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THE DYNAMIC RESPONSE OF REINFORCED CONCRETE
FRAMES WITH STIFFNESS
DEGRADATION

by

Alan Lepore Morris, P. Eng.

A THESIS

submitted to the Faculty
of Graduate Studies in partial fulfilment
of the requirements for the degree of
MASTER OF ENGINEERING

Faculty of Engineering
Carleton University
Ottawa, Ontario

May 1978

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The undersigned recommend to the Faculty
of Graduate Studies acceptance of the thesis:

"THE DYNAMIC RESPONSE OF REINFORCED CONCRETE FRAMES
WITH STIFFNESS DEGRADATION"

submitted by Alan Lenage Morris, P.Eng.,
in partial fulfilment of the requirements for
the degree of
MASTER OF ENGINEERING

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ABSTRACT

The principal objective of this thesis is to explore the dynamic behaviour of reinforced concrete building frames. Particular attention is paid to the influence of cracking on modal and seismic behaviour.

Hence the work is divided into two parts:

1. A comprehensive study of the parameters that may influence modal response.
2. Examination of the relative influence of stiffness degrading behaviour as compared to non degrading behaviour on maximum seismic response.

In so doing this, studies are initiated into the dynamic response of concrete framed buildings having set backs. To handle the large volume of calculations, it was necessary to write a computer program that is "state of the art" in terms of modelling capabilities and computational efficiency for small to moderate size frames.

The program permits the modal, static and seismic analysis of building frames taking into account:

- a) the finite dimensions of the beam-column joints,
- b) column extensibility and
- c) the p-delta effects arising from gravity loads acting on the displaced frame.

Non-linear behaviour may be modelled as either degrading, or nondegrading.

Three multistory building frames were designed according to the current Canadian design codes, two uniform over their entire height and one setback at its mid-height. On the basis of these designs, their modal performance and the applicability of the degrading stiffness model as compared to the commonly used non degrading model is subsequently explored.

It is found that the modal frequencies and mode shapes are hardly influenced by the p -delta forces but the lack of consideration of finite joint dimensions yields a sizable underestimate of the modal frequencies on the one hand, whereas the disregard of cracking leads to a considerable over estimate of the modal frequencies on the other. Finally it is shown that the ductility demand of the various members of a structure are very much influenced by the type of hysteresis model assumed.

ACKNOWLEDGEMENTS

The research objectives to examine the effect of certain parameters on modal response and to explore the relative merits of the stiffness degrading model applied to reinforced concrete buildings both uniform and setback in plan, was suggested to the writer by his supervisor, Dr. J. L. Humar, and his assistance particularly in the areas of modal and seismic analysis is most gratefully acknowledged.

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CHAPTER 1

INTRODUCTION

1.1 BACKGROUND

That the effects of earthquakes can be horrendous in terms of loss of life and damage to property has been common knowledge to mankind since prehistory. The terrible events that took place in Nicaragua 1972, Guatemala 1976, Italy 1976, and the Philippines also in 1976 provide ample recent examples of the awesome energy that can be suddenly released at different places over the earth's surface. The devastating geological consequences of earthquakes include tsunamis (commonly but incorrectly described as tidal waves), landslides, soil liquefaction, surface renting and heaving, and other effects less spectacular.

Modern earthquakes may be classified as resulting from either manmade or from natural causes. Manmade earthquakes result from detonations below the earth's surface, performed perhaps for the purpose of geological investigation, mining or military experiment. Natural earthquakes result from the collapse of underground caves, volcanic action, or tectonic forces. Whatever the cause, three types of waves are generated, two being within the earth's crust, and one at the earth's surface. The interior waves are known as the primary or "P" wave which radiates longitudinally from the focus, and the secondary

or "S" wave that propagates transversely to the "P" wave. The interaction of these waves at the surface induces the long or "L" wave. It is perhaps the "L" waves that can be so destructive to a structure.

Prior to the renaissance there was little that mankind could do to alleviate the danger. However, consequent to the separation of religious and secular thought and the quest for scientific knowledge throughout the last five centuries, particularly this century, there has emerged three main areas of attack. They may be described as the prediction of earthquakes, the prevention of earthquakes and the participation of structures with earthquakes. To the practicing engineer, the latter area promises to be most fruitful particularly in the area of building design.

The ability of buildings to successfully withstand severe earthquakes requires that engineers be knowledgeable in essentially three areas. These include:

1. An understanding of the causes, mechanisms of propagation and measurement of seismic waves.
2. An ability to predict the complex response of given structure given a particular input motion.
3. The application of sound detailing and construction practices aimed at providing the structure with adequate ductility.

The sophisticated application of Newton's Law of motion involving the careful definition of the parameters of the mathematical equation expressing this law, and the use of digital computers have provided deep insight into real building response. However, much remains to be learnt, particularly in the area of material behaviour. Thus the current building codes reflect the fact that although a general understanding of what is a complex dynamic problem is growing, the application of earthquake resistant design is still largely an art based on observation and experience.

Analytical difficulties arise from several sources. They are material behaviour, the presence of non structural materials, ground-building interaction and geometric irregularities. Most buildings are irregular in one way or another. Irregularities may result from particular arrangements of materials, or walls, doors and window openings, and service ducts. Others may result from the abrupt curtailment of structural members or an unusual arrangement of floor plans within the building resulting perhaps from the need to satisfy certain functional, economic or architectural requirements. Buildings with structural or geometric setbacks pose a particularly complex problem, so much so that current codes are not capable of providing the design engineer with refined procedures.

Some analytical work for steel frames with setbacks has been undertaken, however, little if any, investigation of similar frames

employing reinforced concrete has been initiated.

No work has been reported that in addition attempts to reflect degrading stiffness that is characteristic of reinforced concrete members subjected to severe load reversals.

1.2 REVIEW OF PREVIOUS WORK

Basic to the true prediction of building response during seismic excitation is the need to accurately model the behaviour of each member of the structure into the inelastic range. Prior to the availability of digital computers, studies were limited either to elastic analysis of highly idealized structures, or to inelastic studies employing simple hysteresis rules and applied to very small structures. Hand or graphical computation methods were used.

The introduction of the computer to structural analysis gave tremendous impetus to the understanding of dynamic behaviour. What hitherto had entailed enormous computational effort yielding approximate answers only, now meant that complex problems could be quickly and accurately solved with routine application of packaged computer programs.

*

In this context Wen and Janssen [1] proposed subdivision of a member into small segments applying a bilinear moment-curvature

*Numbers in square brackets refer to references given at the conclusion of this study.

relationship to each segment. Naturally, only simple structures could be studied because of the large computer core requirements. Clough, Benuska, and Wilson [2] proposed conceptual vertical division of a member into two parallel components, their combined moment-rotation relationship being elastic-inelastic. Accordingly, one component was elasto-plastic, and the other entirely elastic.

Giberson [3] proposed representation of a member as an elastic line element with two springs one at either end. Inelastic behaviour of the real member was attributed to the springs. Takizawa [4] developed a model for a reinforced concrete member whereby the amplitude and distribution of its flexural rigidity varied along its length according to the value and sign of the applied end moments. Otani [5] conceived a member modelled as two parallel beams, one elastic and the other inelastic with inelastic joint effects assigned to a spring located at each of their far ends.

Only the latter three models are versatile with respect to representing arbitrary moment-rotation relationships. Other researchers adopted one or other of the models as described or in modified form. Aoyama and Sugano [6] achieved a trilinear moment-rotation primary curve for a reinforced concrete member by subdividing the member vertically into four imaginary components. The amounts of reinforcement at each end were considered unequal. This model was limited to non-degrading type behaviour. Takizawa [4] extended the scope of Aoyama's

model to include degrading type behaviour by dynamically subdividing the model according to the instantaneous stiffness at each loading stage. Otari [7] used a model that consisted of an elastic line element and two springs at each end. The innermost spring at each end handled the inelastic effects within the portion of the member spanning between the beam-column joints. The outermost springs handled the bond slip of the tensile reinforcement within the joint.

Prior to the mid-sixties the elasto-plastic moment-curvature relationship was commonly used because of its mathematical simplicity and compliance with observed behaviour. As noted previously, however, Wen and Janssen, and Clough et al reported in 1965, the use of a bilinear relationship that simulated the strain hardening phenomenon of steel members. This model was adopted by other researchers to reinforced concrete members. Jennings [8] in 1963 and Goel [9] in 1969 made use of the Ramberg-Osgood function in their investigations of steel frames. Celebi [10] suggested a modified form of this function to be applied to reinforced concrete members under the influence of shear and bending.

Clough [11] presented a landmark paper in 1966 that represented perhaps for the first time, an attempt to come to grips with the peculiar nature of reinforced concrete members when subjected to cyclic loading. His moment-displacement relationship reflected the fact that when a member had undergone its first excursion of loading

to yield and unloading in say the positive direction, the negative loading regime exhibited a reduced stiffness defined by a line joining the intercept of the displacement axis at unloading and the current negative yield point. Succeeding positive and negative loading regimes exhibited stiffnesses similarly reduced.

Takeda, Sozen, and Nielson [12] refined Clough's simple stiffness degrading rules as a result of comprehensive experiments on test specimens mounted in an earthquake simulator. The validity of their rules has been demonstrated by Otani [13] and others. Sozen [14], Celebi [10], Bertero and Popov [15], Hawkins, Mitchell and Sheu [16], Gulkan and Sozen [17], Park, Kent and Sampson [18], Morita and Kaku [19], Darwin and Pecknold [20], and others examined parameters that could influence the behaviour of the moment-displacement relationship of reinforced concrete members.

Analytical studies of reinforced concrete frames that incorporate the stiffness degrading model have been undertaken. Clough [11, 21] investigated a one story shear type frame using both the degrading and the non degrading models. Various structural properties including the period of vibration were varied to determine the relative behaviour of the two models. One important conclusion drawn from the study was that for long period buildings the earthquake resistance was not materially affected by the degrading stiffness property, whereas the resistance of short period buildings was markedly reduced.

On the basis of this work, Chopra and Kan [22] extended the investigation of the applicability of both models to multistory frames idealized again as shear type structures. They noted that the ductility requirements for stiff buildings are increased if stiffness degrading behaviour acts. They pointed also to the need for a more refined structural model to appreciate better the effects of degradation on local ductility requirements.

Yoshioka, Takeda and Nakagawa [23] analyzed the behaviour of a ten story frame idealized as a series of column stacks with propped cantilever beams spanning from opposite faces of each stack. The beams were permitted to rotate, and the columns were permitted to rotate and sway. The major findings were similar to those of Chopra and Kan.

Umemura, Aoyama, and Takizawa [24], analyzed a three story building idealizing it:

1. as a structure whose beams and columns were permitted to rotate and sway, whose columns in addition were permitted to extend, and whose joints were permitted to deform, and
2. as a shear-type structure only.

They found that idealization as a shear-type building is not sufficient if accurate story responses are required.

Otani and Sozen [12] investigated a three story one bay frame and compared analytical and test results. The analytical model did not allow column axial deformations, although sways and rotations of the columns were permitted. As noted earlier they found reasonable agreement between the two sets of results.

Takayanagi and Schnobrich [25] studied the response of a ten story coupled shear wall model. Analytical and test results compared favourably.

Buildings with setbacks present special problems of analysis and design. If the fundamental frequency of the tower approaches a lower vibrational mode of the building the tower structure may experience motion and lateral forces much larger than those at corresponding levels of a comparable uniform building.

Studies on the response of appendages by Bustamante and Rapoport, as referred to by Newmark and Rosenblueth [26] indicated that the base shear on the appendage was as much as eight times that predicted by static analysis. Berg [27] initiated exploratory studies into the elastic behaviour of buildings with symmetric and asymmetric setbacks. He employed a stepped cantilever shear beam as a model. An approximate method for predicting the peak elastic response of buildings with setbacks placed symmetrically with respect to the direction of excitation was presented by Penzien [28]. This work was later extended

by Penzien and Chopra [29] to include the torsional effects in structures with asymmetric setbacks.

The effects of frequency coincidence of buildings supporting towers were explored by Skinner, Skilton, and Laws [30]. Their conclusions were similar to those of Penzien. A comprehensive study was undertaken by Jhaveri [31]. He considered two elastic models: a stepped rectangular cantilever shear beam and a lumped mass shear type building. Limited inelastic studies were undertaken by Pekau [32], and an important conclusion of his work indicated that elastic analysis may considerably underestimate the level of tower response. Further investigation of the elastic and inelastic response of frames with symmetric setbacks was pursued by Humar [33], and later reported by Humar and Wright [34]. They proposed a simple, accurate method for finding the fundamental period of a setback building. Other related studies have been undertaken by Gluck [35], Brandow, Manning and Weaver [36], Barria, Guendelman, and Monge [37], Kan and Chopra [38], Blume and Jhaveri [39], Shepard [40], and Ban [41], Pekau and Green [42].

The intent of this thesis is to model reinforced concrete building behaviour more precisely perhaps than has previously been undertaken. Columns will be considered as extensible, and both the beams and columns will rotate and translate. Due to the findings of Goel [9], Humar [33] and others it is evident that the p-delta effects are quite influential when large inelastic sways are anticipated. Accordingly,

the modal analyses will include these effects to test the influence on modal behaviour. However, as the seismic analyses are comparative analyses only, rather than examination of absolute responses, the effects will be neglected. For large structures much computational effort will be directed to assembling and solving the simultaneous equations associated with joint displacements. In fact it is reported [43] that up to eighty percent of the total execution time may be so spent. It is therefore imperative that the processing of data be done efficiently. Many papers have been written on algorithms for solving simultaneous equations. The method adopted by the author employs the techniques reported by Mondkar and Powel [44], and Powel [45]. Zero multiplication during the Gauss elimination stage are avoided and only that portion of the structure stiffness matrix that has been altered due to inelastic changes within an integration step, is decomposed.

Basic to the mathematical procedure used for studying dynamic response however, is the method employed to integrate the equations of motion. Typically, for inelastic studies, numerical integration procedures are used. Many researchers have studied this area and reported their findings [46,47]. There are several obstacles that must be overcome when using such a method however. The assumption on the rate of change of joint accelerations and the size of the interval over which the equations are to be integrated are of supreme importance, both from the point of view of accuracy and from the point of computational efficiency.

1.3 SCOPE OF PRESENT STUDY

It is felt that the behaviour of reinforced concrete buildings requires much more examination. Previous analyses have either ignored the p-delta effects, gravity load effects or the influence of column axial deformations. The circumstances where stiffness degrading effects are more applicable than the non degrading effects have been undertaken for only a limited class of structure, also the results reported thus far have been limited by the use only of one or several earthquake records. Thus the generality of the findings may be somewhat limited.

