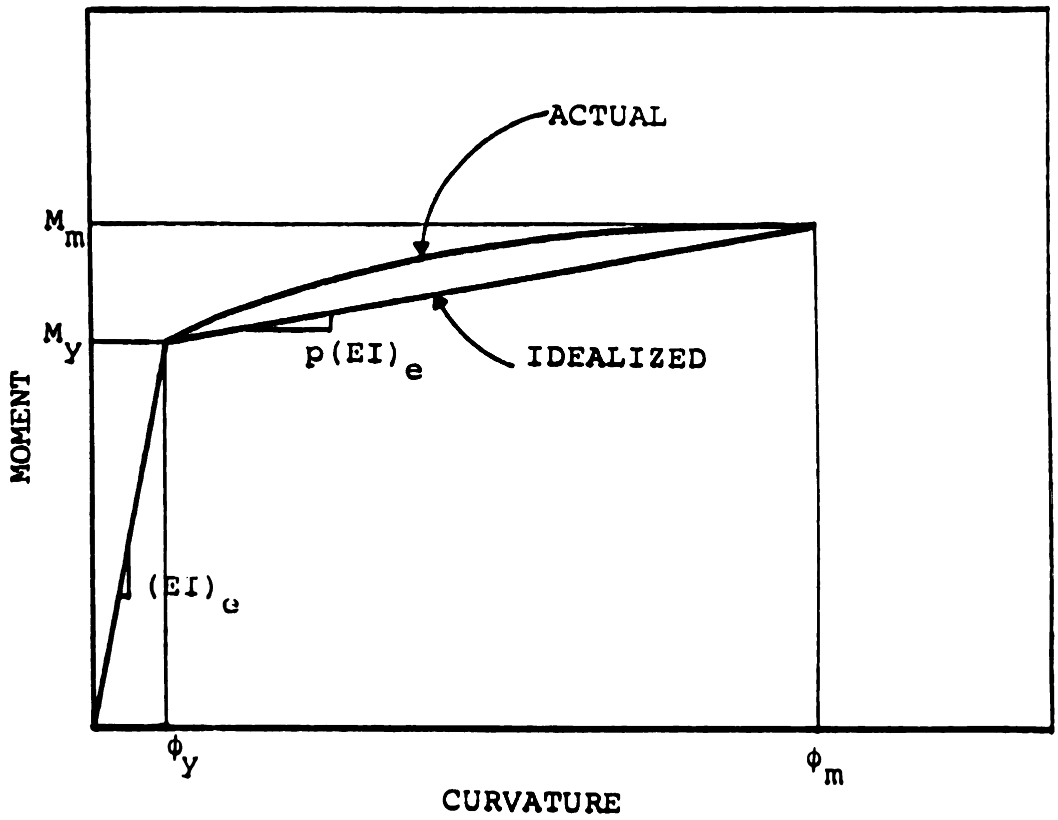

Fig. (A. 4) - Primary moment-curvature relationship for a beam section

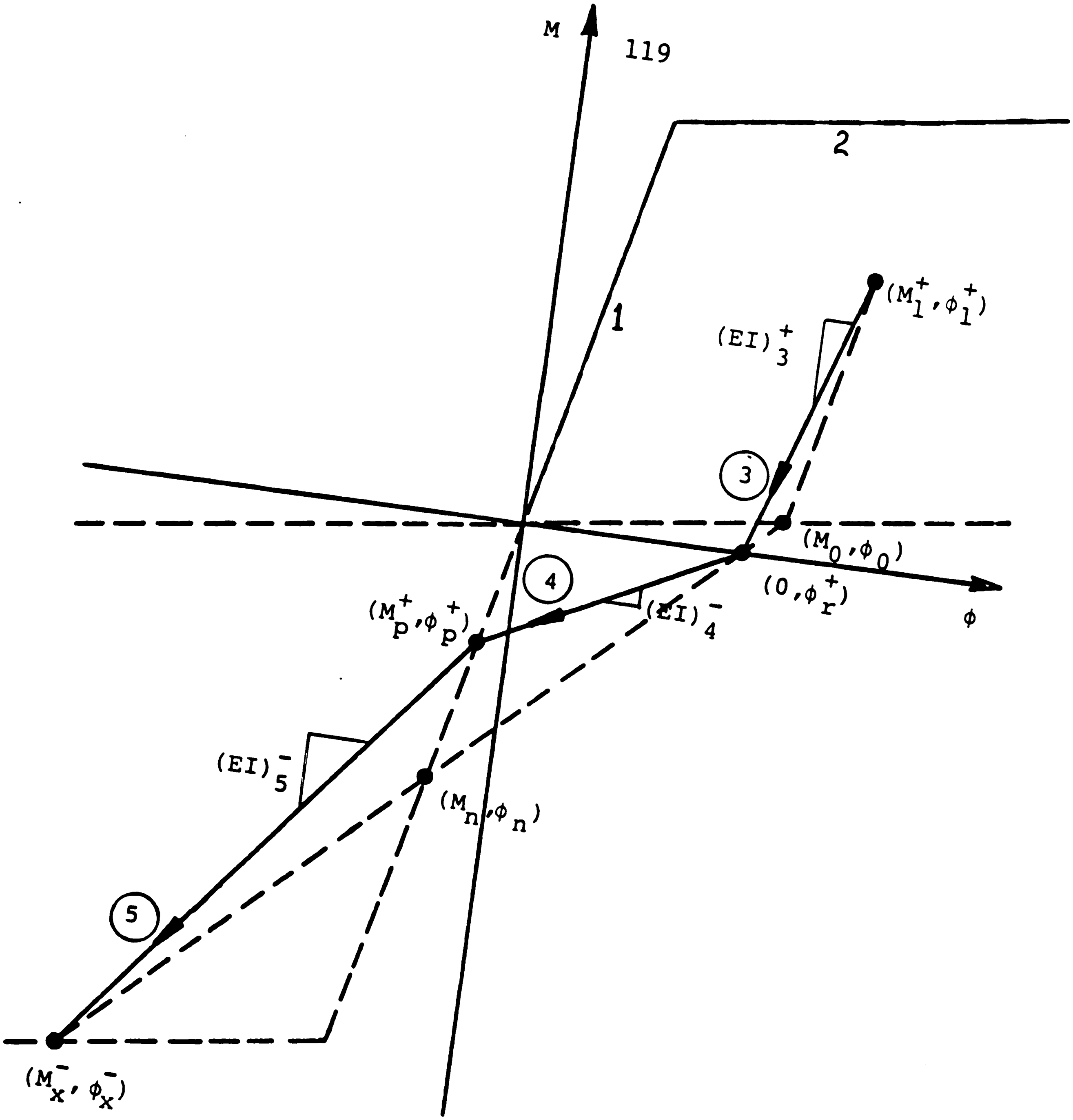

Fig. (A.5) - Modified Takeda-type hysteretic moment-curvature

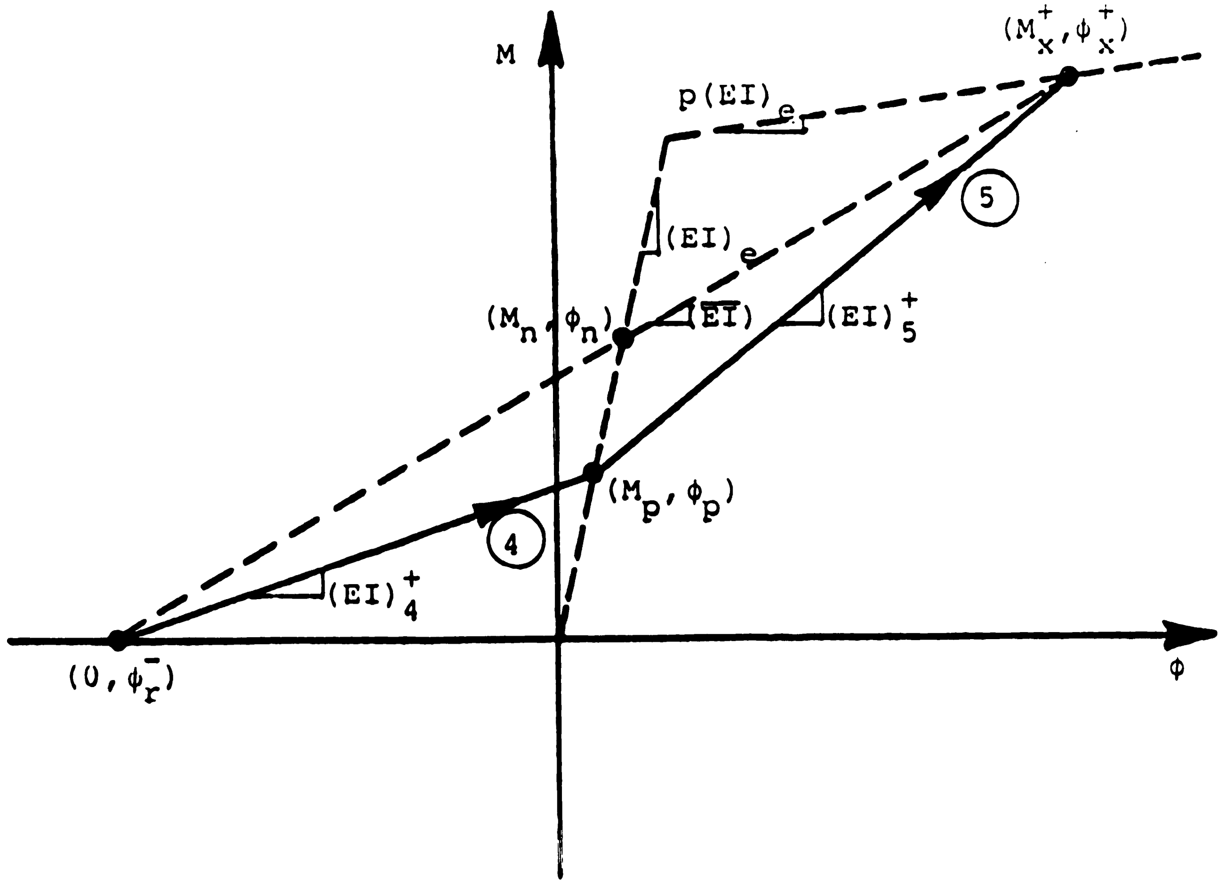

Fig. (A.6) - Pinched reloading curve

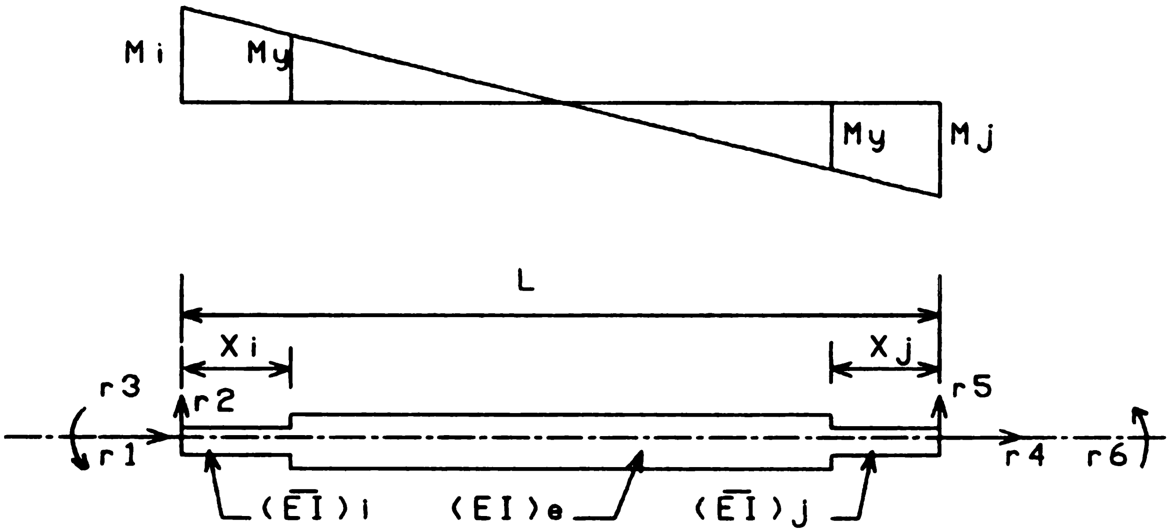

Fig.(A.7)- Frame Member Model

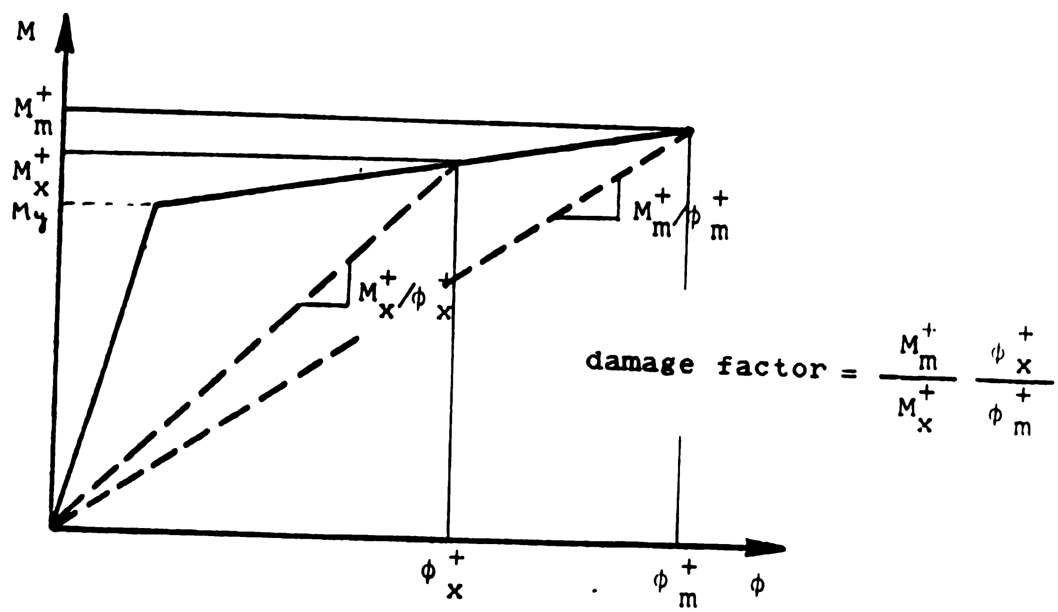

Fig. (A. 8) - Damage Factor

APPENDIX B

EXAMPLE PROBLEM OF DYNAMIC ANALYSIS

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APPENDIX B

EXAMPLE PROBLEM OF DYNAMIC ANALYSIS

A three-story one-bay frame was analyzed by the program. The accelerations values of the first 12 secs of the 1940 El Centro (NS) record were amplified by a factor of one, and used as the base motion of the structure. The analysis was carried out at every 0.02 sec. The total execution time was 28 minutes on an IBM Personal Computer.