Modal analyses likewise have not considered the relative influence of the presence or absence of p-delta forces, joint dimensions, and different degrees of cracking in the concrete.

The purpose of this study then is to model uniform frames removing most of the above noted deficiencies. It hopes to shed further light on the relative importance of the two stiffness models, and the influence of the above noted parameters on modal response. Further, the studies will be extended to begin to explore the inelastic response of reinforced concrete frames with set backs, with and without stiffness degradation.

Accordingly the work reported here is approached in the following stages:

1. Extensive survey of the literature reported elsewhere.
2. Development of the analytical tools in the form of computer programs capable of modelling the modal, static and elastic-inelastic dynamic response of building frames.
3. The structural design of three frames according to the current design codes.
4. The parametric investigation of the modal and seismic responses of these models.

Chapter 2 is devoted to developing the mathematical equations supporting this work. Chapter 3 examines the underlying assumptions, and the rules governing stiffness degrading behaviour. Chapter 4 describes the design process of the building frames. Chapter 5 evaluates the results of the computer runs, and Chapter 6 presents a summary, conclusions drawn, and recommendations for future studies.

CHAPTER 2

MATHEMATICAL FORMULATION OF THE EQUATIONS OF MOTION

2.1 INTRODUCTION

The research work for this thesis relies heavily upon an efficient means of integrating the equations of motion. There are two fundamental methods [48] that in theory might be used:

1. The so-called exact or closed form method of integration.
2. Approximate methods of integration that include graphical, power series, or numerical techniques.

The first method is very suitable when both the structural properties remain constant and the forcing function is either non existent or regular in wave form. This method is thus eminently suitable for studying the modal characteristics of an elastic structure, or its elastic response during periodic excitation. In fact, the method has been extended to the study of inelastic behaviour by considering elastic behaviour within the different stages of plasticity [49].

However, it is unusual for a structure to remain elastic during a severe earthquake, and indeed it would be hoped that inelastic behaviour would occur in order to assist energy absorption. In addition the wave form of the ground acceleration is both highly irregular and

random. Therefore, it is felt necessary here to consider the second alternative. It is obvious that the only viable technique for this study is the numerical integration procedure.

2.2 THE GENERAL FORM OF THE EQUATIONS OF MOTION

In so far that numerical integration procedures are perfectly general in their application, they present inherent difficulties to the analyst, chief of which are the problems of numerical stability and accuracy. In 1959 Newmark [46] presented a most important paper that described a method of integration suitable to researchers in structural dynamics that was stable (or stable within certain limits), accurate, and efficient. Many other researchers have investigated the problem, and have presented formulae and the conditions under which they are stable. In 1974, Humar and Wright [47] investigated the formulae currently available for structural dynamics and demonstrated that they all belonged to the single-point predictor-corrector class. They presented a general method for deriving both single-point and multiple-point formulae.

Both the conditionally stable and the unconditionally stable forms (commonly referred to as the linear and the constant acceleration methods) derived by Newmark can be used by the program developed for this study. Consider now the sketch of the multi-story frame in Fig.2.1, and the sketch shown in Fig.2.2:

FIG. 2.1 MULTISTORY-MULTIBAY FRAME SUBJECTED TO BASE MOTION

$$\ddot{z} = \ddot{z}_0 + (\ddot{z}_1 - \ddot{z}_0) \frac{t}{\Delta t}$$

$$\dot{z} = \dot{z}_0 + \int_0^t \ddot{z} dt$$

$$\dot{z}_1 = \dot{z}_0 + \ddot{z}_0 \frac{\Delta t}{2} + \ddot{z}_1 \frac{\Delta t}{2}$$

$$z = z_0 + \int_0^t \dot{z} dt$$

$$z_1 = z_0 + \dot{z}_0 \Delta t + \ddot{z}_0 \frac{\Delta t^2}{3} + \ddot{z}_1 \frac{\Delta t^2}{6}$$

LINEAR ACCELERATION METHOD

$$\ddot{z} = \frac{1}{2} (\ddot{z}_0 + \ddot{z}_1)$$

$$z = \dot{z}_0 + \int_0^t \ddot{z} dt$$

$$\dot{z}_1 = \dot{z}_0 + \ddot{z}_0 \frac{\Delta t}{2} + \ddot{z}_1 \frac{\Delta t}{2}$$

$$z = z_0 + \int_0^t \dot{z} dt$$

$$z_1 = z_0 + \dot{z}_0 \Delta t + \ddot{z}_0 \frac{\Delta t^2}{4} + \ddot{z}_1 \frac{\Delta t^2}{4}$$

CONSTANT ACCELERATION METHOD

FIG. 2.2 ACCELERATION METHODS

The equation of motion using matrix algebra notation may be written as:

$$[M] \{\ddot{z}\} + [C] \{\dot{z}\} + [K] \{z\} = \{F\} \quad (2.1)$$

where $[M]$ is the mass matrix, $[C]$ is the damping matrix, $[K]$ is the structure stiffness matrix, and $\{F\}$ is the exciting force vector.

Assuming then that the parameters $[C]$, and $[K]$ change during the course of excitation, it is convenient to rewrite Eq.2.1 in incremental form as follows:

$$[M] \{\Delta \ddot{z}\} + [C] \{\Delta \dot{z}\} + [K] \{\Delta z\} = \{\Delta F\} \quad (2.2)$$

It can be seen from Fig.2.2 that general expressions for total (rather than incremental) joint accelerations, velocities, and displacements can be written as:

$$\{\ddot{z}\} = \{A\} + \{\ddot{z}_1 + S \cdot \ddot{z}_0\} B \quad (2.3)$$

$$\{\dot{z}\} = \{\dot{z}_0\} + \int_0^t \{\ddot{z}\} dt \quad (2.4)$$

$$\{z\} = \{z_0\} + \int_0^t \{\dot{z}\} dt \quad (2.5)$$

where the subscripts 0 and 1 represent respectively, values at the start and the end of the interval Δt under consideration. If it is assumed that the joint accelerations vary linearly over the small interval of time Δt ($\{A\} = \{\ddot{z}_0\}$, $S=1$, $B=\frac{t}{\Delta t}$) as is shown in Fig.2.2, then it can be shown that by integrating Eqs.2.4, and 2.5 that the following two expressions represent joint velocities and displacements at the end of the interval:

$$\{\dot{z}_1\} = \{\dot{z}_0\} + \{\ddot{z}_0 + \ddot{z}_1\} \frac{\Delta t}{2} \quad (2.6)$$

$$\{z_1\} = \{z_0\} + \{\dot{z}_0\} \Delta t + \{2\ddot{z}_0 + \ddot{z}_1\} \frac{\Delta t^2}{6} \quad (2.7)$$

By substituting $\{\Delta \dot{z}\}$ for $\{\dot{z}_1\} - \{\dot{z}_0\}$, $\{\Delta z\}$ for $\{z_1\} - \{z_0\}$, and eliminating $\{\ddot{z}_1\}$, it can be further shown that the incremental joint velocities and accelerations can be expressed as:

$$\{\Delta \dot{z}\} = -\{\ddot{z}_0\} \frac{\Delta t}{2} - 3\{\dot{z}_0\} + \{\Delta z\} \frac{2}{\Delta t} \quad (2.8)$$

$$\{\Delta \ddot{z}\} = -3\{\ddot{z}_0\} - \{\dot{z}_0\} \frac{6}{\Delta t} + \{\Delta z\} \frac{6}{\Delta t^2} \quad (2.9)$$

or in more compact form as:

$$\begin{bmatrix} \{\Delta \dot{z}\} \\ \{\Delta \ddot{z}\} \end{bmatrix} = \begin{bmatrix} -\frac{\Delta t}{2} & -3 & \frac{3}{\Delta t} \\ -3 & -\frac{6}{\Delta t} & \frac{6}{\Delta t^2} \end{bmatrix} \begin{bmatrix} \{\ddot{z}_0\} \\ \{\dot{z}_0\} \\ \{\Delta z\} \end{bmatrix} \quad (2.10)$$

If however, it is assumed that the joint accelerations remain constant during the small interval of time (i.e. $\{A\} = \{0\}$, $S=1$, $R=\frac{1}{2}$) it can be shown by a similar procedure as described above, that the incremental joint velocities and accelerations can be expressed as:

$$\{\Delta \dot{z}\} = -2\{\dot{z}_0\} + \{\Delta z\} \frac{2}{\Delta t} \quad (2.11)$$

$$\{\Delta \ddot{z}\} = -2\{\ddot{z}_0\} - \{\dot{z}_0\} \frac{4}{\Delta t} + \{\Delta z\} \frac{4}{\Delta t^2} \quad (2.12)$$

or in more compact form as:

$$\begin{Bmatrix} \{\Delta \dot{z}\} \\ \{\Delta \ddot{z}\} \end{Bmatrix} = \begin{bmatrix} 0 & -2 & \frac{2}{\Delta t} \\ -2 & -\frac{4}{\Delta t} & \frac{4}{\Delta t^2} \end{bmatrix} \begin{Bmatrix} \{\dot{z}_0\} \\ \{\ddot{z}_0\} \\ \{\Delta z\} \end{Bmatrix} \quad (2.13)$$

If it is further assumed that the damping matrix $[C]$ can be expressed as a linear combination of the mass and the stiffness matrices i.e.:

$$[C] = \alpha[M] + \beta[K] \quad (2.14)$$

where α and β are constants, and if $\{\Delta \ddot{y}\}$ is replaced by $-[M] \{\Delta \ddot{v}\}$, then by substituting the expressions for Eqs. 2.8, 2.9, and 2.14 into Eqs. 2.2, and thence rearranging, the following expression is obtained for the case of linear joint accelerations:

$$\begin{aligned} & \left[[M] \left(\frac{6}{\Delta t^2} + \frac{3\alpha}{\Delta t} \right) + [K] \left(\frac{3\beta}{\Delta t} + 1 \right) \right] \{\Delta z\} = \\ & [M] \left[-\{\Delta \ddot{y}\} + \{\ddot{z}_0\} \left(3 + \alpha \frac{\Delta t}{2} \right) + \{\dot{z}_0\} \left(\frac{6}{\Delta t} + 3\alpha \right) \right. \\ & \left. + \beta [K] \left[3\{\dot{z}_0\} + \frac{\Delta t}{2} \{\ddot{z}_0\} \right] \right] \quad (2.15) \end{aligned}$$

or:

$$[K^*] \{\Delta z\} = \{\Delta F^*\} \quad (2.16)$$

where

$$[K^*] = \left[[M] \left(\frac{6}{\Delta t^2} + \frac{3\alpha}{\Delta t} \right) + [K] \left(\frac{3\beta}{\Delta t} + 1 \right) \right]$$

$$\text{and } \{\Delta F^*\} = [M] \left[-\{\Delta \ddot{y}\} + \{\ddot{z}_0\} \left(\frac{3 + \alpha \Delta t}{2} \right) + \{\dot{z}_0\} \left(\frac{6 + 3\alpha}{\Delta t} \right) \right] \\ + \beta [\gamma] \left[3\{\dot{z}_0\} + \frac{\Delta t}{2} \{\ddot{z}_0\} \right]$$

are commonly referred to as the effective stiffness matrix and the effective incremental joint load vector respectively.

Similarly if Eqs. 2.11, 2.12, and 2.14 are substituted into Eq. 2.2 and the resulting expression is rearranged, then the following expression is obtained for the case of constant joint accelerations:

$$\left[[M] \left(\frac{4}{\Delta t^2} + \frac{2\alpha}{\Delta t} \right) + [K] \left(\frac{2\beta}{\Delta t} + 1 \right) \right] \{\Delta z\} = \\ [M] \left[-\{\Delta \ddot{y}\} + 2\{\ddot{z}_0\} + \{\dot{z}_0\} \left(\frac{4}{\Delta t} + 2\alpha \right) \right] + 2\beta [K] \{\dot{z}_0\} \quad (2.17)$$

or again as:

$$[K^*] \{\Delta z\} = \{\Delta F^*\}$$

where

$$[K^*] = \left[[M] \left(\frac{4}{\Delta t^2} + \frac{2\alpha}{\Delta t} \right) + [K] \left(\frac{2\beta}{\Delta t} + 1 \right) \right]$$

and

$$\{\Delta F^*\} = [M] \left[-\{\Delta \ddot{y}\} + 2\{\ddot{z}_0\} + \{\dot{z}_0\} \left(\frac{4}{\Delta t} + 2\alpha \right) \right] + 2\beta [K] \{\dot{z}_0\}$$

again represent the effective values.

Finally, if $[K^*]$ changes within an integration step and correction forces are applied at the end of the step as negative joint loads then the general form of the force-displacement function can be further generalized as follows:

$$[K^*] \{\Delta z\} = \{\Delta F^*\} - \{U\} \quad (2.18)$$

where $\{U\}$ represents the correction forces or equilibrium corrections as they are frequently called. The matrix $[K^*]$ and the joint load vectors $\{\Delta F^*\}$ and $\{U\}$ are defined for each step of the process, and thus the incremental joint displacements may be easily determined by any one of many methods. The incremental joint velocities and accelerations are then found by substituting the known values of the incremental joint displacements into Eqs. 2.10 or 2.13. The total response then at the end of a step is simply the sum of the total response at the beginning of the step and the incremental values determined for the step, i.e.:

$$\{z_1\} = \{z_0\} + \{\Delta z\} \quad (2.19)$$

$$\{\dot{z}_1\} = \{\dot{z}_0\} + \{\Delta \dot{z}\} \quad (2.20)$$

$$\{\ddot{z}_1\} = \{\ddot{z}_0\} + \{\Delta \ddot{z}\} \quad (2.21)$$

alternatively $\{\ddot{z}_1\}$ may be found directly from Eq. 2.1.

2. EVALUATION OF THE STIFFNESS MATRIX $[K^*]$

The stiffness matrix is perhaps the most important parameter of Eq. 2.18. Certainly it requires the most work to define. The following section then, discusses the steps required to assemble $[K^*]$. These steps may be summarized as follows:

1. Evaluate the moment-axial load-curvature relationship for each section.
2. From the results of 1. above, derive the rotations at the member ends, given a moment pattern.
3. Define the element stiffness matrices taking into consideration both the elastic and inelastic effects.
4. Transform these element matrices to the member's rigid ends if the joint dimensions are considered finite.
5. Transform the matrices defined in 4. from the member system to the structural (global) system.
6. Add the p-delta effects if required.
7. Take into consideration the damping constants α and β .