INPUT DATA

```
****************************************************
*                                                  *
*    NONLINEAR ANALYSIS OF REINFORCED CONCRETE     *
*                                                  *
*                   STRUCTURES                     *
*                                                  *
*                DYNAMIC ANALYSIS                  *
*                                                  *
****************************************************
*               INPUT DATA PROGRAM                 *
****************************************************
*                       by                         *
*               Aecio Freitas Lira                 *
****************************************************


 NOTE FOR CORRECTING DATA:
 ************************

 (0) NEW DATA : TYPE THE NUMBER IN BETWEEN [ ]
 (1) CORRECT DATA : TYPE THE VARIABLE NUMBER
                    IN BETWEEN ( ) : USE OPTION
                    2 : LIST DATA TO OBTAIN
                    THE VARIABLE NUMBER.
 **************************************************
```

```
   INPUT DATA FILE NAME =EXAMPLE



   INPUT DATA OPTION
         0: NEW DATA
         1: CORRECT DATA
         2: LIST DATA
         3: END        =0

TYPE THE TITLE OF THE PROBLEM :  EXAMPLE FOR DISSERTATION



###################
GENERAL INFORMATION
###################


(1)[1] NUMBER OF NODAL POINTS=8

(2)[2] NUMBER OF CONTROL NODES FOR X-Y GENERATION=4

(3)[3] NUMBER OF NODE GENERATION COMMANDS=2

(4)[4] NUMBER OF ZERO DISPLACEMENTS COMMANDS=2

(6)[6] NUMBER OF MASS GENERATION COMMANDS=1

(7)[7] DATA FOR CHECKING ONLY
            0: RUN
            1: CHECK ONLY       =0

ARE ALL THE DATA CORRECT (Y/N) [Y]?:   Y
```

```
************************
CONTROL JOINT COORDINATES
************************


 TOTAL NUMBER OF CONTROL COMMANDS=          4
 ................................

 CONTROL COMMAND NUMBER=          1

( 10)[10] NODE NUMBER = 1

( 11)[11] X-COORDINATE = 0

( 12)[12] Y-COORDINATE = 400

 ARE ALL THE DATA CORRECT(Y/N)  [Y]?:   Y

 CONTROL COMMAND NUMBER=          2

( 13)[10] NODE NUMBER = 2

( 14)[11] X-COORDINATE = 250

( 15)[12] Y-COORDINATE = 400

 ARE ALL THE DATA CORRECT(Y/N)  [Y]?:   Y

 CONTROL COMMAND NUMBER=          3

( 16)[10] NODE NUMBER = 7

( 17)[11] X-COORDINATE = 0

( 18)[12] Y-COORDINATE = 0

 ARE ALL THE DATA CORRECT(Y/N)  [Y]?:   Y

 CONTROL COMMAND NUMBER=          4

( 19)[10] NODE NUMBER = 8

( 20)[11] X-COORDINATE = 250

( 21)[12] Y-COORDINATE = 0

 ARE ALL THE DATA CORRECT(Y/N)  [Y]?:   Y
```

```
************************
NODE GENERATION COMMANDS
************************


 TOTAL NUMBER OF CONTROL COMMANDS=         2
 ..............................

 CONTROL COMMAND NUMBER=         1

( 22)[13] FIRST NODE NUMBER = 1

( 23)[14] LAST NODE NUMBER = 7

( 24)[15] NUMBER OF NODES TO BE GENERATED=2

( 25)[16] DIFFERENCE BETWEEN  TWO SUCESSIVE NODES=2

 ARE ALL THE DATA CORRECT(Y/N)  [Y]?:   Y

 CONTROL COMMAND NUMBER=         2

( 26)[13] FIRST NODE NUMBER = 2

( 27)[14] LAST NODE NUMBER = 8

( 28)[15] NUMBER OF NODES TO BE GENERATED=2

( 29)[16] DIFFERENCE BETWEEN  TWO SUCESSIVE NODES=2

 ARE ALL THE DATA CORRECT(Y/N)  [Y]?:   Y
```

```
***************************
ZERO DISPLACEMENTS COMMANDS
***************************
   1: FOR FIXED D.O.F.
   0: FOR FREE  D.O.F.


TOTAL NUMBER OF CONTROL COMMANDS=          2
................................

CONTROL COMMAND NUMBER=          1


( 30)[17] FIRST NODE IN THE GROUP=1

( 31)[18] CODE FOR X-DISPLACEMENT=0

( 32)[19]  CODE FOR Y-DISPLACEMENT=0

( 33)[20] CODE FOR ROTATION=0

( 34)[21] LAST NODE IN THE GROUP=6

( 35)[22] DIFFERENCE BETWEEN TWO SUCESSIVE NODES=1

ARE ALL THE DATA CORRECT(Y/N)  [Y]?:   Y

CONTROL COMMAND NUMBER=          2


( 36)[17] FIRST NODE IN THE GROUP=7

( 37)[18] CODE FOR X-DISPLACEMENT=1

( 38)[19]  CODE FOR Y-DISPLACEMENT=1

( 39)[20] CODE FOR ROTATION=1

( 40)[21] LAST NODE IN THE GROUP=8

( 41)[22] DIFFERENCE BETWEEN TWO SUCESSIVE NODES=1

ARE ALL THE DATA CORRECT(Y/N)  [Y]?:   Y
```

```
************************
MASS GENERATION COMMANDS
************************


TOTAL NUMBER OF CONTROL COMMANDS=          1
................................

CONTROL COMMAND NUMBER=          1

( 42)[23] FIRST NODE NUMBER IN THE GROUP=1

( 43)[24]  X-MASS=0.065

( 44)[25]  Y-MASS=0.065

( 45)[26] ROTARY INERTIA=0

( 46)[27] LAST NODE NUMBER IN THE GROUP=6

( 47)[28] DIFFERENCE BETWEEN TWO SUCESSIVE NODES=1

ARE ALL THE DATA CORRECT(Y/N)  [Y]?:   Y

*****************
LOAD CONTROL DATA
*****************


INPUT NUMBER OF NODAL STATIC  LOAD COMMANDS=2

FIRST NODE NUMBER=1

FORCE IN THE X-DIR.=0

FORCE IN THE Y-DIR.=-25.0

MOMENT =1067.0

LAST NODE NUMBER=5

DIFF. BETWEEN NODES=2

FIRST NODE NUMBER=2

FORCE IN THE X-DIR.=0

FORCE IN THE Y-DIR.=-25.0

MOMENT =-1067.0

LAST NODE NUMBER=6

DIFF. BETWEEN NODES=2
```

```
( 48)[29] GRAVITY ACCELERATION   =384

( 49)[30] NUMBER OF INTEGRATION STEPS FOR DYNAMIC ANALYSIS =600

( 50)[31] INTEGRATION TIME STEP SIZE=0.02

( 51)[32]  FACTOR TO BE APPLIED TO GROUND ACCELERATION RECORD IN THE HORIZONTAL DIRECTION =1.0

( 52)[33]  FACTOR TO BE APPLIED TO TIME COORDINATE ACCELERATION IN THE X-DIR. =1.0

( 55)[36]  ABS. VALUE OF THE MAX. DISPLACEMENT=30

 ARE ALL THE DATA CORRECT(Y/N)  [Y]?:   Y
```

```
*************************************************
EARTHQUAKE ACCELERATION CONTROL - DAMPING FACTORS
*************************************************


( 56)[37] NUMBER OF TIME-GROUND ACCELERATION PAIRS DEFINING THE HORIZONTAL DIRECTION INPUT =220

( 58)[39] CODE FOR PRINTING ACCELERATION AS INPUT:
          0 : DO NOT PRINT ACCELERATION RECORD
          1 : PRINT ACCELERATION RECORD        =0

( 59)[40]  NUMBER OF STEPS TO SKIP WHEN PRINT=1

( 60)[41]  TITLE TO IDENTIFY RECORD
                1 : ELCENTRO
                2 : CALTEC 1
                3 : CALTEC 2
                4 : CALTEC 3
                5 : CALTEC 4
                6 : CALTEC 5
                7 : TAFT
                8 : SAN FERNANDO
                9 : OTHER    =1

( 61)[42]  MASS PROPORTIONAL DAMPING FACTOR: ALPHA =0.397

( 62)[43]  TANGENT STIFFNESS PROPORTIONAL DAMPING: BETA =0.00086

 ARE ALL THE DATA CORRECT(Y/N)  [Y]?:   Y

 **************
 OUTPUT CONTROL
 **************


( 64)[45] NUMBER OF NODES X-DISPLACEMENT IS REQUIRED=1

 ARE ALL THE DATA CORRECT(Y/N)  [Y]?:   Y
```

```