2.3.1 MEMBER STIFFNESS MATRIX $[k]$

Assume for the while that steps 1 and 2, section 2.3 have been completed, and consider the case of a member subjected to varying end moments that are of the same sign but not necessarily the same value

and experiencing inelastic deformations as indicated in Fig. 2.3. It is immediately obvious that at any instant the moment diagram may be easily transformed into a curvature diagram as shown. The curvature diagram then, may be conceptually partitioned into elastic and inelastic areas as shown. If it is further postulated that the curvature diagram can be decomposed into two elementary curvature diagrams, each associated with application of a particular end moment, then, according to the moment area theorem [50], by integrating the elastic and the inelastic areas separately over their respective lengths the following expressions relating end moments and end rotations (deformations) result:

$$\theta_L^1 = (F_L^{E1} + F_L^{I1}) M_L \quad (2.22)$$

$$\theta_R^1 = (F_R^{E1} + F_R^{I1}) M_L \quad (2.23)$$

$$\theta_L^2 = (F_L^{E2} + F_L^{I2}) M_R \quad (2.24)$$

$$\theta_R^2 = (F_R^{E2} + F_R^{I2}) M_R \quad (2.25)$$

where θ , M , and F represent end rotations, end moments, and flexibility coefficients respectively. The superscripts E, I, 1, and 2 denote the elastic contribution, the inelastic contribution and the loading cases respectively and finally the subscripts L, and R refer to the left or the right hand end of the member.

DEFORMED MEMBER

ARBITRARY MOMENT DIAGRAM

CORRESPONDING CURVATURE DIAGRAM

ELEMENTARY CURVATURE DIAGRAMS

FIG. 2.3 INELASTIC MOMENT-CURVATURE-ROTATION RELATIONSHIP

Formation of the left end and the right end rotation components for the two cases yields the following matrix expression:

$$\begin{Bmatrix} \theta_L \\ \theta_R \end{Bmatrix} = \begin{bmatrix} \frac{E1}{F_L} & \frac{E2}{F_L} \\ \frac{E1}{F_R} & \frac{E2}{F_R} \end{bmatrix} + \begin{bmatrix} F_L^{11} & F_L^{12} \\ F_R^{11} & F_R^{12} \end{bmatrix} \begin{Bmatrix} M_L \\ M_R \end{Bmatrix} \quad (2.26)$$

where θ_L and θ_R are the total rotations at the left and the right hand ends respectively. It is to be noted that the member flexibility matrix has now been arranged into elastic and inelastic submatrices.

2.2.1.1 ELASTIC-FLEXIBILITY COEFFICIENTS

Expressions for the elastic flexibility coefficients that reflect shear deformations [51] may be easily determined. Consider Fig. 2.4. Use of the Method of Fictitious Loads [50] with the separate application of a moment at the left and the right end yields the following expressions:

$$\theta_L^L = \frac{M_L \ell}{3EI} + \frac{M_L}{2A_e G} \quad (2.27)$$

$$\theta_R^L = \frac{M_L \ell}{6EI} + \frac{M_L}{2A_e G} \quad (2.28)$$

$$\theta_L^R = -\frac{M_R \ell}{6EI} + \frac{M_R}{2A_e G} \quad (2.29)$$

$$\theta_R^R = \frac{M_R \ell}{3EI} + \frac{M_R}{2A_e G} \quad (2.30)$$

FIG. 2.4 DETERMINATION OF MEMBER ELASTIC FLEXIBILITY COEFFICIENTS

where ℓ , E , I , A_e , and G denote member length, Young's modulus, moment of inertia, effective shear area and the shear modulus respectively.

The superscripts L, or R denote whether the rotation results from application of the left or the right end moment. The subscripts L or R refer to the left or the right end.

The above expressions can be represented in matrix form as follows:

$$\begin{Bmatrix} \theta_L \\ \theta_R \end{Bmatrix} = \begin{bmatrix} \frac{\ell}{3EI} + \frac{1}{\ell A_e G} & -\frac{\ell}{6EI} + \frac{1}{\ell A_e G} \\ -\frac{\ell}{6EI} + \frac{1}{\ell A_e G} & \frac{\ell}{3EI} + \frac{1}{\ell A_e G} \end{bmatrix} \begin{Bmatrix} M_L \\ M_R \end{Bmatrix} \quad (2.31)$$

where θ_L and θ_R are total rotations (deformations) at the left and the right ends.

Setting $\psi = \frac{6EI}{\ell A_e G}$ (ψ is commonly called the shear flexibility factor), and inverting the above flexibility matrix it can be shown that the resulting member stiffness matrix can be expressed as:

$$\frac{2EI}{\ell(1+2\psi)} \begin{bmatrix} (2+\psi) & (1-\psi) \\ (1-\psi) & (2+\psi) \end{bmatrix} \quad (2.32)$$

or as:

$$\frac{2EI}{\ell} \begin{bmatrix} 2 & 1 \\ 1 & 2 \end{bmatrix} \frac{1}{1+2\psi} \begin{bmatrix} 1+\psi & -\psi \\ -\psi & 1+\psi \end{bmatrix} \quad (2.33)$$

Finally, Eq. 2.33 can be expressed in its flexibility form as:

$$\begin{bmatrix} 1+\psi & \psi \\ \psi & 1+\psi \end{bmatrix} \frac{\ell}{3EI} \begin{bmatrix} 1 & -1/2 \\ -1/2 & 1 \end{bmatrix} \quad (2.34)$$

For the particular case where one of the member ends is pinned say the left (or lower) end, it can be shown that with application of the Method of Substructures [52], Eq. 2.34 reduces to:

$$\begin{bmatrix} 0 & 0 \\ 0 & 2+\psi \end{bmatrix} \frac{l}{3EI} \begin{bmatrix} 0 & 0 \\ 0 & 1/2 \end{bmatrix} \quad (2.35)$$

A similar expression can be given when the right end is pinned by interchanging the main diagonal elements in both matrices.

2.3.1.2 INELASTIC FLEXIBILITY COEFFICIENTS

Expressions for the inelastic flexibility coefficients of reinforced concrete members are extremely difficult to evaluate. In fact, at present there does not exist any precise method that takes into consideration the general situation whereby both the numerical values of the end moments and their signs differ instant by instant. The essential difficulty of the modelling problem has been well expressed by Takizawa [4] and Sozen [14]. Concrete is a non homogeneous material.

The structure of the molecular bonds of the cement paste and the aggregate are subject to fracture. The stress-strain relationship of every bond varies depending upon the history of the loading and there are sudden shifts in the location of the inflexion point. If for example that a member had experienced severe anti-symmetric distress and then unloaded to the point where the moments were those normally associated with elastic behaviour as shown in Fig.2.5 one might assume that elastic flexibility coefficients would be appropriate in this

FIG. 2.5 MEMBER STRAINED BEYOND THE ELASTIC LEVEL

situation. In fact, although portions of the member might well be elastic, that portion that had suffered the greatest distress would most assuredly not be elastic due to the presence of cracks, and hence greater stiffness would be attributed to the member than should be warranted.

The method adopted in this study was to make the not unreasonable assumption that the inflexion point for line type members subjected to end moments only, tends to be located near the member's center during the entire response. If the inelastic rotations are considered concentrated or "lumped" at the member's ends the member can be represented as an elastic line element with inelastic "springs" at either ends, as noted in Chapter 1. This being so the off-diagonal terms of the inelastic flexibility sub matrix in Eq.2.26 are zero. The flexibility of these springs can be derived by examining the behaviour of an identical member subject to varying anti-symmetrical moments during an experiment [12] or by theory [18]. The behaviour can be rationalized as a system of rules to be applied in a particular study at each stage of the loading. These rules will be discussed in detail in Chapter 3.

2.3.1.3 SPRING MOMENT-ROTATION PRIMARY CURVE

Fig.2.6 represents a typical moment-axial-load-curvature

FIG. 2.6 MOMENT-AXIAL LOAD-CURVATURE PRIMARY CURVE

relationship at a section of a typical reinforced concrete member.

Park and Paulay [52] present an excellent discussion on a mathematical procedure for calculating the cardinal points on the curve viz the cracking, yielding, and the ultimate points, given the cross section, dimensions, the amount, grade and location of the reinforcement, Young's modulus and the stress-strain curves of both the concrete and the reinforcement, the ultimate strength of the concrete the modular ratio and the amount of axial load present.

The assumptions underlying this method are:

1. Plane sections before bending remain plane after bending.
2. The strains in the reinforcement and the concrete at the same level are identical. *✓*
3. The maximum useful compressive concrete strain is 0.004.

It is to be noted that immediately after the cracking curvature is reached there is a sudden increase in curvature without increase in moment. This phenomenon results from redistribution of the internal forces to accommodate the section depth reduced by the cracking [14].

There is also a sudden shift when the reinforcement yields. It should be noted also, that unlike structural steel, the effect of increasing the axial load is to increase the ultimate moment provided that the maximum load is less than the balanced load.

For mathematical convenience it is usual to represent the curvilinear relationship as piecewise linear indicated as a broken

line in Fig. 2.6. The curve has been adopted in this study. The moment-rotation relationship at the member ends may be derived by:

1. Applying anti-symmetrical moments to the member at the cracking, yielding and ultimate level in turn.
2. Transforming each moment diagram into a curvature diagram.
3. Applying the moment-area theorem to each curvature diagram to obtain the rotations.

Park and Paulay have suggested certain modifications to account for material properties between cracked sections, etc. These modifications have not been employed in this study.

The resulting moment-rotation diagram is trilinear. Sometimes a bilinear representation is used. Referring to Fig. 2.7, the following expressions for the cracking, yielding, and ultimate rotations at the left or right ends, and in both loading directions are:

$$\theta_c = \frac{M_c l}{6EI} \quad (2.36)$$

$$\theta_y = \frac{l}{6} [(1-\lambda^3) \phi_y + \lambda^2 \phi_c] \quad (2.37)$$

$$\text{where } \lambda = \frac{M_c}{M_y} \quad (2.38)$$

$$\text{and } \theta_u = \frac{l}{12} [\{ (2+\lambda_2) (1-\lambda_2) (1+\lambda_2(\lambda_3-1)) / \lambda_3 + \lambda_2(1+\lambda_2) - 2\lambda_1^3 \} \phi_y / \lambda_2 + 2\lambda_1^2 \phi_c] \quad (2.39)$$

FIG. 2.7 DERIVATION OF CRACKING, YIELDING, AND ULTIMATE ROTATIONS

$$\text{where } \lambda_1 = \frac{M_c}{M_u} \quad (2.40)$$

$$\lambda_2 = \frac{M_y}{M_u}, \text{ and} \quad (2.41)$$

$$\lambda_3 = \frac{(M_u - M_y)}{(\phi_u - \phi_y)} \left(\frac{\phi_y}{M_y} \right) \quad (2.42)$$

ϕ represents the curvature, and the subscripts c, y, and u represent the cracking, yielding, and ultimate condition respectively.

It is assumed in the above expressions that:

1. The top and the bottom reinforcement areas are constant over the length of the member.
2. The grade, area, and location of the top and bottom reinforcement is identical.

If however, assumption 2 above is violated then the following expressions represent modifications to Eqs. 2.36, 2.37, and 2.39 for left end spring values viz

$$-\frac{l}{24} \left(1 - \frac{M_c^R}{M_c^L} \right)^2 \cdot (M_c^L + 2M_c^R) \cdot \left(\frac{\phi_y^R}{M_y^R} - \frac{1}{EI} \right) \quad (2.43)$$

for the case when the left end cracking moment exceeds the right end cracking capacity,

$$\frac{l}{24} \left[\left(1 - \frac{M_c^L}{M_c^R} \right)^2 \cdot (M_y^L + 2M_c^L) \cdot \left(\frac{\phi_y^L}{M_y^L} - \frac{1}{EI} \right) - \left(1 - \frac{M_c^R}{M_y^L} \right)^2 \right] \quad (2.44)$$

$$(M_y^L + 2M_c^R) \cdot \left(\frac{\phi_y^R}{M_y^R} - \frac{1}{EI} \right) \Big]$$

for the case when the left end yielding moment is less than the right end yielding capacity,

expression 2.44 minus:

$$\frac{l}{24} (M_y^L - M_y^R)^3 \cdot (\phi_u^R M_y^R - \phi_y^R M_u^R) / ((M_y^L)^2 \cdot M_y^R \cdot (M_u^R - M_y^R)) \quad (2.45)$$

for the case when the left end yielding moment exceeds the right end yielding capacity,

$$\begin{aligned} & \frac{l}{24} \left[\left(1 - \frac{M_c^L}{M_u^L}\right)^2 \cdot (M_u^L + 2M_c^L) \cdot \left(\frac{\phi_y^L}{M_y^L} - \frac{1}{EI}\right) + \left(1 - \frac{M_y^L}{M_u^L}\right)^2 \cdot \right. \\ & \left. \left(\phi_u^L - \phi_y^L \frac{M_u^L}{M_y^L}\right) - \left(1 - \frac{M_c^R}{M_u^R}\right)^2 \cdot (M_u^R + 2M_c^R) \cdot \left(\frac{\phi_y^R}{M_y^R} - \frac{1}{EI}\right) \right] \end{aligned} \quad (2.46)$$

for the case when the left end ultimate moment is less than the right end yielding capacity and expression 2.46 minus:

$$\frac{l}{24} (M_u^L - M_y^R)^3 \cdot (\phi_u^R \cdot M_y^R - \phi_y^R \cdot M_u^R) / ((M_u^L)^2 \cdot M_y^R \cdot (M_u^R - M_y^R)) \quad (2.47)$$

for the case when the left end ultimate moment exceeds the right end ultimate capacity.

Referring to Fig.2.8, it can be seen that the moment-rotation relationship is no longer trilinear. The right hand spring properties are equal to the interchanged positive and negative left end values.

TENSION AND COMPRESSION REINFORCEMENT EQUAL

TENSION AND COMPRESSION REINFORCEMENT UNEQUAL

FIG. 2.8 MOMENT-ROTATION RELATIONSHIP

Should the shear effects be considered important then the rotations so derived must be further modified. There appears to be little research into this area. Both Takayanagi and Schnobrich [25], and Takizawa [4] have reported on the problem, and the approach adopted in the program is to assure that the ratio of the sum of the elastic flexural and shear rotations at cracking to the flexural rotation at cracking remains constant throughout the entire loading.

2.2.1.4 JOINT DIMENSIONS

Reinforced concrete members tend to be larger than corresponding members of a steel frame, therefore the presence of finite beam-column joints may have a not insignificant effect on member stiffness and hence structure stiffness, particularly if some of the members are shear walls.