##############################################
LIST OF NODES FOR X-DISPLACEMENT TIME HISTORIES
##############################################


TOTAL NUMBER OF NODES FOR X-DISPLACEMENT=        1
.......................................


INPUT NUMBER=         1


( 68)[49] SPECIFY "NHOUT" NODE NUMBER=1

IS  THE DATA CORRECT(Y/N) [Y]?:   Y

####################
BEAM ELEMENT CONTROL
####################


( 70)[53] NUMBER OF MEMBERS IN THE STRUCTURE=9

( 71)[54] REINFORCING STEEL PROPERTY TYPES=1

( 72)[55] NUMBER OF CONCRETE PROPERTY TYPES=2

ARE ALL THE DATA CORRECT(Y/N)  [Y]?:   Y

###########################
REINFORCING STEEL PROPERTIES
###########################


STEEL PROPERTIE NUMBER=         1
...........................

( 73)[56] MODULUS OF ELASTICITY=29000

( 74)[57] STRAIN HARDENING RATIO=0.01

( 75)[58] YIELD STRESS=60

( 76)[59] ULTIMATE STRAIN=0.015

ARE ALL THE DATA CORRECT(Y/N) [Y]?:   Y
```

```
********************
CONCRETE  PROPERTIES
********************

CONCRETE PROPERTIE NUMBER=          1
..............................

( 77)[60] CONCRETE STRENGTH: Fc=3

( 78)[61] STRAIN AT MAXIMUM STRESS: Eo=0.002

( 79)[62] CONFINEMENT RATIO: RO=0.008

ARE ALL THE DATA CORRECT(Y/N)  [Y]?:   Y

CONCRETE PROPERTIE NUMBER=          2
..............................

( 80)[60] CONCRETE STRENGTH: Fc=3

( 81)[61] STRAIN AT MAXIMUM STRESS: Eo=0.002

( 82)[62] CONFINEMENT RATIO: RO=0.008

ARE ALL THE DATA CORRECT(Y/N)  [Y]?:   Y
```

```
****************
MEMBER PROPERTIES
****************


( 83)[63] MEMBER NUMBER(NEGATIVE VALUE FOR SYMMETRIC SECTION) =-1

( 84)[64] NODE NUMBER AT END OF THE MEMBER=1

( 85)[65] NODE NUMBER AT THE OTHER END=2

( 86)[66] STEEL TYPE NUMBER:
          IF THIS VALUE IS ZERO ALL THE INFORMATION
          EXCEPT NODE NUMBERS WILL BE THE SAME AS
          THE PRECEDING ELEMENT =1

( 87)[67] CONCRETE TYPE NUMBER=1

 ARE ALL THE DATA CORRECT(Y/N)  [Y]?:   Y

 ************************
 CROSS SECTIONAL PROPERTIES - MEMBER NUMBER=          1
 ************************


( 89)[69] MAGNITUDE OF AXIAL LOAD DUE TO GRAVITY=0

( 90)[70] " HT " (HEIGHT)=30

( 91)[71] " BB " (WIDTH-BOTTOM)=10

( 92)[72] " DCB "(COVER-BOTTOM)=3

( 93)[73] " ASB "(REINF.-BOTTOM)=2.4

 ARE ALL THE DATA CORRECT(Y/N)  [Y]?:   Y
```

```
****************
MEMBER PROPERTIES
****************

( 97)[63] MEMBER NUMBER(NEGATIVE VALUE FOR SYMMETRIC SECTION) =-2

( 98)[64] NODE NUMBER AT END OF THE MEMBER=3

( 99)[65] NODE NUMBER AT THE OTHER END=4

(100)[66] STEEL TYPE NUMBER:
          IF THIS VALUE IS ZERO ALL THE INFORMATION
          EXCEPT NODE NUMBERS WILL BE THE SAME AS
          THE PRECEDING ELEMENT =0

 ARE ALL THE DATA CORRECT(Y/N)  [Y]?:   Y
```

```
****************
MEMBER PROPERTIES
****************


(111)[63] MEMBER NUMBER(NEGATIVE VALUE FOR SYMMETRIC SECTION) =-3

(112)[64] NODE NUMBER AT END OF THE MEMBER=5

(113)[65] NODE NUMBER AT THE OTHER END=6

(114)[66] STEEL TYPE NUMBER:
          IF THIS VALUE IS ZERO ALL THE INFORMATION
          EXCEPT NODE NUMBERS WILL BE THE SAME AS
          THE PRECEDING ELEMENT =0

 ARE ALL THE DATA CORRECT(Y/N)  [Y]?:   Y
```

```
****************
MEMBER PROPERTIES
****************


(125)[63] MEMBER NUMBER(NEGATIVE VALUE FOR SYMMETRIC SECTION) =-4

(126)[64] NODE NUMBER AT END OF THE MEMBER=5

(127)[65] NODE NUMBER AT THE OTHER END=7

(128)[66] STEEL TYPE NUMBER:
          IF THIS VALUE IS ZERO ALL THE INFORMATION
          EXCEPT NODE NUMBERS WILL BE THE SAME AS
          THE PRECEDING ELEMENT =1

(129)[67] CONCRETE TYPE NUMBER=2

 ARE ALL THE DATA CORRECT(Y/N)  [Y]?:   Y

 **************************
 CROSS SECTIONAL PROPERTIES - MEMBER NUMBER=          4
 **************************


(131)[69] MAGNITUDE OF AXIAL LOAD DUE TO GRAVITY=75

(132)[70] " HT " (HEIGHT)=20

(133)[71] " BB " (WIDTH-BOTTOM)=20

(134)[72] " DCB "(COVER-BOTTOM)=2

(135)[73] " ASB "(REINF.-BOTTOM)=2

 ARE ALL THE DATA CORRECT(Y/N)  [Y]?:   Y
```

```
****************
MEMBER PROPERTIES
****************

(139)[63] MEMBER NUMBER(NEGATIVE VALUE FOR SYMMETRIC SECTION) =-5

(140)[64] NODE NUMBER AT END OF THE MEMBER=6

(141)[65] NODE NUMBER AT THE OTHER END=8

(142)[66] STEEL TYPE NUMBER:
          IF THIS VALUE IS ZERO ALL THE INFORMATION
          EXCEPT NODE NUMBERS WILL BE THE SAME AS
          THE PRECEDING ELEMENT =0

 ARE ALL THE DATA CORRECT(Y/N)  [Y]?:   Y
```

```
****************
MEMBER PROPERTIES
****************


(153)[63] MEMBER NUMBER(NEGATIVE VALUE FOR SYMMETRIC SECTION) =-6

(154)[64] NODE NUMBER AT END OF THE MEMBER=3

(155)[65] NODE NUMBER AT THE OTHER END=5

(156)[66] STEEL TYPE NUMBER:
          IF THIS VALUE IS ZERO ALL THE INFORMATION
          EXCEPT NODE NUMBERS WILL BE THE SAME AS
          THE PRECEDING ELEMENT =1

(157)[67] CONCRETE TYPE NUMBER=2

 ARE ALL THE DATA CORRECT(Y/N)  [Y]?:   Y

 *************************
 CROSS SECTIONAL PROPERTIES - MEMBER NUMBER=          6
 *************************


(159)[69] MAGNITUDE OF AXIAL LOAD DUE TO GRAVITY=50

(160)[70] " HT " (HEIGHT)=20

(161)[71] " BB " (WIDTH-BOTTOM)=20

(162)[72] " DCB "(COVER-BOTTOM)=2

(163)[73] " ASB "(REINF.-BOTTOM)=2

 ARE ALL THE DATA CORRECT(Y/N)  [Y]?:   Y
```

```
*****************
MEMBER PROPERTIES
*****************


(167)[63] MEMBER NUMBER(NEGATIVE VALUE FOR SYMMETRIC SECTION) =-7

(168)[64] NODE NUMBER AT END OF THE MEMBER=4

(169)[65] NODE NUMBER AT THE OTHER END=6

(170)[66] STEEL TYPE NUMBER:
          IF THIS VALUE IS ZERO ALL THE INFORMATION
          EXCEPT NODE NUMBERS WILL BE THE SAME AS
          THE PRECEDING ELEMENT =0

ARE ALL THE DATA CORRECT(Y/N)  [Y]?:   Y
```

```
****************
MEMBER PROPERTIES
****************


(181)[63] MEMBER NUMBER(NEGATIVE VALUE FOR SYMMETRIC SECTION) =-8

(182)[64] NODE NUMBER AT END OF THE MEMBER=1

(183)[65] NODE NUMBER AT THE OTHER END=3

(184)[66] STEEL TYPE NUMBER:
          IF THIS VALUE IS ZERO ALL THE INFORMATION
          EXCEPT NODE NUMBERS WILL BE THE SAME AS
          THE PRECEDING ELEMENT =1

(185)[67] CONCRETE TYPE NUMBER=2

 ARE ALL THE DATA CORRECT(Y/N)  [Y]?:   Y

 **************************
 CROSS SECTIONAL PROPERTIES - MEMBER NUMBER=          8
 **************************


(187)[69] MAGNITUDE OF AXIAL LOAD DUE TO GRAVITY=25

(188)[70] " HT " (HEIGHT)=20

(189)[71] " BB " (WIDTH-BOTTOM)=20

(190)[72] " DCB "(COVER-BOTTOM)=2

(191)[73] " ASB "(REINF.-BOTTOM)=2

 ARE ALL THE DATA CORRECT(Y/N)  [Y]?:   Y
```