Apparently it is usual for many researchers to consider the joint as being rigid, and thus consider the distance from a joint's center line to its face to be a rigid extension of a member framing into the joint perpendicular to the face. Hall [54] considered the question of the true length of a rigid zone in some detail. He presented a rather unusual procedure to obtain the length. The usual representation noted above has been used in this study however:

Consider the sketch shown in Fig. 2.2. From geometric considerations it can be shown that:

$$\theta'_L = \theta_L + v_L \theta'_L + v_R \theta'_R \quad (2.48)$$

$$\theta'_R = \theta_R + v_L \theta'_L + v_R \theta'_R \quad (2.49)$$

or in matrix form as:

$$\begin{bmatrix} \theta'_L \\ \theta'_R \end{bmatrix} = \begin{bmatrix} 1+v_L & v_R \\ v_L & 1+v_R \end{bmatrix} \begin{bmatrix} \theta_L \\ \theta_R \end{bmatrix} \quad (2.50)$$

where θ and v refer to the rotation and the ratio of the rigid zone length to the flexible length. The superscript refers to the value at the outer end of the rigid zone, and the subscripts L and R again refer to the left and the right end of the member. Again from Fig. 2.9 it can be shown that the relationship between applied moments at the outer end of the rigid zone and the moments at the end of the flexible member can be expressed by the following matrix equation:

$$\begin{bmatrix} M'_L \\ M'_R \end{bmatrix} = \begin{bmatrix} 1+v_L & v_L \\ v_R & 1+v_R \end{bmatrix} \begin{bmatrix} M_L \\ M_R \end{bmatrix} \quad (2.51)$$

If Eqs. 2.26, 2.50, and 2.51 are combined and the general expressions for the flexibility coefficients replaced by the particular expressions developed before, the following matrix equation results:

$$\begin{bmatrix} M'_L \\ M'_R \end{bmatrix} = \begin{bmatrix} 1+v_L & v_L \\ v_R & 1+v_R \end{bmatrix} \begin{bmatrix} 1+\psi & \psi \\ \psi & 1+\psi \end{bmatrix} \frac{l}{3EI} \begin{bmatrix} 1 & -1/2 \\ -1/2 & 1 \end{bmatrix} + \begin{bmatrix} F_L^{I1} & 0 \\ 0 & F_R^{I2} \end{bmatrix}^{-1} \begin{bmatrix} 1+v_L & v_R \\ v_L & 1+v_R \end{bmatrix} \begin{bmatrix} \theta'_L \\ \theta'_R \end{bmatrix} \quad (2.52)$$

$(V_L = V_R = V)$

FIG. 2.9 RIGID END EFFECTS

Eq. 2.52 represents the member stiffness matrix $[k]$ in the member coordinate system.

2.3.2 TRANSFORMED MEMBER STIFFNESS MATRIX

The relationship between member deformations and joint displacements can be readily derived. From Fig. 2.10 adopting the convention shown, it is a simple matter to show that for the case where a member is permitted to rotate and sway:

$$\begin{bmatrix} \theta_L' \\ \theta_R' \\ \Delta_L' \\ \Delta_R' \end{bmatrix} = \begin{bmatrix} 1 & 0 & -\frac{1}{L} & \frac{1}{L} \\ 0 & 1 & -\frac{1}{L} & \frac{1}{L} \end{bmatrix} \begin{bmatrix} \theta_L \\ \theta_R \\ \Delta_L \\ \Delta_R \end{bmatrix} \quad (2.53)$$

where θ , Δ , and L represent joint rotations, joint sways and the total length of member including the rigid zones respectively.

Similarly it can be shown that the relationship between joint moments and shears, and member moments can be expressed as:

$$\begin{bmatrix} M_L' \\ M_R' \\ V_L' \\ V_R' \end{bmatrix} = \begin{bmatrix} 1 & 0 \\ 0 & 1 \\ -\frac{1}{L} & -\frac{1}{L} \\ \frac{1}{L} & \frac{1}{L} \end{bmatrix} \begin{bmatrix} M_L \\ M_R \end{bmatrix} \quad (2.54)$$

where V represents the joint shears.

Combining Eqs. 2.52, 2.53 and 2.54 there results:

MEMBER DEGREES OF FREEDOM SHOWN
WITH POSITIVE SENSE

FIG. 2.10 TRANSFORMATION FROM MEMBER TO GLOBAL
COORDINATES

$$\begin{Bmatrix} M_L \\ M_R \\ V_L \\ V_R \end{Bmatrix} = [K] \begin{Bmatrix} \theta_L \\ \theta_R \\ \Delta_L \\ \Delta_R \end{Bmatrix} \quad (2.55)$$

in which:

$$[K] = \begin{bmatrix} 1 & 0 \\ 0 & 1 \\ -\frac{1}{L} & -\frac{1}{L} \\ \frac{1}{L} & \frac{1}{L} \end{bmatrix} \begin{bmatrix} 1+\nu_L & \nu_L \\ \nu_R & 1+\nu_R \end{bmatrix} \begin{bmatrix} 1+\psi & \psi \\ \psi & 1+\psi \end{bmatrix} + \frac{\ell}{3EI} \begin{bmatrix} 1 & -1/2 \\ -1/2 & 1 \end{bmatrix} \quad (2.56)$$

$$\begin{bmatrix} F_L^{I1} & 0 \\ 0 & F_R^{I2} \end{bmatrix}^{-1} \begin{bmatrix} 1+\nu_L & \nu_R \\ \nu_L & 1+\nu_R \end{bmatrix} \begin{bmatrix} 1 & 0 & -\frac{1}{L} & \frac{1}{L} \\ 0 & 1 & -\frac{1}{L} & \frac{1}{L} \end{bmatrix}$$

is the member stiffness matrix expressed in the structure (global) coordinate system.

If the member is considered as axially extensible then Eq. 2.55 can be augmented as follows:

$$\begin{Bmatrix} M_L \\ M_R \\ V_L \\ V_R \\ P_L \\ P_R \end{Bmatrix} = \begin{bmatrix} [K] & 0 & 0 \\ 0 & 0 & 0 \\ 0 & 0 & 0 \\ 0 & 0 & 0 \\ 0 & 0 & 0 \end{bmatrix} \begin{Bmatrix} \theta_L \\ \theta_R \\ \Delta_L \\ \Delta_R \\ \epsilon_L \\ \epsilon_R \end{Bmatrix} \quad (2.57)$$

in which P and ϵ represent the axial load and extension respectively.

The axial stiffness coefficients, $\frac{AE}{L}$ are assumed to be elastic at all times. The augmented form of $[k]$ will be called $[k]'$.

In this study the columns are considered as extensible, thus they require the above transformation*. However, the beams are considered axially rigid and hence require no further transformation. It is to be noted that Eq. 2.56 is perfectly valid for the case when the joint displacements are incremental, provided that the inelastic flexibility coefficients are appropriately defined.

Finally the entire stiffness matrix is assembled member by member through the direct correspondence between the structural coordinate system and the member degrees of freedom.

2.3.3 THE P-DELTA EFFECTS

There are several secondary effects associated with deformed frames [55]. They are the so called p- Δ , p- δ , and p-x effects and are illustrated in Fig.2.11. The p- Δ forces result from gravity loads acting through the displaced joints. They increase the story shears and can have a significant effect on structural behaviour as mentioned in Chapter 1. The p- δ effects result from axial loads acting on the deformed member. Internal member moments and shears are increased accordingly, but these increases are usually insignificant for most practical structures. The p-x effects augment the overturning moments. Only the p- Δ , and p-x effects have been included in this study.

* but with $-\frac{1}{L}$ and $\frac{1}{L}$ interchanged.

FIG. 2.11 SECONDARY EFFECTS IN A STRUCTURE

There are two philosophies with respect to handling the $p-\Delta$ effects:

1. An iterative procedure involving the continued adjustment of $p-\Delta$ shears until the displacements calculated in successive static analyses, converge.
2. The use of the concept of "initial stress" or "geometric" stiffness, which depends upon the total member forces and not the incremental forces associated with deformations.

Consider the free body diagram of two displaced columns as shown in Fig. 2.12. It is a simple matter to show that the additional shear required to be resisted by the structure can be expressed as:

$$F_i = (Z_i - Z_{i-1}) \frac{P_i}{h_i} - (Z_{i+1} - Z_i) \frac{P_{i+1}}{h_{i+1}} \quad (2.58)$$

in which F , P , h and z refer to the additional floor shear, accumulated gravity load, floor to floor height, and sway at the floor i under consideration.

Rearranging the right side:

$$F_i = -\frac{P_i}{h_i} Z_{i-1} + \left(\frac{P_{i+1}}{h_{i+1}} + \frac{P_i}{h_i} \right) Z_i - \frac{P_{i+1}}{h_{i+1}} Z_{i+1} \quad (2.59)$$

FIG. 2:12 P-DELTA SHEARS

In matrix notation Eq. 2.59 can be rewritten as:

$$\begin{Bmatrix} \vdots \\ F_i \\ \vdots \end{Bmatrix} = \begin{bmatrix} \vdots & \vdots & \vdots \\ -\frac{P_i}{h_i} & \left(\frac{P_{i+1}}{h_{i+1}} + \frac{P_i}{h_i} \right) & -\frac{P_{i+1}}{h_{i+1}} \\ \vdots & \vdots & \vdots \end{bmatrix} \begin{Bmatrix} \vdots \\ z_{i-1} \\ z_i \\ z_{i+1} \\ \vdots \end{Bmatrix} \quad (2.60)$$

The coefficients thus determined are derived for all floors and are added with signs reversed to the associated coefficients of the structure stiffness matrix. It is to be noted that if the gravity loads can be assumed constant throughout the earthquake, then the coefficients also remain constant.

2.3.4 THE DYNAMIC TERMS

So far only the static structure stiffness matrix has been constructed. Reference to Eq. 2.16 indicates additional terms arising from the presence of damping.

To date a true understanding of the mechanism of damping is still forthcoming. Hurty and Rubinstein [56] expressed this point

very well. It is recognized however, that damping assists both in resisting the imposed forces and in energy dissipation. Most researchers consider that the damping effects can be well represented by a "viscous" type model, where by the coefficients can be expressed as a linear combination of mass and stiffness. Accordingly, it is usual to see damping treated at the structure level rather than the member level, and therefore the damping coefficient matrix set as a linear combination of the mass matrix and the structure stiffness matrix.

As it is usual to consider only the horizontal behaviour of a structure, the mass of the structure is considered lumped at the joints and accelerated only in the horizontal direction, and is thus a diagonal matrix in sway terms only. Similarly all the terms of the structure stiffness matrix not associated with sway are "reduced" out. If a modal or an elastic dynamic analysis of the structure is required then the work involved in reducing the matrix is certainly worth while. If however, an inelastic analysis is contemplated that involves joint rotations and column extensions, then the procedure in terms of computational efficiency is questionable.

The tactic employed by the computer program supporting this thesis, is to multiply the terms of the unreduced stiffness matrix by the damping stiffness constant β . It is felt that the computations can be performed more accurately and quickly. It is recognized however, that this approach

extends the artificial nature of the viscous model further, and it has been found necessary to use the constant acceleration method, if the β term is used.

The damping constants; α and β are easily determined from the following expressions:

$$\left. \begin{aligned} \alpha &= \frac{2 \omega_1 \omega_2 (\xi_2 \omega_1 - \xi_1 \omega_2)}{\omega_1^2 - \omega_2^2} \\ \beta &= \frac{2 (\xi_1 \omega_1 - \xi_2 \omega_2)}{\omega_1^2 - \omega_2^2} \end{aligned} \right\} \quad (2.61)$$

when both mass and stiffness are considered, or:

$$\left. \begin{aligned} \alpha &= 0 \\ \beta &= 2 \frac{\xi_1}{\omega_1} \end{aligned} \right\} \quad (2.62)$$

when stiffness only is considered, or:

$$\left. \begin{aligned} \alpha &= 2 \xi_1 \omega_1 \\ \beta &= 0 \end{aligned} \right\} \quad (2.63)$$

when mass only is considered.

The terms ω_1 and ω_2 represent the two undamped non-dimensional frequencies (usually those of the first and second mode), and ξ_1 and ξ_2 are the associated modal damping factors.

2.3.5 THE SCOMP FORM OF THE STRUCTURE STIFFNESS MATRIX

The computer storage requirements for a structural system involving many degrees of freedom can be very large particularly if the stiffness coefficients are stored in double precision format. Fortunately, however, the stiffness matrix is both symmetric and banded, and thus all information required to define the matrix can be stored in a smaller array of dimensions equal to the number of equations and the half bandwidth.

There are several storage schemes; one that stores all zeros embedded within the half bandwidth and one that does not. The first scheme is used here. The main diagonal is stored in the first column, thus if the address of an element in the original matrix is i, j , where i is the row, and j is the column, then the address of the element stored in the compact form is $i, j-i+1$.

2.4 JOINT LOADS

The two terms on the right hand side of Eq. 2.18 consist of joint loads computed for the interval under consideration and equilibrium corrections carried over from the previous step.

Most of the terms of Eq. 2.15 or 2.17 require no further explanation. However, it is useful to discuss the background of the acceleration records; $\ddot{y}$ used in this study. As mentioned in Chapter 1,

the generality of conclusions can be undermined if only one or several earthquake records is used in a study.

Newmark, Flume, and Kanur investigated the spectral response of an oscillator to some thirty earthquake records. They thus determined a particular spectrum that represented a 90% probability of not being exceeded by the records. Tsai [57] presented a method for "filtering" any record so that spectral response to the resulting wave form would closely comply with a chosen target response. The acceleration records used in this study are derived by "filtering" the 1940 El Centro N.S. component to comply with the spectrum derived by Newmark et al at 5% of critical damping.

Inherent with numerical integration is the assumption that member stiffness and hence the structure stiffness remains constant during a time step. Sometimes, however, a member "overshoots" from one stiffness regime to another whilst loading. Hence the predicted response is in error. If the "overshoot" is excessive, the validity of the response is seriously undermined at best and meaningless at worst. Several methods for correcting this problem are [58,7]:

1. Retrace the response using shorter time steps.
2. Use iterative procedures to satisfy equilibrium requirement within a time step.
3. Use joint loads to correct for equilibrium imbalances.
4. Ignore the overshoot.

5. Combine any of the first three methods:

For degrading type hysteresis models where very small loops may result, method 4 must be ruled out. Method 2 is appealing but the rate of convergence may be slow and thus the entire process time consuming. Thus method 1 might be more useful. However, it suffers from the fact that the yield criteria is not enforced. Method 3 by itself does not permit the use of large integration intervals because there is then the problem of serious disturbances being carried forward. Thus method 5 has been used in this study and is a combination of methods 1 and 3.

If stiffness dependent damping is included, then revision of the structure stiffness matrix at the end of a step causes a discontinuity in the damping terms for the next step [55].

In general, the corrections manifest themselves in three ways:

1. Unbalanced joint moments.
2. Floor-floor shears.
3. Bay-bay shears when columns are treated as being axially deformable.

Corrections for type 1 and 2 have been incorporated into the computer program.

Joint loads also result when loads are applied along the length

of a member. They are found by summation at the fixed end forces at the joints. The fixed end moments for the two simple cases of:

1. Uniformly distributed load, and
2. Concentrated loads,

are with reference to Fig. 2.13:

$$\begin{aligned} M_L^F &= -M_R^F = -\frac{wl^2}{12} \\ M_L^F &= -\frac{Pab^2}{l^2} \\ M_R^F &= \frac{Pa^2b}{l^2} \end{aligned} \quad (2.64)$$

in which the superscript denotes fixed end forces. If the shear effects are considered important then it can be shown that the following equations apply:

$$\begin{aligned} M_L^F &= -M_R^F = -\frac{wl^2}{12} \quad (\text{as above}) \\ M_L^F &= -\frac{Pab}{l^2} \left(\frac{b+\psi l}{1+2\psi} \right) \\ M_R^F &= \frac{Pab}{l^2} \left(\frac{a+\psi l}{1+2\psi} \right) \end{aligned} \quad (2.65)$$

where a represents the distance of the load P from the left end and b represents the distance to the right end.

The associated shears are derived simply from:

$$\frac{M_L^F + M_R^F}{l} \quad (2.66)$$

FIG. 2.13 JOINT LOADS FROM GRAVITY LOADS

For the case when the joint dimensions are finite, the moments only need to be transformed to the joints using the transformation matrix of Eq. 2.51.

Eqs. 2.64 and 2.65 are only true if the structure behaves elastically. If an inelastic study is contemplated that uses a bilinear spring primary curve, and if the gravity loads are defined at the working level, no difficulty should arise. However, if the primary curve is trilinear as is the case usually adopted for reinforced concrete members, then the possibility exists that the members will crack rendering the evaluation of the internal forces and rotations very difficult. Prior to a seismic analysis it is imperative that equilibrium and compatibility be established otherwise serious disturbances may be carried forward from the static analysis stage to the seismic stage. Several researchers have reported on this problem [58,7] and suggested approximate methods to deal with the situation. However, gravity loads have not been included in this study.

CHAPTER 3

METHOD OF ANALYSIS

3.1 INTRODUCTION

Chapter 2 discussed various details taken into consideration when deriving the equations of motion. This chapter examines the resulting mathematical model from the stand point of practical application of the theory to an efficient method of dynamic analysis.

It must be restated that the actual behaviour of even a simple building subject to seismic loading is very complex. The true nature of the ground motion, and the behaviour and influence of both the structural and non structural materials can only be modelled approximately at this time. None the less use of such models can yield valuable information on the appropriateness of competing structural systems, identification of parts having excessive ductility demand, estimates of damage, and the adequacy of various building code provisions.

Thus follows a declaration of the assumptions underlying this study, an examination of the problems of selecting an appropriately sized integration interval, a discussion on the rules employed to govern stiffness degrading behaviour, a discussion on ductility demand, examination of the requirements of modern computer programs, and a

discussion on the capabilities of the program used.

3.2 ASSUMPTIONS UNDERLYING AN ANALYSIS

The following assumptions are necessary to reduce a complex problem to a tractable one. They should therefore clarify the limitations and application of the procedure followed.

3.2.1 ASSUMPTIONS ABOUT A STRUCTURE

1. A structure consists of a set of parallel, rectangular, rigid jointed frames.
2. The frames are linked by floors that are rigid in the horizontal direction, and the height of the floors are the same in each constituent frame.
3. The centroidal axis of the columns must be straight and continuous from the base to the roof whether setback or not.
4. The centroidal axis of the beams must be straight and continuous.
5. The frames are connected (hinged or fixed) to an infinitely rigid foundation.
6. For the purposes of computing horizontal inertial forces, the dead load only contributes. Its mass is concentrated outside the structure and connected to the joints by rigid truss elements. Vertical, torsional and rotational inertial

forces are ignored.

7. Mass either dead or alive is considered invariant in time.
8. The damping forces in the structure can be adequately represented by a viscous type model, i.e., proportional to the floor velocities. The damping coefficients are considered to be a linear combination of the mass matrix and the instantaneous unreduced structure stiffness matrix.
9. The base excitation is a horizontal motion in the plane of the frames of constant intensity in the horizontal direction perpendicular to the frames. Therefore the frames vibrate equally in their own planes.
10. The structures resist all loading by essentially flexural behaviour.

3.2.2 ASSUMPTIONS ABOUT MEMBERS AND JOINTS

1. Compatibility of deformations and equilibrium of forces at a joint are satisfied.
2. The beam column effects of axial loads on member stiffness properties are negligible. The p-delta effects may be considered however.
3. Local and out of plane buckling is prevented.
4. Axial deformations of the beams are negligible.
5. Beam-column joints are entirely rigid.
6. Inelastic flexural deformation of a member is concentrated

and approximated by the deformation of two non linear rotational springs conceptually placed at the ends of the flexible part of the member.

7. The moment-rotation relationship of the springs are governed (for the case of degrading behaviour) by the rules developed by Takeda et al [12] .
8. The above relationship is not practically effected by the effects of shear pinching, strength decay, time dependent axial loads, gravity loads, joint deformations, joint crushing, bond destruction, and other miscellaneous causes of degradation.
9. The inflexion point for beams and columns lies at the member center throughout the excitation. Shear walls are thus excluded from inelastic studies.

3.3 THE SIZE OF THE INTEGRATION INTERVAL

There are several inherent difficulties associated with numerical integration. They are:

1. numerical stability...
2. truncation, round off and accumulated errors.
3. convergence characteristics of iterative type solutions.
4. disturbances resulting from change of stiffness.
5. excessive computational effort.

The degree to which they (except round off errors) manifest themselves depends for a given structure and earthquake upon the size of the interval selected. Unfortunately, the conditions for removal of one problem may be at odds with the conditions of another, for example, the need to accommodate the highest frequency of a structure may be at the expense of computational efficiency.

The equations of motion are solved directly in this study, thus convergence in that sense is not a consideration. Humar and Wright [47] indicated that the magnitude of the truncation error resulting from finite Taylor series expansion depends on the value $(\omega \Delta t)^p$, where p is the degree of the numerical integration formula used. If Δt is chosen to be less than $\frac{1}{\omega}$, the error is kept low. Round off errors have been reduced by the use of double precision arithmetic (64 Bits) in this study.

Accumulated errors result from the fact that the starting point at each step is incorrect and each step introduces an additional error. Depending upon the behaviour of the additional errors, the solution of the difference form of the differential equation will either oscillate progressively further from the true solution, become out of phase with the true solution, or follow it closely [59,47]. Elastic behaviour is well controlled if Δt is less than $\frac{N}{\omega}$, where N is a function of the particular approximating acceleration formula used and is $\sqrt{12}$ for the linear acceleration method.

Instability in the highest mode can eventually dominate the entire behaviour. Thus for a structure with a large number of degrees of freedom, stability rather than truncation error controls the value of Δt . The opposite is true for a structure with only several degrees of freedom.

Nonlinearities result from abrupt change from one stiffness state to another. Various difficulties occur [55, 58, 7], . These include spurious hinge formation, unaccounted changes in the elastic members, undershoot and overshoot. Overshoot occurs when a member passes from a certain stiffness regime to a less stiffer regime within a time step. If the overshoot is not controlled, the solution will become disturbed. Accordingly, if the study is repeated with progressively smaller intervals, the respective results will converge. Complete convergence however, may be at the expense of computational efficiency. Alternatively, if the overshoots are treated as joint loads to be subtracted from the joint load vector of the next step, and their values exceeds the incremental member forces, the solution will again be disturbed, and only a very small interval will reduce the problem to an acceptable level. Methods used for mitigating the effects of overshoot have been described in Chapter 2. No explicit methods have been used to mitigate the effects of the other aforementioned difficulties, other than suitable selection of the interval.

Experience then is required to select an appropriate interval.

Mahim [58] suggested that the interval should not exceed the smallest of:

1. one tenth of the period of the highest mode that has a significant effect on the elastic internal forces.
2. one fortieth of the first mode period.
3. the discretization interval of the earthquake records.

3.4 HYSTERESIS RULES

Experimental work by Clough and Takeda et al [11,12] indicates that reinforced concrete members do not exhibit a bilinear moment-rotation (hysteresis) relationship. Rather, the members degrade as a function of their stress history. Characteristically after several load cycles, a member will not reload parallel to the initial elastic slope but will aim towards the maximum moment attained in that direction at a previous cycle (Ref. Fig.3.1).

Sozen [14] presented a paper that examined in detail the mechanism of stiffness degradation. He confirmed what Clough had said earlier, that degrading behaviour is closely related to the crack closing phenomenon.

A brief summary of the behaviour is perhaps instructive. Consider a cantilever loaded monotonically in a clock wise (positive) direction

ORDINARY ELASTO-PLASTIC MODEL

DEGRADING STIFFNESS MODEL

FIG. 3.1 HYSTERESIS MODELS

as shown in Fig. 3.2. Initially the force displacement relationship is a straight line: oc and can be easily evaluated from elementary mechanics. After initial cracking there is an immediate redistribution of the internal forces and the member loads thereafter at a reduced slope to the point y where the tensile reinforcement starts to yield. The yielding slope: y_u depends primarily on the strain hardening properties of the reinforcement.

If the member is gradually unloaded, the slope u_D is influenced by the misfit of the cracked surfaces and its average value is less than the initial elastic value, and can be expressed as [14] :

$$k_{uD} = k_{oy} (D_{max})^{-\alpha}$$

where D_{max} is the ratio of the maximum displacement attained at loading to the yield displacement, and α is taken as 0.5.

On reloading, the slope is influenced by cracking on the tension side and closing of cracks on the compression side. Thus prior to tensile cracking, the slope is relatively large, but thereafter exhibits considerable reduction. The reloading slope for later cycles can be represented by an almost straight line joining the unloading point and the maximum moment point attained earlier. The remaining unloading slope for the first cycle is similar to the first unloading slope.

Bertero and Popov [15] concluded that the use of inclined web reinforcement in conjunction with vertical ties minimized the degrading effects and

FIG. 3.2 BEHAVIOUR OF A CYCLICALLY LOADED CANTILEVER

and sophisticated as a representation of the physical phenomenon.

In this light several available programs were considered as potentially useful to these studies. They are Drain-2D [45], SAKE [7], two programmes developed at Carleton University [33], and one recently developed at the University of Illinois [25]. After some consideration it was decided that the best course of action was to develop two new programs.

The larger of the two permits:

1. A static analysis.
2. A seismic analysis.
3. A combined static-seismic analysis.
4. A modal analysis.

The major features of the program are:

1. Either stiffness degrading or non degrading behaviour can be studied.
2. Either the linear or constant acceleration method may be used.
3. Column axial deformations are included with the rotational and translational degrees of freedom.
4. Where changes of linearity occur, the calculations may be retraced using smaller intervals. Unbalanced joint loads are computed and applied to the following step.

SPRING PRIMARY CURVE WITH ELASTIC AND INELASTIC ROTATIONS

SPRING-PRIMARY CURVE WITH INELASTIC ROTATIONS ONLY

FIG. 3.3 REPRESENTATION OF SPRING INELASTIC PROPERTIES

- 1: Rule 1
- 2: Rule 2
- 3: Rule 4
- 4: Rule 14
- 5: Rule 2
- 6: Rule 3
- 7: Rule 5
- 8: Rule 6

- 1: Rule 1
- 2: Rule 2
- 3: Rule 3
- 4: Rule 5
- 5: Rule 15
- 6: Rule 16
- 7: Rule 3
- 8: Rule 5
- 9: Rule 6

SEVERAL INITIAL POSSIBILITIES

FIG. 3.4 HYSTERESIS RULES

1: Rule 1	8: Rule 9
2: Rule 2	9: Rule 10
3: Rule 3	10: Rule 11
4: Rule 5	11: Rule 12
5: Rule 6	12: Rule 13
6: Rule 7	13: New Rule 10
7: Rule 8	

LOOPS REDUCING

1: Rule 1	7: Rule 8
2: Rule 2	8: Rule 9
3: Rule 3	9: Rule 10
4: Rule 5	10: New Rule 6
5: Rule 6	11: New Rule 7
6: Rule 7	12: New Rule 8

LOOPS REDUCING THEN INCREASING
- SEVERAL LATER POSSIBILITIES -

FIG. 3.4 (CONT'D) HYSTERESIS RULES

1: Rule 1	4a: Rule 4
2: Rule 2	5: Rule 15
3: Rule 3	6: Rule 3
4: Rule 5	7: Rule 5

- NON DEGRADING MODEL -

FIG. 3.4 (CONT'D) HYSTERESIS RULES

Rule 12: Loading after Rule 11 towards the maximum point attained during Rule 8.

Rule 13: Unloading from the maximum point attained during Rule 12.

Rule 14: Loading in the uncracked direction after cracking in the other direction.

Rule 15: Loading in the uncracked direction after yielding in the other direction.

Rule 16: Loading towards the yield point after Rule 15.

Non degrading behaviour (typical of structural steel members) can be achieved by applying only Rules 1, 2, 3, 4, 5 and 15. Obviously, the computational effort involved with the former behaviour can be much larger than the latter because the chances of linearity changes during an interval are so much greater. Thus the question may be asked as to under what circumstances is its use justified.

3.5 DUCTILITY DEMAND

For a structure to successfully withstand a severe earthquake, its members must be able to redistribute the loads accumulated at critical points by inelastic action. This requires that the members undergo substantial inelastic deformation without failure. The deformations should not be that severe as to render the building unusable however. Thus the ductility demand must be measured somehow and compared to a tolerable value [33,60]. The best measure of ductility demand is the maximum strain attained. However, in practice ductility is

measured in terms of either lateral displacements, curvature, or spring (hinge) rotations.

Displacement ductility demand μ can be expressed as:

$$\mu = 1 + \frac{\Delta_I}{\Delta_y} \quad (3.2)$$

where Δ_y represents the displacement of the building at the onset of yielding, and Δ_I represents the maximum inelastic excursion. This definition was appropriate for shear type building with elasto-plastic behaviour. However, the real ductility demand of the components was often masked, and Δ_y is difficult to define.

Curvature ductility demand is appropriate if the inelastic curvature is modelled as distributed over a finite length of the member. However, the inelastic curvature is considered concentrated at the member ends in this study, and so, the rotational ductility demand is used.

With reference to Fig.3.5, there are several possible ways to define this demand [3] :

1. The ratio of the maximum absolute rotation to the yield rotation.
2. The ratio of the total of the yield rotation and the maximum inelastic rotation (the latter defined wrt the average yield slope), to the yield rotation.