#####
MEMBER
#####

(95)(
(96)(
(97)(
(98)(

ARE A

INP

```
*****************
MEMBER PROPERTIES
*****************


(195)[63] MEMBER NUMBER(NEGATIVE VALUE FOR SYMMETRIC SECTION) =-9

(196)[64] NODE NUMBER AT END OF THE MEMBER=2

(197)[65] NODE NUMBER AT THE OTHER END=4

(198)[66] STEEL TYPE NUMBER:
          IF THIS VALUE IS ZERO ALL THE INFORMATION
          EXCEPT NODE NUMBERS WILL BE THE SAME AS
          THE PRECEDING ELEMENT =0

 ARE ALL THE DATA CORRECT(Y/N)  [Y]?:   Y

    INPUT DATA OPTION
          0: NEW DATA
          1: CORRECT DATA
          2: LIST DATA
          3: END        =
```

OUTPUT RESULTS

```
****************************************************
*                    DRAIN                         *
*                                                  *
*      DYNAMIC RESPONSE ANALYSIS OF INELASTIC      *
*                 STRUCTURES                       *
*                                                  *
*                     by                           *
*                                                  *
*              Aecio Freitas Lira                  *
*                                                  *
*                    1985                          *
*                                                  *
****************************************************


INPUT FILE DATA NAME =example




TOTAL NUMBER OF NODES                 =    8

NO. OF CONTROL NODES                  =    4
NO. OF NODE GENERATION COMMANDS       =    2

NO. OF ZERO DISPLACEMENT COMMANDS     =    2
NO. OF IDENTICAL DISPLACEMENT COMMANDS =   0

NO. OF MASS GENERATION COMMANDS       =    1

DATA CHECKING ONLY                    =    0
```

CONTROL NODE COORDINATES

NODE	X-COORD	Y-COORD
1	0.000	480.000
2	250.000	480.000
7	0.000	0.000
8	250.000	0.000

NODE GENERATION COMMANDS

FIRST NODE	LAST NODE	NO.OF NODES	NODE DIFF	DISTANCE
1	7	2	2	0.000
2	8	2	2	0.000

COMPLETE NODE COORDINATES

NODE	X-COORD	Y-COORD
1	0.000	480.000
2	250.000	480.000
3	0.000	320.000
4	250.000	320.000
5	0.000	160.000
6	250.000	160.000
7	0.000	0.000
8	250.000	0.000

ZERO DISPLACEMENT COMMANDS

FIRST NODE	X CODE	Y CODE	ROTN CODE	LAST NODE	NODE DIFF
1	0	0	0	6	0
7	1	1	1	8	1

EQUAL DISPLACEMENT COMMANDS

NONE

ID ARRAY (FOR INTEREST)

NODE	X	Y	R
1	1	2	3
2	4	5	6
3	7	8	9
4	10	11	12
5	13	14	15
6	16	17	18
7	0	0	0
8	0	0	0

MASS GENERATION COMMANDS

FIRST NODE	X MASS	Y MASS	ROTN MASS	LAST NODE	NODE DIFF	MODIFYING FACTOR
1	0.6500E-01	0.6500E-01	0.0000E+00	6	1	1.00

COMPLETE NODAL MASSES

NODE	X-MASS	Y-MASS	R-MASS
1	0.065000	0.065000	0.000000
2	0.065000	0.065000	0.000000
3	0.065000	0.065000	0.000000
4	0.065000	0.065000	0.000000
5	0.065000	0.065000	0.000000
6	0.065000	0.065000	0.000000
7	0.000000	0.000000	0.000000
8	0.000000	0.000000	0.000000