FIG. 3.5 DUCTILITY DEMAND DEFINITIONS

3. The ratio of the maximum absolute rotation to the maximum "elastic" rotation.

Method 1 is adopted here and the maximum (and minimum) spring rotations; $\theta_{\max(\min)}$, and the associated yield rotations are computed routinely in the computer program. The ductility ratios are represented then as:

$$\mu = \frac{\theta_{\max(\min)}}{\theta_y} \quad (3.3)$$

where θ_y is given by Eq.2.37.

The ratios are not modified if there are gravity loads present however.

3.6 COMPUTER PROGRAMS

Two computer programs have been written by the author to carry out the bulk of the computations required for this study. Some initial observations on computers and programs will be made at this stage however.

Computers are increasing both in addressable core size and in speed of processing. The cost of using them is dropping, and the operating system software, language translators and utility packages are improving in design.

In this context it becomes increasingly possible to devise more accurate mathematical models. Good models require that the algorithms be efficient in coding and fast in execution, mathematically stable,

and sophisticated as a representation of the physical phenomenon.

In this light several available programs were considered as potentially useful to these studies. They are Drain-2D [45], SAKE [7], two programmes developed at Carleton University [33], and one recently developed at the University of Illinois [25]. After some consideration it was decided that the best course of action was to develop two new programs.

The larger of the two permits:

1. A static analysis.
2. A seismic analysis.
3. A combined static-seismic analysis.
4. A modal analysis.

The major features of the program are:

1. Either stiffness degrading or non degrading behaviour can be studied.
2. Either the linear or constant acceleration method may be used.
3. Column axial deformations are included with the rotational and translational degrees of freedom.
4. Where changes of linearity occur, the calculations may be retraced using smaller intervals. Unbalanced joint loads are computed and applied to the following step.

5. Damping is expressed as a linear combination of the mass matrix and the instantaneous unreduced structure stiffness matrix.
6. The spring primary curves do not necessarily have to be symmetrical about the moment axis.
7. Arguments are passed to the subroutines using Labelled Common only.
8. The effects of shear deformations and joint dimensions may be included in the analysis.
9. The simultaneous equations are solved very efficiently by the Crout reduction method. Zero multiplications are eliminated and only that portion of the stiffness matrix that has experienced change during a step is decomposed for the next step.
10. Frames with set backs can be analysed.
11. The output includes all input data, and for a seismic analysis floor responses at user determined intervals, and member and story response envelopes. In addition should gravity loads be included, static analysis results will be output. For a modal analysis the output includes all the natural frequencies associated with drift, and the mode shape for each frequency.

The second program computes transformed section properties and the spring primary curve coordinates for beams and columns given the axial load.

The first program was tested for its capability to perform a modal and combined static-seismic analysis. The seismic analysis involved non-degrading, non-linear behaviour. The results matched very closely those computed by two other programs [33].

The steps followed in a typical static-seismic analysis are:

1. Read the input data.
2. Output the input data.
3. Process the input data by building the static joint load vector, and the structure stiffness matrix.
4. Perform a static analysis.
5. Output the results of the static analysis.
6. Read an acceleration record and compute the dynamic response.
 - If a change in linearity is detected, revert to the beginning of the step and retrace the calculations using smaller steps. Output the results if required.
7. Repeat step 6 until all records are processed.
8. Print the member and floor response envelopes and the member ductility demands.

The steps followed in a typical modal analysis are:

1. Read the input data.
2. Output the input data.
3. Process the input data by building the structure stiffness matrix thence eliminating all but the drift degrees of freedom.

4. Invert the reduced stiffness matrix and form the dynamic flexibility matrix.
5. Evaluate the mode shapes and frequencies using the matrix iteration technique.

The method used for reducing the structure stiffness matrix is well presented by Clough [61], and Humar [33] .

If a seismic analysis is undertaken, two structural stiffness matrices are used. They are identical in size. The matrix referred to in the simultaneous equation solving process, referred to here as the working matrix, is at any time either partly or completely decomposed. On the other hand, the other matrix referred to as the storage matrix is always in its original form. If a member stiffness changes because of non linearities, the stiffness contributions of that member are removed from the storage matrix and replaced by updated values. As the p-delta (geometric) coefficients are defined at the structure rather than the member level, their contributions to the storage matrix once added, remain unchanged for the duration of the study.

Thus if a number of members change stiffness during a time step, only that area of the storage matrix that is updated is read to the working matrix. This area is defined by the structural coordinate corresponding to the lowest rotational degree of freedom affected in the updating process, and the structural coordinate corresponding to the

total number of degrees of freedom. Because the working matrix is decomposed column by column from left to right, it is sufficient then to start decomposition only in the column that is the lower bound of the area defined above.

Considerable saving in computational effort is thus possible. Further savings are possible if the half bandwidth of each row is known in advance. This feature eliminates zero multiplications and is used in the program.

Consider Figs. 3.6 and 3.7. The relationship between the construction of the storage matrix and the working matrix, both in square form, is evident. So too is the area of the storage matrix that is read to the working matrix, for decomposition.

FIG. 3.6 STIFFNESS MATRICES CONTRIBUTING TO THE WORKING STIFFNESS MATRIX

2

ELEMENT ADDRESS RELATIONSHIP

FIG. 3.7 STIFFNESS MATRICES : RELATIONSHIP BETWEEN
AREA TO BE READ AND AREA TO BE REDUCED

CHAPTER 4

THE STRUCTURAL DESIGN OF THREE MULTISTORY FRAMES

4.1 INTRODUCTION

Chapters 2 and 3 described the theoretical background and methodology of this study. This chapter describes the building frames selected for study, the method for sizing the members and the procedure used for investigating the dynamic response in accordance with the assumptions stated in Chapter 3.

4.2 DESCRIPTION OF THE FRAMES

Three frames were selected for this study. They are Figs. 4.1, 4.2, and 4.3.

1. 5 stories, uniform over the entire height.
2. 10 stories, uniform over the entire height.
3. 10 stories, set back at the fifth floor level.

The basic geometry of the frames in terms of centerline to centerline dimensions are:

1. frame spacing	24 ft.
2. bay spacing	24 ft.
3. floor-floor height	12 ft.

* Gross, Transformed I
 † Gross, Transformed A

FIG. 4.1 GEOMETRIC DETAILS OF THE
 5 STORY UNIFORM FRAME

3 @ 24' E-E				
	12597*	10783*		
13949* 449.5†	18790*	16544*	36245* 718.2†	10 @ 12' E-E
13949* 449.5†	18790*	16544*	36245* 718.2†	
13949* 449.5†	18790*	16544*	36245* 718.2†	
13949* 449.5†	18790*	16544*	36245* 718.2†	
13949* 449.5†	18790*	18598*	36245* 718.2†	
14633* 473.†	20932*	18598*	37926* 750.7†	
14633* 473.†	20932*	18598*	42077* 828.†	
15652* 503.2†	20932*	18598*	42077* 828.†	
15652* 503.2†	20932*	18598*	42077* 828.†	
15652* 503.2†			42077* 828.†	

* Gross, Transformed I
† Gross, Transformed A

FIG. 4.2 GEOMETRIC DETAILS OF THE
10 STORY UNIFORM FRAME

* Gross, Transformed I
+ Gross, Transformed A

FIG. 4.3 GEOMETRIC DETAILS OF THE
10 STORY FRAME WITH SETBACK

The basic loading is:

1. Dead (D)
all floors 107 p.s.f.
roof 80 p.s.f.
2. Live (L; occupancy and snow)
all floors and roof 50 p.s.f.
3. Seismic (Q)
assigned horizontal
design ground
acceleration (A) 5%
seismic coefficient (S) $0.5 / \sqrt[3]{1.0N}$, N=5,10.
ductility coefficient (K) 0.7
importance factor (I) 1.0
foundation factor (F) 1.0
weight (W) including 25%
snow
902 k - uniform 5
1824 k - uniform 10
1208 k - setback 10
4. Wind not applicable

The material properties are:

1. Concrete
28 day compressive
strength, (F'_c) 4000 p.s.i.
cracking stress (F_r) $7.5 \sqrt{F'_c} = 475$ p.s.i.

Young's modulus (E)

$$57 \sqrt{F'_c} = 3605 \text{ k.s.i.}$$

modular ratio (n)

$$8.0$$

stress-strain relationship

(Fig. 4.4)

$$0.0 < \epsilon_c < 0.002$$

$$\sigma_c = F'_c \left[2 \frac{\epsilon_c}{.002} - \left(\frac{\epsilon_c}{.002} \right)^2 \right]$$

$$0.002 < \epsilon_c < 0.004$$

$$\sigma_c = F'_c [1 - 100 (\epsilon_c - .002)]$$

where (σ_c, ϵ_c) is a point on the $\sigma - \epsilon$ curve, for concrete.

2. Reinforcement

yield strength (E'_y)

$$60 \text{ k.s.i.}$$

Young's modulus

$$29000 \text{ k.s.i.}$$

stress-strain relationship

(Fig. 4.4)

$$0.0 < \epsilon_s < 0.00207$$

$$\sigma_s = 29000 \epsilon_s$$

$$0.00207 < \epsilon_s < \infty$$

$$\sigma_s = F'_y$$

where (σ_s, ϵ_s) is a point on the $\sigma - \epsilon$ curve, for steel.

The codes referred to are [62,63,64]:

1. the National Building Code of Canada 1977 (N.B.C.)
2. Code For The Design Of Concrete Structures for Buildings (CSA A23.3-1973)
3. Building Code Requirements for Reinforced Concrete (ACI 318-71).

CONCRETE

REINFORCEMENT

FIG. 4.4 STRESS-STRAIN RELATIONSHIPS

4.3 METHOD OF DESIGN

Several fundamental decisions were made prior to sizing the members:

1. The load factor method of design would be used.
2. All beams would be 12 inches wide.
3. Columns would be square with reinforcement evenly distributed around all faces. The dimensions of the columns would be held constant where possible whilst the % reinforcement was varied within allowable limits.
4. The two load combinations: $D + L$, and $D + L + W$, the terms of each suitably factored, would be sufficient to impose the most severe loading requirements on the beams. An extra requirement, that the combined beam capacities at a joint be less than the combined column capacities framing into the same joint was imposed when designing the columns.
5. Member capacity dependent upon good detailing practice would be assumed to be available. For example, the concrete would be assumed to be well confined by suitably spaced ties.
6. The live load acting upon individual beams and columns would be reduced as provided for by the N.B.C.
7. A checker board pattern of live load would not be required for these particular frames.

The design process is comprised of two steps:

1. Estimation of the member forces at the working level.
2. Selection of suitable dimensions and reinforcement areas at ultimate conditions.

The maximum positive and negative gravity moments in the beams were easily obtained for the two separate cases of dead and live load, by applying the formula:

$$M = k w L^2 \quad (4.1)$$

where k is continuity factor provided by CSA A23.3 for different support conditions and w is the working load assumed to be uniformly distributed over the entire length. Similarly, the axial loads in the columns were computed for the above two cases, by accumulating appropriately reduced half bay loads at each floor. Lateral loads were determined by distributing the base shear to each floor, thence further distributing the floor shears to the beams and columns, using the portal method. The floor shears of the set back frame were selected as the maximum obtained considering:

1. The tower and the base act as separate structures.
2. The entire structure is replaced by an equivalent uniform frame of weighted average width.

The beams were sized first. Each beam was examined for the worst possible loading combination; positive or negative. For the case when dead, live, and earthquake forces act simultaneously, the ultimate

capacity is given by the following equation:

$$M_{uc} = \frac{1.4 D + .7 (1.7 L + 1.8 Q)}{\phi} \quad (4.2)$$

where M_{uc} represents the ultimate moment capacity, and ϕ represents the capacity reduction factor. This expression is slightly more conservative than the expression often used:

$$M_{uc} = .75 \frac{(1.4 D + 1.7 L + 1.8 Q)}{\phi} \quad (4.3)$$

The positive and negative reinforcement areas so obtained were in general unequal, and extended beyond their design locations for the entire beam length.

Column design then followed. The primary moments were magnified to account for the fact that the frames are unbraced. Whilst the beams were sized directly once the moments had been established, the column details were obtained only after several iterations. The ACI design handbook [64] was used extensively for this purpose. Finally the beam and the column details were separately tabulated and for each frame, rationalized where possible. Tables 4.1 through 4.3 present a summary of the design details chosen.

4.4 ANALYSIS PROCEDURE

A typical analysis may be divided into four steps given the

TABLE 4.1

5 STORY UNIFORM FRAME: DESIGN DETAILS

FLOOR	EXTERIOR BAYS		INTERIOR BAYS	
	COLUMNS	BEAMS	COLUMNS	BEAMS
1	19*19 with 16#P	12*24 $\frac{1}{2}$ with 3#10T, 3#8P	24*24 with 16-#10	12*23 $\frac{1}{2}$ with 3#10T, 5#6P
2	"	"	"	"
3	"	"	"	"
4	"	"	"	"
5	"	12*21 $\frac{1}{2}$ with 4#8T, 5#6B	"	12*20 $\frac{1}{2}$ with 2#11T, 2#9P

Note: 1. Dimensions are in inches

2. Clear cover to reinforcement 1 $\frac{1}{2}$ "

3. T-top, B-bottom

4. 3#6: 3 is the number of bars, and 6 is the diameter in 1/8 inches

TABLE 4.2

10 STORY UNIFORM FRAME DESIGN DETAILS

FLOOR	EXTERIOR BAYS		INTERIOR BAYS	
	COLUMNS	BEAMS	COLUMNS	BEAMS
1	19*19 with 16#10	12*25 $\frac{1}{2}$ with 3#10T, 5#6B	24*24 with 16#14	12*24 $\frac{1}{2}$ with 3#10T, 2#9B
2	"	"	"	"
3	"	"	"	"
4	19*19 with 16#8	"	"	"
5	"	12*24 $\frac{1}{2}$ with 3#10T, 3#8B	"	"
6	19*19 with 16#8	"	24*24 with 16#11	12*23 $\frac{1}{2}$ with 3#10T, 5#6B
7	"	"	24*24 with 16#10	"
8	"	"	"	"
9	"	"	"	"
10	"	12*21 $\frac{1}{2}$ with 4#8T, 5#6B	"	12*20 $\frac{1}{2}$ with 2#11T, 2#9B

TABLE 4.3

10 STORY SETBACK FRAME: DESIGN DETAILS

FLOOR	EXTERIOR BAYS		INTERIOR BAYS	
	COLUMNS	BEAMS	COLUMNS	BEAMS
1	19*19 with 16#8	12*26 ¹ / ₂ with 5#8 ¹ / ₂ , 2#6/4#5B	24*24 with 16#11	12*25 ¹ / ₂ with 3#10T, 2#9B
2	"	12*25 ¹ / ₂ with 3#10T, 5#6B	"	12*14 ¹ / ₂ with 3#10T, 2#9B
3	"	"	"	"
4	"	"	24*24 with 16#10	"
5	"	12*22 ¹ / ₂ with 2#8/3#7T, 2#9B	"	"
6	NA	NA	"	12*22 with 2#8/3#7T, 2#8/2#7B
7	"	"	18*18 with 16#7	"
8	"	"	"	12*20 ¹ / ₂ with 3#9T, B
9	"	"	"	"
10	"	"	"	12*14 with 2#7/2#8T, 5#6B

member dimensions and reinforcement areas.