STATIC LOAD CONTROL INFORMATION

STATIC LOAD CODE = 1 LOADS APPLIED

NO. OF NODAL LOAD COMMANDS = 2

EARTHQUAKE CONTROL INFORMATION

NO. OF INTEGRATION TIME STEPS = 600
INTEGRATION STEP SIZE =.2000E-01

MAGNIFICATION FACTORS FOR X EARTHQUAKE
ACCELERATION = 384.00
TIME = 1.00

MAGNIFICATION FACTORS FOR Y EARTHQUAKE
ACCELERATION = 0.00
TIME = 1.00

MAX. PERMISSIBLE DISPLACEMENT = 30.00
1STATIC NODAL LOAD GENERATION

FIRST NODE	X LOAD	Y LOAD	MOMENT LOAD	LAST NODE	NODE DIFF
1	0.000	-25.000	1067.000	5	2
2	0.000	-25.000	-1067.000	6	2

COMPLETE STATIC NODAL LOADS

NODE	X-LOAD	Y-LOAD	MOMENT
1	0.000	-25.000	1067.000
2	0.000	-25.000	-1067.000
3	0.000	-25.000	1067.000
4	0.000	-25.000	-1067.000
5	0.000	-25.000	1067.000
6	0.000	-25.000	-1067.000
7	0.000	0.000	0.000
8	0.000	0.000	0.000

EARTHQUAKE TITLE :
EL CENTRO - NS 1940

```
NO. OF X INPUT PAIRS        =  220
NO. OF Y INPUT PAIRS        =    0


PRINT CODE =  0
    0 - DO NOT PRINT
    1 - PRINT




NUMBER OF STEPS TO SKIP WHEN PRINT        =    1


1DAMPING COEFFICIENTS



MASS PROPORTION, ALPHA                =    0.397000

TANGENT STIFFNESS PROPORTION, BETA    =    0.000860


TIME HISTORY OUTPUT INTERVALS

NODE DISPLACEMENTS =    1


NO. OF NODES FOR X DISPL HISTORY =    1
NO. OF NODES FOR Y DISPL HISTORY =    0
NO. OF NODES FOR ROTATION HISTORY=    0


CODE FOR JOINT TIME HISTORY PRINT     =    2




NODES FOR X DISPL HISTORY

   1
```

ELEMENT SPECIFICATION, GROUP 1

B E A M   E L E M E N T S   W I T H   P L A S T I C   R E G I O N S :

NUMBER OF BEAMS = 9
NUMBER OF STEEL TYPES = 1
NUMBER OF CONCRETE TYPES= 2

S T E E L   P R O P E R T I E S :

	DATA			COMPUTED		
TYPE NO	ES	PS	FSY	EPSSU	EPSSY	FSU
1	0.2900E+05	0.1000E-01	0.6000E+02	0.1500E-01	0.2069E-02	0.6375E+02

C O N C R E T E   P R O P E R T I E S :

	DATA			COMPUTED				
TYPE NO.	FCP	EPSO	ROD	FCY	EPSCY	FCU	EPSCU	EPSM
1	0.3000E+01	0.2000E-02	0.8000E-02	0.2160E+01	0.7200E-03	0.3240E+01	0.2160E-02	0.1469E-01
2	0.3000E+01	0.2000E-02	0.8000E-02	0.2160E+01	0.7200E-03	0.3240E+01	0.2160E-02	0.1469E-01

M E M B E R   D A T A :

MEMBER NO.	CONCRETE TYPE	STEEL TYPE	CONNECTING NODES		LENGTH	S.S.R. (A/L)	AXIAL FORCE	CODE	ECCENTRICITIES ECCXI	ECCXJ	EXXYI	ECCYJ
1	1	1	1	2	0.2500E+03	0.5000E+00	0.0000E+00	0	0.000E+00	0.000E+00	0.000E+00	0.000E+00
2	1	1	3	4	0.2500E+03	0.5000E+00	0.0000E+00	0	0.000E+00	0.000E+00	0.000E+00	0.000E+00
3	1	1	5	6	0.2500E+03	0.5000E+00	0.0000E+00	0	0.000E+00	0.000E+00	0.000E+00	0.000E+00
4	2	1	5	7	0.1600E+03	0.5000E+00	0.7500E+02	0	0.000E+00	0.000E+00	0.000E+00	0.000E+00
5	2	1	6	8	0.1600E+03	0.5000E+00	0.7500E+02	0	0.000E+00	0.000E+00	0.000E+00	0.000E+00
6	2	1	3	5	0.1600E+03	0.5000E+00	0.5000E+02	0	0.000E+00	0.000E+00	0.000E+00	0.000E+00
7	2	1	4	6	0.1600E+03	0.5000E+00	0.5000E+02	0	0.000E+00	0.000E+00	0.000E+00	0.000E+00
8	2	1	1	3	0.1600E+03	0.5000E+00	0.2500E+02	0	0.000E+00	0.000E+00	0.000E+00	0.000E+00
9	2	1	2	4	0.1600E+03	0.5000E+00	0.2500E+02	0	0.000E+00	0.000E+00	0.000E+00	0.000E+00

X - S E C T I O N   D A T A :

MEMBER	HT	BB	DCB	ASB	BT	DCT	AST
1	0.3000E+02	0.1000E+02	0.3000E+01	0.2400E+01	0.1000E+02	0.3000E+01	0.2400E+01
2	0.3000E+02	0.1000E+02	0.3000E+01	0.2400E+01	0.1000E+02	0.3000E+01	0.2400E+01
3	0.3000E+02	0.1000E+02	0.3000E+01	0.2400E+01	0.1000E+02	0.3000E+01	0.2400E+01
4	0.2000E+02	0.2000E+02	0.2000E+01	0.2000E+01	0.2000E+02	0.2000E+01	0.2000E+01
5	0.2000E+02	0.2000E+02	0.2000E+01	0.2000E+01	0.2000E+02	0.2000E+01	0.2000E+01
6	0.2000E+02	0.2000E+02	0.2000E+01	0.2000E+01	0.2000E+02	0.2000E+01	0.2000E+01
7	0.2000E+02	0.2000E+02	0.2000E+01	0.2000E+01	0.2000E+02	0.2000E+01	0.2000E+01
8	0.2000E+02	0.2000E+02	0.2000E+01	0.2000E+01	0.2000E+02	0.2000E+01	0.2000E+01
9	0.2000E+02	0.2000E+02	0.2000E+01	0.2000E+01	0.2000E+02	0.2000E+01	0.2000E+01

C O M P U T E D   M E M B E R   P R O P E R   T I E S:

MEMBER	EIE	P	MY1	MY2	EIU1	EIU2	EA
1	0.3156E+08	0.1267E-01	0.3476E+04	0.3476E+04	0.1289E+07	0.1289E+07	0.1039E+07
2	0.3156E+08	0.1267E-01	0.3476E+04	0.3476E+04	0.1289E+07	0.1289E+07	0.1039E+07
3	0.3156E+08	0.1267E-01	0.3476E+04	0.3476E+04	0.1289E+07	0.1289E+07	0.1039E+07
4	0.1486E+08	0.9660E-02	0.2518E+04	0.2518E+04	0.6789E+06	0.6789E+06	0.1316E+07
5	0.1486E+08	0.9660E-02	0.2518E+04	0.2518E+04	0.6789E+06	0.6789E+06	0.1316E+07
6	0.1424E+08	0.1031E-01	0.2342E+04	0.2342E+04	0.5775E+06	0.5775E+06	0.1316E+07
7	0.1424E+08	0.1031E-01	0.2342E+04	0.2342E+04	0.5775E+06	0.5775E+06	0.1316E+07
8	0.1353E+08	0.1103E-01	0.2158E+04	0.2158E+04	0.4907E+06	0.4907E+06	0.1316E+07
9	0.1353E+08	0.1103E-01	0.2158E+04	0.2158E+04	0.4907E+06	0.4907E+06	0.1316E+07