1. Perform a modal analysis and consequently obtain:
 - (a) the damping coefficients α , and β from Eqs. 2.61, through 2.63.
 - (b) an estimate of the size of the integration interval needed if a seismic analysis is intended.
2. Perform a static analysis.
3. Evaluate the spring primary curves, defined at the static load levels.
4. Perform various seismic analysis changing parameters as required.

A comprehensive modal examination was undertaken for each frame. The effects of the absence or presence of p-delta forces, cracked members, and finite joint dimensions were established through a series of computer runs. It was decided that the values of α and β to be used in this study are those associated with finite joints, cracked members and the absence of p-delta forces. The value of ρ was set as zero.

The evaluation of the spring primary curves for a member having unequal positive and negative reinforcement areas is lengthy. The discussion in Chapter 2 noted the fact that the primary curve is not trilinear. Coupling prevents a spring at one end from being defined independently from the other end. Therefore, to avoid having the results of the seismic analysis "coloured" by spurious calculations it was

decided to set the lesser of the reinforcement areas equal to the greater. Obviously only the beams were so altered. The original design details were used for the modal analyses however.

On this basis, a static analysis was undertaken for each frame that included only the dead working loads. The axial loads so determined and the column cross section details were input into the program described in Chapter 3, resulting in the cardinal points of the primary curves. As it is assumed that the beams are inextensible, it was sufficient to input their cross section details only.

Each frame was analysed assuming:

1. Stiffness degradation existed.
2. Stiffness degradation did not exist.

The integration interval for case 1 was established after several preliminary runs and was selected as 0.001 second. The resulting small interval was used directly for case 2. The acceleration scaling factors were determined by computing the base shear for each frame using an initial scaling factor of .001. This value guaranteed entirely elastic behaviour. Thus the factor A_f normally used for inelastic studies would be given by:

$$A_f = \frac{4 \Lambda}{1000 V} \quad (4.4)$$

where Λ and V represent the code base shear and computed elastic base

shear respectively. In accordance with Assumption 6 Para 50, only the dead load is used. However, as the dead and live gravity loads are not distributed to the members in this study; that is only seismic analyses rather than static-seismic analyses are undertaken, it is necessary to further amplify A_f to guarantee 'hinge' formation. The value adopted was 0.2.

CHAPTER 5

RESULTS OF DYNAMIC ANALYSIS

5.1 INTRODUCTION

The purpose of this chapter is to report on the results of the various modal and seismic analyses performed on the three frames described in Chapter 4.

The elastic analysis of building frames comprised of structural steel members can safely assume that:

1. The moments of inertia of the constituent members remains constant throughout the loading.
2. Because steel possesses great strength, the joint dimensions are small compared to the centreline to centreline distances between the members, and therefore contribute very little to the overall structural stiffness.
3. Unless a building is particularly slender or possesses unusually high levels of gravity loading, the p-delta magnification of the floor shears is negligible.

Unlike steel frames, however, the dimensions of reinforced concrete members can often be a significant portion of the overall

length, and microscopic cracks are present at random locations along the member's length even before significant load levels are reached. Further the amount of cracking increases with increased loading. It would be expected then that the response of a frame would be very much influenced by the degree to which cracking takes place, and the dimensions of the joints. Obviously cracks that occur in the columns would tend to be closed due to the presence of the axial loads and their effects may well be of little importance, but this is not the case for the beams, particularly if they carry significant levels of gravity load. Hence, it is important to get an appreciation of these effects in order to obtain an accurate understanding of the response of such buildings.

The modal analysis carried out for the three frames investigates the effects of the presence or absence of cracking, δ -delta effects and finite joint dimensions. It is intended that their contributions to the modal frequencies and displacement profiles be measured, and their relative importance defined.

The seismic analyses too, investigate the effects of cracking, and the response of the frames that assumes that stiffness degradation takes place is compared to the response that assumes that degradation does not take place. The frames are exposed to 10 seconds of the earthquake records as described in Chapter 2. Thus, not only is the instant by instant behaviour defined in terms of floor shears, sways,

and accelerations examined, but also the maximum response.

It is hoped that the studies will give an insight into the conditions necessary to require the stiffness degrading model rather than the more simple model. The former model is quite complex and much computational effort is spent imposing it. If the maximum response obtained in this manner can be obtained more quickly by using the latter model, then substantial cost savings will be realized. Obviously, this is a most important question.

5.2 MODAL ANALYSIS

Various researchers have investigated the relationship between predicted and measured modal frequencies. Otani and Sozen [13] examined several three story assemblages of two parallel rigidly interconnected frames. Their predicted results were based on the following assumptions:

1. The mass was concentrated at the joints.
2. Columns were inextensible.
3. A cracked section could be well defined by the slope of a line between the origin and a point corresponding to yielding of the tension reinforcement in the moment curvature diagram.
4. The gross section could be well represented by the transformed section of reinforcement converted to concrete.

They found that the measured initial frequencies of the frames were approximately 80% of the frequencies of the uncracked frames, suggesting that the presence of shrinkage and other cracks had quite an influence. However, it was found that the measured frequencies associated with the cracked section were 115% of the predicted value, suggesting that the concrete in tension between the cracks was providing unaccounted extra stiffness. Shepherd [40] compared the code predicted and measured fundamental frequencies of a series of actual reinforced concrete buildings. He found reasonable agreement between the results. Gulkan and Sozen [17] examined the variation of measured frequencies for a series of one story-one bay frames subjected to loading of varying intensity.

Before discussing the findings of the analyses some points require clarification.

1. A cracked section is taken here to mean a section whose moment of inertia at working loads is given by:

$$I_{cr} = kbd^3 \quad (5.1)$$

for beams, and:

$$I_{cr} = kbh^3 \quad (5.2)$$

for columns, where b , d , and h represent the width of the member, the depth from the compression face of a beam to the centroid of the tension reinforcement, and the overall depth

of a column respectively. k is given in reference [65], and is a function of the percentage of tension and compression reinforcement for a beam, and in addition a function of the ratio of the design dead load moment to the design total load moment for a column.

2. The gross cross section is defined for the separate cases of:
 - (a) the section whose value of k is $1/12$.
 - (b) the transformed section.
3. In no case are the effects of shear deformations included.
4. The p-delta amplification factor is taken as either 0 or 1. That is the effects are considered to be either non-existent or at their maximum value. No intermediate value is used.
5. Where joint dimensions are considered, they are taken from face to face. In the case where co-linear members of different depth frame into a joint, the greater depth is adopted.

Table 5.1, 5.3, and 5.5 contain the non dimensional frequencies of all the modes for the 5 story uniform frame (U5), the 10 story setback frame (S10) and the 10 story uniform frame (U10). The abbreviations in brackets () will be used hence forth. Tables 5.2, 5.4, and 5.6 contain the associated fundamental period mode shape vectors.

TABLE 5.1

MODAL FREQUENCIES FOR THE 5 STORY UNIFORM FRAME

MODE	CRACKED I				UNCRACKED I			
	JOINT DIMS=0		JOINT DIMS≠0		JOINT DIMS=0		JOINT DIMS≠0	
	P-Δ≠0	P-Δ=0	P-Δ≠0	P-Δ=0	P-Δ≠0	P-Δ=0	P-Δ≠0	P-Δ=0*
1	4.834	4.916	5.527	5.599	7.505	7.558	8.554	9.744
2	15.514	15.656	17.812	17.934	23.866	23.959	27.355	31.009
3	28.601	28.783	33.416	33.574	43.751	43.870	51.080	58.112
4	43.997	44.208	53.462	53.637	67.053	67.192	81.439	92.836
5	57.831	58.067	74.314	74.496	87.946	88.102	112.943	128.974

* Transformed section

TABLE 5.2
FUNDAMENTAL PERIOD MODE SHAPE VECTOR FOR THE 5 STORY UNIFORM FRAME

STORY	CRACKED I				UNCRAKED I			
	JOINT DIMS=0		JOINT DIMS≠0		JOINT DIMS=0		JOINT DIMS≠0	
	P-Δ≠0	P-Δ=0	P-Δ≠0	P-Δ=0	P-Δ≠0	P-Δ=0	P-Δ≠0	*P-Δ=0
1	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
2	2.614	2.609	2.427	2.436	2.599	2.597	2.417	2.423
3	4.081	4.077	3.697	3.699	4.054	4.053	3.680	3.696
4	5.144	5.150	4.613	4.625	5.118	5.121	4.602	4.629
5	5.772	5.790	5.164	5.183	5.762	5.769	5.169	5.208

*Transformed section

TABLE 5.3

MODAL FREQUENCIES FOR THE 10 STORY SETBACK FRAME

MODE	CRACKED I				UNCRACKED I			
	JOINT DIMS=0		JOINT DIMS≠0		JOINT DIMS=0		JOINT DIMS≠0	
	P-Δ≠0	P-Δ=0	P-Δ≠0	P-Δ=0	P-Δ≠0	P-Δ=0	P-Δ≠0	P-Δ=0*
1	3.049	3.122	3.537	3.600	4.603	4.740	5.387	6.105
2	5.999	6.136	6.917	7.035	9.165	9.252	10.446	11.863
3	12.206	12.386	14.146	14.300	18.833	18.950	21.580	24.385
4	16.667	16.902	19.413	19.613	25.999	26.151	30.024	34.028
5	23.150	23.394	27.208	27.416	37.200	37.448	43.338	48.831
6	29.361	29.653	34.994	35.239	46.583	46.772	55.173	62.629
7	35.702	35.961	43.406	43.623	60.195	60.370	72.451	81.567
8	42.233	42.591	52.440	52.728	69.222	69.422	85.702	97.160
9	51.170	51.499	64.399	64.664	81.652	81.880	103.342	117.120
10	61.221	61.584	80.149	80.423	91.015	91.236	119.122	136.939

* Transformed Section

TABLE 5.4

FUNDAMENTAL PERIOD MODE SHAPE VECTOR FOR THE 10 STORY SETBACK FRAME

STORY	CRACKED I				UNCRACKED I			
	JOINT DIMS=0		JOINT DIMS≠0		JOINT DIMS=0		JOINT DIMS≠0	
	P-Δ≠0	P-Δ=0	P-Δ≠0	P-Δ=0	P-Δ≠0	P-Δ=0	P-Δ≠0	P-Δ=0*
1	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
2	2.715	2.702	2.511	2.506	2.664	2.659	2.470	2.486
3	4.503	4.481	4.062	4.055	4.402	4.303	3.900	4.026
4	6.250	6.227	5.558	5.556	6.034	6.026	5.419	5.489
5	8.011	7.992	7.085	7.088	7.662	7.657	6.864	6.986
6	10.625	10.595	9.386	9.389	10.137	10.131	9.107	9.305
7	14.996	14.902	13.145	13.120	13.853	13.835	12.487	12.826
8	18.726	18.600	16.467	16.428	17.331	17.306	15.771	16.225
9	21.268	21.154	18.767	18.739	20.005	19.984	18.344	18.863
10	22.675	22.585	20.079	20.070	21.641	21.628	19.959	20.531

*Transformed Section

TABLE 5.5

MODAL FREQUENCIES FOR THE 10 STORY UNIFORM FRAME

MODE	CRACKED I			UNCRACKED I					
	JOINT DIMS=0		JOINT DIMS≠0		JOINT DIMS=0		JOINT DIMS≠0		
	P-Δ≠0	P-Δ=0	P-Δ≠0	P-Δ=0	P-Δ≠0	P-Δ=0	P-Δ≠0	P-Δ=0	P-Δ=0*
1	2.455	2.539	2.838	2.910	3.645	3.703	4.208	4.258	4.869
2	7.431	7.602	8.594	8.742	11.103	11.222	12.806	12.909	14.754
3	13.235	13.475	15.355	15.563	19.573	19.745	22.723	22.871	26.178
4	19.779	20.074	22.999	23.255	28.930	29.145	33.779	33.963	39.022
5	27.454	27.674	32.373	32.644	39.528	39.772	46.650	46.856	54.040
6	36.207	36.551	43.412	43.704	51.301	51.564	61.479	61.699	71.452
7	45.228	45.579	55.125	55.423	63.753	64.031	78.105	78.332	91.113
8	54.546	54.894	68.401	68.686	75.998	76.285	95.522	95.751	111.741
9	60.835	61.238	78.779	79.077	86.536	86.834	111.875	112.098	131.010
10	74.208	74.699	96.987	97.361	93.724	94.025	123.790	124.025	146.698

*Transformed Section

TABLE 5.6)

FUNDAMENTAL PERIOD MODE SHAPE VECTOR FOR THE 10 STORY UNIFORM FRAME

STORY	CRACKED I				UNCRACKED I			
	JOINT DIMS=0		JOINT DIMS≠0		JOINT DIMS=0		JOINT DIMS≠0	
	P-Δ≠0	P-Δ=0	P-Δ≠0	P-Δ=0	P-Δ≠0	P-Δ=0	P-Δ≠0	P-Δ=0
1	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
2	2.794	2.772	2.587	2.577	2.667	2.657	2.472	2.468
3	4.716	4.668	4.251	4.231	4.403	4.382	3.982	3.974
4	6.474	6.413	5.798	5.774	6.064	6.039	5.427	5.419
5	8.305	8.239	7.374	7.353	7.618	7.595	6.787	6.784
6	10.036	9.979	8.860	8.851	9.029	9.014	8.033	8.037
7	11.493	11.460	10.118	10.129	10.225	10.223	9.096	9.110
8	12.607	12.605	11.086	11.121	11.155	11.167	9.929	9.954
9	13.360	13.386	11.746	11.802	11.800	11.824	10.516	10.549
10	13.801	13.847	12.142	12.212	12.200	12.233	10.892	10.932
								11.513

*Transformed Section

Consider the ρ -delta effects first. It can be stated for the three frames, inclusion of their effects has negligible influence on the modal frequencies, and mode shapes. In absolute terms their presence slightly decreases the frequencies and slightly exaggerates the mode shape.

For any given frame considering the case of the absence of ρ -delta effects as the base case, the % reduction in frequency decreases as the structure is considered progressively stiffer. The fundamental frequencies experience the most reduction, the reduction diminishing with increasing mode.