******************** RESULTS ********************

STATIC NODAL DISPLACEMENTS

NODE	X-DISPL	Y-DISPL	ROTATION
1	0.000	-0.009	0.00081
2	0.000	-0.009	-0.00081
3	0.000	-0.008	0.00033
4	0.000	-0.008	-0.00033
5	0.000	-0.005	0.00048
6	0.000	-0.005	-0.00048
7	0.000	0.000	0.00000
8	0.000	0.000	0.00000

```
I am in step = 1 to 105 (COUNTER)

I reached the first yield at step = 106

I am in step = 107 to 600 (COUNTER)
```

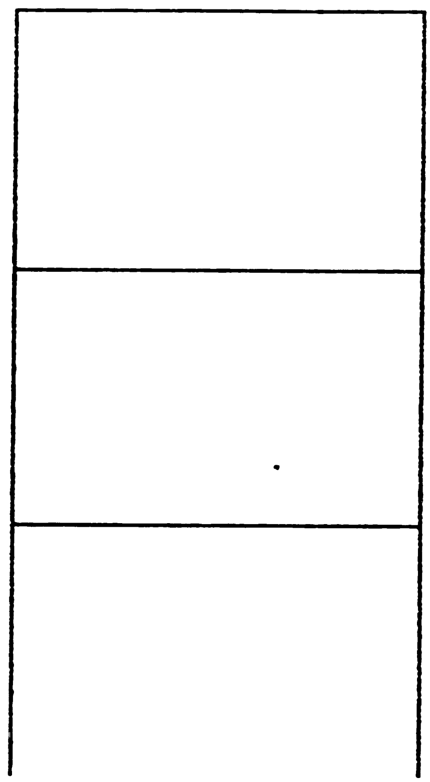
NODAL DISPLACEMENT ENVELOPES

	X-DISPLACEMENT				Y-DISPLACEMENT				ROTATION			
NODE	POSITIVE	TIME	NEGATIVE	TIME	POSITIVE	TIME	NEGATIVE	TIME	POSITIVE	TIME	NEGATIVE	TIME
1	3.549	2.14	-4.132	2.40	0.005	2.14	-0.026	2.40	0.00324	2.38	-0.00123	2.14
2	3.549	2.14	-4.131	2.40	0.007	2.40	-0.024	2.14	0.00164	2.38	-0.00289	5.00
3	2.634	2.14	-3.060	2.40	0.006	2.14	-0.022	2.40	0.00558	2.40	-0.00452	5.00
4	2.634	2.14	-3.060	2.40	0.007	2.40	-0.021	2.14	0.00469	2.40	-0.00478	2.14
5	1.192	2.16	-1.407	2.42	0.004	2.14	-0.014	2.40	0.00710	2.40	-0.00511	2.14
6	1.191	2.16	-1.406	2.42	0.005	2.40	-0.013	2.14	0.00629	2.40	-0.00618	2.14
7	0.000	0.00	0.000	0.00	0.000	0.00	0.000	0.00	0.00000	0.00	0.00000	0.00
8	0.000	0.00	0.000	0.00	0.000	0.00	0.000	0.00	0.00000	0.00	0.00000	0.00

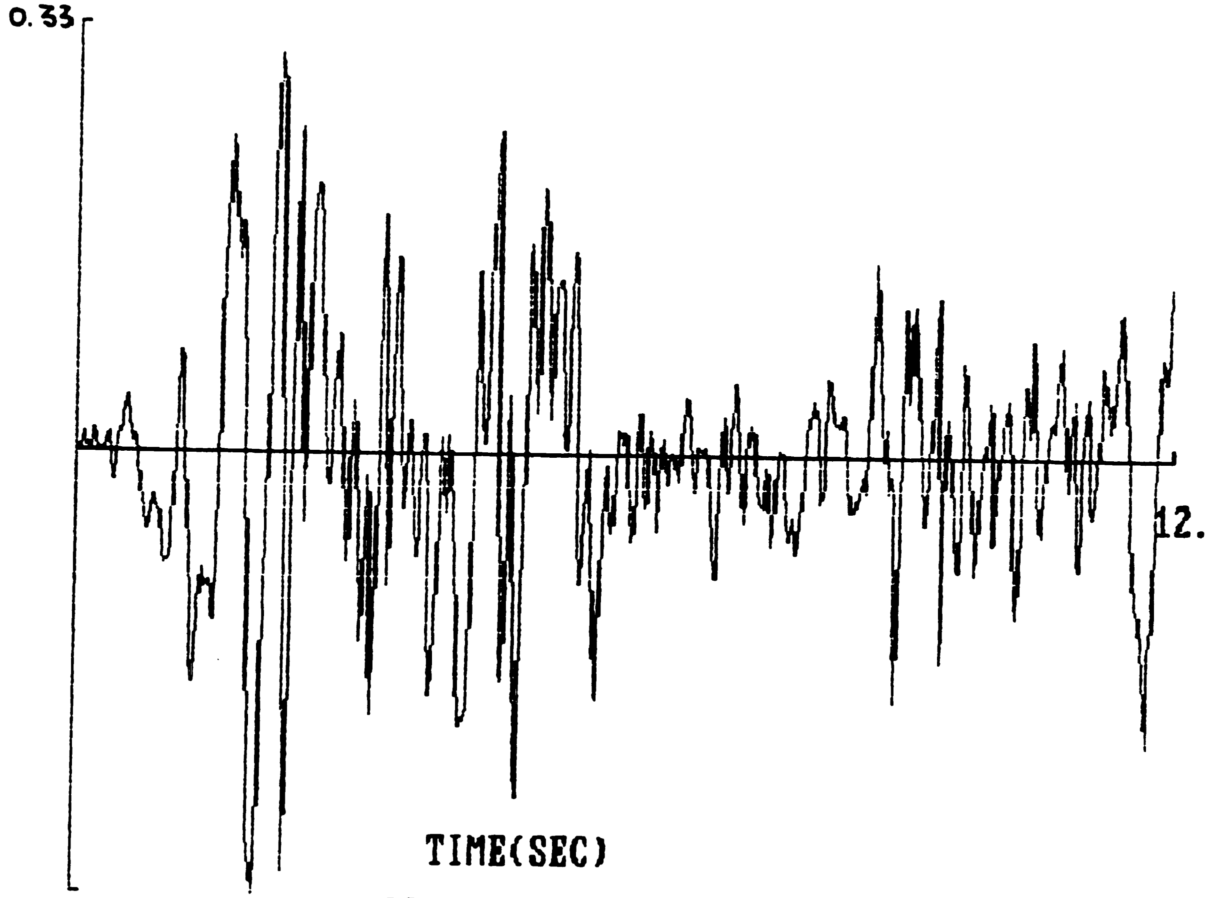
```
    PLASTIC RESULTS RATIO
**************************************************************************************************
  *    CURV.DUCT.*          PLASTIC        *  DAMAGE  *           MAXIMUM MOMENT              *
  *    PROVIDED  *          LENGTHS        *  FACTOR  *
**************************************************************************************************
MEMBER      CU/CY       LI/L       LJ/L      NODEI      NODEJ       MAX.MOM / YIELD MOM.
     1         35.052     0.000     0.000     0.000     0.000              0.56
     2         35.052     0.045     0.020     0.320     0.170              1.10
     3         35.052     0.026     0.041     0.208     0.295              1.09
     4         27.481     0.000     0.058     0.000     0.519              1.11
     5         27.481     0.000     0.049     0.000     0.450              1.09
     6         32.716     0.013     0.000     0.142     0.000              1.03
     7         32.716     0.026     0.004     0.241     0.073              1.05
     8         39.188     0.000     0.000     0.000     0.000              0.83
     9         39.188     0.000     0.000     0.000     0.000              0.96


DUCTILITY DISPLACEMENT
*********************


 FIRST YIELD DISPLACEMENT            =    3.11740
 MAX.DISPLACEMENT AT THE TOP         =    4.13208
 DISPLACEMENT DUCTILITY              =    1.32549
```

PLOT OPTIONS OUTPUT

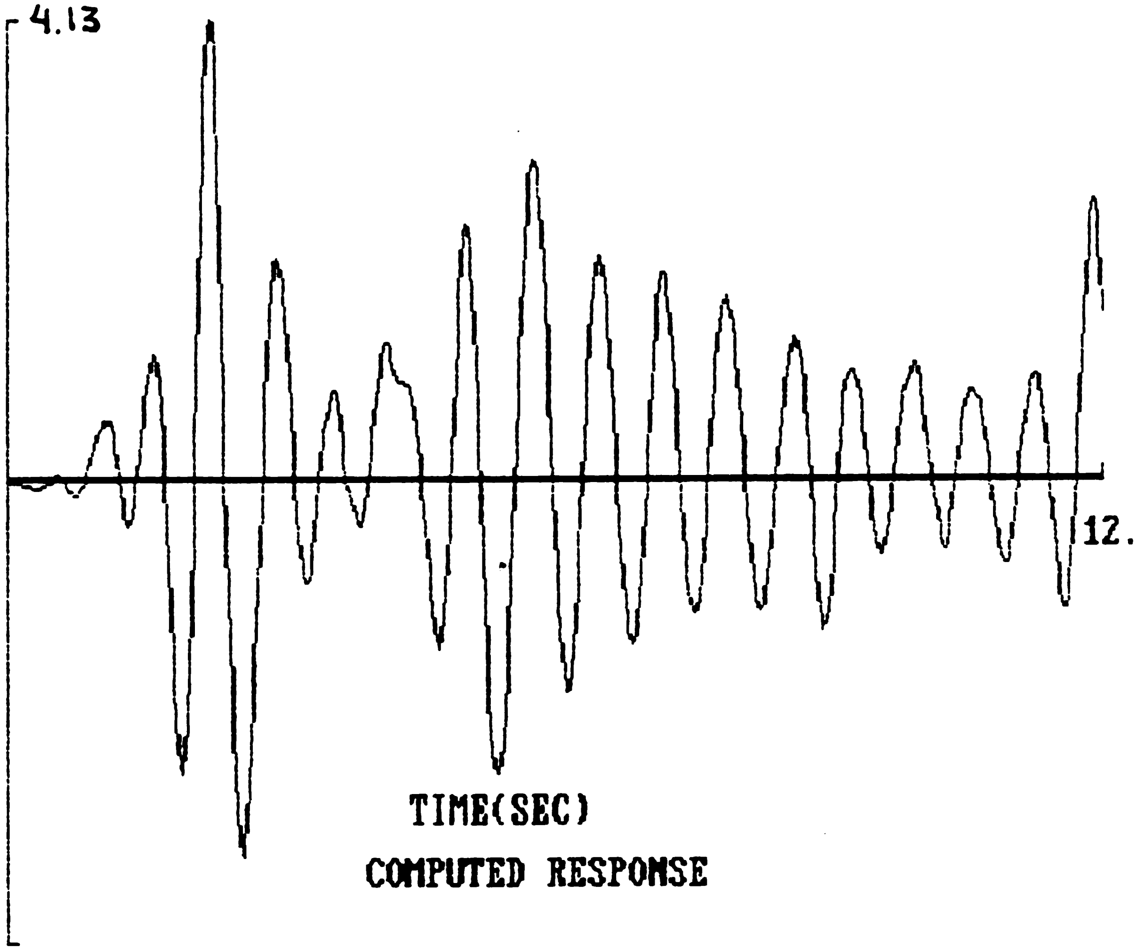
```
************PLOT OPTIONS*************

1: UNDEFORMED STRUCTURE
2: GROUND ACCELERATION INPUT
3: DISPLACEMENT AT THE TOP
4: DEFORMED STRUCTURE
5: FINITE PLASTIC REGIONS
6: END

NOTE ABOUT PRINTING THE SCREEN:
******************************

- PRESS  "SHIFT/PRTSC" FOR PRINTING
- PRESS  "ENTER" TWICE TO EXIT


TYPE YOUR INPUT OPTION  =
```

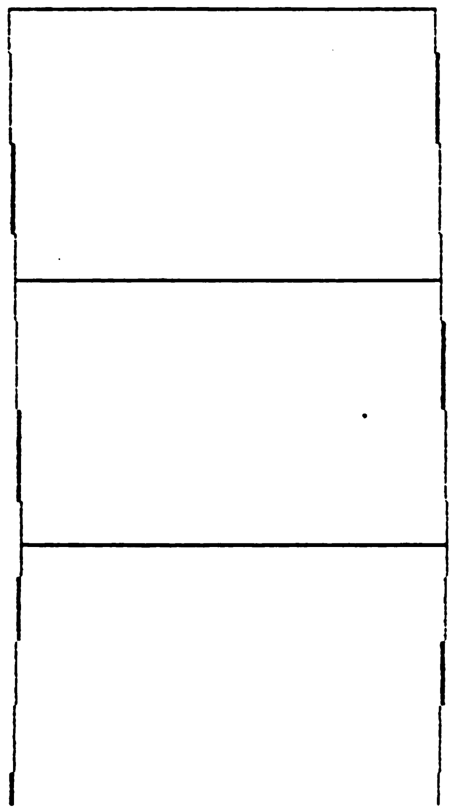


UNDEFORMED STRUCTURE

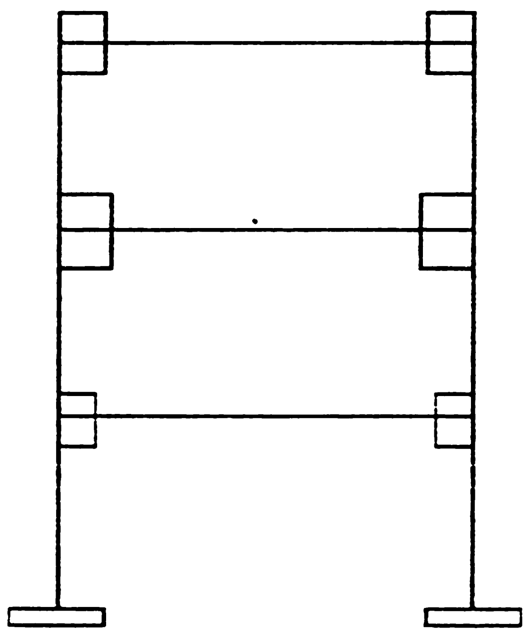


GROUND ACCELERATION (g)


4.13
12.
TIME(SEC)
COMPUTED RESPONSE



DEFORMED STRUCTURE



FINITE SIZE OF PLASTIC REGIONS
PLASTIC LENGTH
DAMAGE FACTOR
1.0