Comparing the 3 frames as a group, it is seen that the % reduction increases in the order U5-S10-U10. With U10 experiencing a reduction of approximately 3.3%, noting that under similar conditions the % reduction for U5 is 1.67%, it is suspected that a linear relationship exists and therefore a 20 story frame could be expected to experience a maximum reduction of $2 \times 3.3\%$ or 6.6%, approximately.

It can also be seen that the ratio of the % reduction for the softest version of any given frame to the stiffest version holds constant at approximately 3. Because of these results it was decided to disregard the effects during the seismic analysis.

The exclusion of joint dimensions has a more pronounced effect. The fundamental frequencies are reduced by 12-13%, in the order U5-U10-S10. The reduction increases with the increasing mode to about 22-24% for the highest mode, again in the order U5-U10-S10. Accordingly joint dimensions have been included in the seismic analysis.

The most significant factor however, is the reduction in stiffness due to cracking. Irrespective of frame type, the ratio of the fundamental frequency of the stiffest possible form of a given frame, to the fundamental frequency of the softest form is 2. That is, their stiffnesses are in the ratio of 4:1. It is interesting that this relationship holds essentially true for all other modes.

The % reduction in any frequency from the stiffest possible form of any frame to the stiffest possible form of the cracked frame is 40% approximately. Thus the lower and upper bound of the % stiffness reduction is 40% and 50% respectively. It is not surprising therefore, that significant changes in dynamic response take place when members are severely distressed. What remains to be seen however, is the effects of such changes on the maximum response values of members subjected to seismic loading.

The fundamental period mode shapes are shown in Figs. 5.1, 5.2, and 5.3. As expected there is a dramatic amplification of response above the 5th floor level of S10. The lack of coincidence of profiles for a given frame can be attributed to non uniform stiffness changes from one case to another. These changes arise from four sources:

1. The p-delta contributions which increase with decreasing floor level.
2. The joint dimensions which reduce with increasing floor level.
3. Lack of homogeneity in the treatment of cracked beams and columns.
4. Computational errors involved in the Eigenvector solution process.

It is seen that the p-delta effects are negligible. It would be expected that the computational errors are also very small because of the general use of double precision arithmetic. The frames U10, and S10 indicate that the joint dimensions exert little influence, hence it must be concluded that the lack of homogeneity is the dominant variable.

5.3 SEISMIC ANALYSIS

Before commenting on the results of the various computer runs, it must be made clear that use of the word "maximum", in fact means

FIG. 5.1 FUNDAMENTAL MODE SHAPE OF
THE 5 STORY UNIFORM FRAME

FIG. 5.2 FUNDAMENTAL MODE SHAPE OF THE 10 STORY UNIFORM FRAME

FIG. 5.3 FUNDAMENTAL MODE SHAPE FOR THE
10 STORY SETBACK FRAME

FIG. 5.4 INELASTIC RESPONSE - MAXIMUM SWAY (IN)

FIG. 5.5 INELASTIC RESPONSE - MAXIMUM RELATIVE SWAY (IN)

FIG. 5.6 INELASTIC RESPONSE - MAXIMUM SHEARS (KIPS)

FIG. 5.7 INELASTIC RESPONSE - MAXIMUM FLOOR ACCELERATIONS. (IN /SEC/SEC)

FIG. 5.8 INELASTIC RESPONSE - BEAM DUCTILITY RATIOS

FIG. 5.9 INELASTIC RESPONSE - COLUMN DUCTILITY RATIOS

"absolute maximum" with the exception of the floor accelerations. Only the absolute greater of the positive and negative response values has been plotted at each floor level.

With reference to Fig. 5.6, it is seen that the degrading model produces a large decrease in the maximum first floor shear for all frames, and with the exception of S10, this difference persists at all other floor levels. Irrespective of the model used, the first floor shears for U5 and U10 are higher than that for S10, and the shear envelopes for the degrading model are quite smooth. S10 displays a "converging" effect above the second floor level.

The maximum acceleration profiles (Fig. 5.7) for both models of a given frame are quite dissimilar too and the maximum degrading value is always at a higher floor. There is a tendency for the degrading model to produce constant values of acceleration and there is a considerable reduction for S10 above floor 6 associated with the non-degrading model, but as mentioned above, the absolute values have not been computed.

The most interesting results however, are evident from Figs. 5.4, 5.5, 5.8, and 5.9. The degrading model indicates that for all frames, there is a marked decrease in ductility demand as compared to the non degrading model. The work of Chopra and Kan [22] shows the same tendency for a frame of comparable period, but their mathematical models

were much less detailed as noted in Chapter 1, and so only a limited comparison is possible.

It would be expected that the increase in period expected as a result of severe cracking would mollify the response. However, the energy absorption capability of the degrading model should be less than the non degrading model, particularly for small loops, and this would counteract somewhat the former effect. It appears therefore, for these frames, that this former effect is dominant.

The profiles for each frame (Fig. 5.4) have a similar shape with the exception of S10. The degrading model has smaller abscissae than non degrading model. Again S10 exhibits quite an accentuation above its mid height, and of the three frames, experiences the greatest sway and relative sway.

The maximum non degrading ductility demand of the beams (Fig. 5.8) are differently distributed for the three frames, and with the exception of S10 are approximately constant throughout most of the height. The range of height over which yielding takes place with the degrading model is dramatically less than the other model.

The column ductility demands indicate the same general relationship between the degrading and the non degrading model as do the beams, but for a given frame the ratio of the maximum non degrading ductility to

the maximum degrading ductility is greater than the comparable ratio of the beams. In accordance with the strong column beam design philosophy of the N.B.C., it is seen that the range of column yielding is considerably less than that of the beams for either model.

Thus far the discussion has only involved the maximum response. During the course of the computer runs, intermediate floor responses were output every 0.5 seconds of seismic loading. The results (not shown) indicate considerable difference between the models at any instant, not only in magnitude but also in direction.

Over all, an important conclusion may be drawn for the models. The effects of stiffness degradation appears to be very favourable in that its modeling indicates a significant mollification of the response of the frames to a severe earthquake. In particular, the setback frame appears to be the most favoured in that motion that otherwise would be accentuated at and above the set back level appears as smooth, of gradual variation, and reduced in magnitude.

CHAPTER 6

SUMMARY AND CONCLUSIONS

6.1 SUMMARY

This study was concerned with the dynamic analysis of multistory rigid jointed reinforced concrete frames. Particular attention was paid to the effects of cracking on their modal and seismic response. To meet this task, the study was organized into the following stages:

1. A thorough review of the literature to establish the present state of knowledge on the related theory and its application.
2. A detailed development of the parameters of the equations of motion.
3. Application of the theory to practical and efficient methods of analysis, as exemplified by the capabilities of the several computer programs developed.
4. Selection and design of three multistory frames.
5. A parametric investigation of these frames.
6. An examination of the various results for any general trends or relationships.

It has been shown that reinforced concrete buildings cannot be treated as steel framed buildings. The effects of cracking are too

pervasive. Modal frequencies are dramatically altered, the internal distribution of gravity loads is very difficult to determine, and the intermediate and maximum seismic response values bear scant resemblance to those expected if non degrading behaviour is assumed.

In so far that considerable effort was expended in developing a refined mathematical model, the spirit of the studies was that of establishing practical guidelines for engineering analysis where ever possible.

The important conclusions drawn from this study are summarized in this chapter. They are relevant only if the following requirements are satisfied and bearing in mind the very limited evidence :

1. All members of the frame are reinforced concrete.
2. The members are rigidly connected at the joints.
3. Special stiffening elements such as shear walls, bracing etc. are absent.
4. All out of plane movements are suppressed. Notwithstanding these limitations, the conclusions hopefully will be of use.

6.2 MODAL ANALYSIS CONCLUSIONS

The following conclusions may be drawn from the modal analysis.

1. The p-delta forces have negligible influence on both the mode shapes and modal frequencies for small to medium

(20 story) sized frames, and maybe thus ignored in any practical analysis.

2. The exclusion of joint dimensions from an analysis results in the under estimation of the fundamental period, given a degree of cracking, of some 13-14%. Even though joints are not entirely rigid and distortions do take place, its influence cannot be ignored. Unlike the p-delta effects the influence increases rather than decreases with increase in mode. In addition, the mode shape is amplified due to irregular joint size distribution throughout the structure.
3. The presence of cracking has a profound influence on the modal frequencies. As suggested by Otani, microscopic cracks present in what might be considered an uncracked section, may reduce the fundamental period by some 15%. On the other hand, if cracking is taken into consideration at working levels, the reduction would be in the order of 40 to 50%. However, as the concrete is not cracked at all locations, the reduction will lie somewhere between these limits.

6.3 SEISMIC ANALYSIS CONCLUSIONS

Given the type of hysteresis models used in this study, it is apparent that the presence of cracking exemplified by the degrading

model, has a profound influence on the seismic response. The effects can be summarized as follows:

1. Both the beam and column ductility demand ratios are reduced if stiffness degradation is assumed (S10 particularly).
2. The sways for each the frames using the degrading model are reduced to about 68% to 70% of their value when stiffness degradation is ignored.
3. Floor accelerations and shears are appreciably reduced when degradation is modelled.

The cost of the computer runs using the degrading model were twice that of the non degrading model. The 10 story uniform frame was the most costly to analyse, being \$200 using a Xerox Sigma-9 digital computer, as against \$80 for the same frame using the non degrading model. However, it would appear that this cost is warranted if such a response difference is characteristic.

Until further clarification of the phenomenon is obtained it is felt that the essential cause of the difference lies in the ability of the degrading model to undergo small absorption loops whereas, provided the force level in a member is less than the positive and negative yield values, the non degrading member will simply load and unload without any absorption whatsoever.

6.4 RECOMMENDATIONS FOR FUTURE STUDIES

During the course of this research, certain topics of significance have been identified for which there is scope for additional work. The topics may be broadly classified as:

1. Areas where the modelling capability of the computer programs may be improved.
2. Selection and examination of a larger range of frames for a given class of structure.

The first classification includes the following suggestions.

1. There is a need to be able to realistically predict the internal forces and deformation arising from static application of gravity loads to reinforced concrete members. The direct consequence of this ability is the possibility of testing the adequacy of the code provisions respecting ductility demand and shear force distribution. It is possible that such studies may suggest that loading on reinforced concrete buildings be provided for separately in the codes rather than collectively with steel framed buildings as is the present practice.
2. It is felt necessary to model material behaviour expressed by hysteretic rules more realistically. A member of a frame is not in isolation from other members, therefore, the

interaction effects must be reflected by the rules employed. The effects of axial loads, joint deformations, bond slip of the tension reinforcement and shear pinching should be thoroughly examined and evaluated. Although Otari [7] has included the effects of bond slip in some of his work, and Takayanari and Schnobrich [25] accounted for shear pinching and changing axial loads in their study, it is none the less felt this is only a beginning in what appears to be a fruitful but complex area.

Finally and with respect to the second classification, it is suggested that a variety of uniform frames ranging from very stiff to very flexible be examined for their response to seismic loading according to different hysteretic models. Also, frames with varying degrees of setback, multiplicity of setback, and tower to base ratios should be examined before firm conclusions can be drawn as to their response characteristics and compatibility (or lack thereof) with the code requirements.

NOMENCLATURE

Symbols are defined where they first appear in the text. In addition, they are defined below in alphabetical order. Whenever possible the same definition of a symbol is retained throughout: when a symbol has been used in more than one sense, all the alternative definitions are given below.

A	- Assigned horizontal design ground acceleration
A_e	- Effective shear area
A_f	- Ground acceleration scaling factor
{A}	- Parameter of the general form of the joint acceleration formula
a	- Location of a concentrated beam load wrt the left end
B	- Parameter of the general form of the joint acceleration formula
b	- Location of a concentrated beam load wrt the right end
[C]	- Structure damping matrix
D	- Dead load
D_{max}	- Ratio of absolute maximum displacement to yield displacement
E	- Young's modulus
F	- Seismic foundation factor
F_i	- p-delta shear at floor i

- f_r - Concrete tensile strength
- f_y - Reinforcement yield strength
- f_c - Concrete strength at 28 days
- f_{E1}^L, f_{E2}^L - Elastic flexibility coefficients associated with a member's left (lower) end derived from application of an applied left (lower) end moment (1), right (upper) end moment (2)
- f_{E1}^R, f_{E2}^R - Elastic flexibility coefficients associated with a member's right (upper) end, derived as above.
- f_{I1}^L, f_{I2}^L - Inelastic flexibility coefficients associated with a member's left (lower) end derived as above.
- f_{I1}^R, f_{I2}^R - Inelastic flexibility coefficients associated with a member's right (upper) end derived as above.
- $\{F\}$ - Structure exciting force vector
- $\{\Delta F\}$ - Structure incremental exciting force vector
- $\{\Delta F^*\}$ - Effective structure incremental joint load vector
- G - Shear modulus
- h_i - Distance from floor i-1 to floor i
- I - Moment of inertia, seismic importance factor
- K - Seismic ductility coefficient
- $[K]$ - Structure static stiffness matrix
- $[K^*]$ - Effective (dynamic) structure stiffness matrix
- $[k]^6$ - Member stiffness matrix - 6 degrees of freedom
- $[k]^4$ - Member stiffness matrix - 4 degrees of freedom
- k - Cracking factor of moment of inertia

- L - Length of a member from centerline to centerline of joints, live load
- l - Flexible length of a member
- M - Moment
- M_L, M_R - Moment applied at the left (lower) end of the flexible length of a member or the right (upper) end, joint moment
- M_L^j, M_R^j - Member Moment applied at the centerline of the left (lower) joint or the right (upper) joint
- M_c, M_y, M_u - Cracking, yielding and ultimate moment of a member under antisymmetric flexure or of a section when these thresholds are reached
- M_{uc} - Section design moment at ultimate strength
- M_L^F, M_R^F - Fixed end moments at the left or right end of a member
- M_C^L, M_C^R - Cracking moment capacities in both loading direction when tension and compression reinforcement areas are unequal
- M_Y^L, M_Y^R - Yielding moment capacities when tension and compression reinforcement areas are unequal
- M_U^L, M_U^R - Ultimate moment capacities, when tension and compression reinforcement areas are unequal
- $[M]$ - Mass matrix
- m - Member moment due to a unit moment
- N - parameter specifying the limits of numerical stability
- n - Modular ratio
- P - Concentrated load
- P_1 - Accumulated gravity load to and including floor 1

- P_L, P_R - Axial load of lower and upper end of a member.
- p - Degree of the numerical integration formula, axial load
- $\sim$ - Seismic load
- S - Parameter of the general form of the joint acceleration formula (+1 or -1).
- t - Time elapsed from start of seismic loading
- Δt - Time step of numerical integration
- $\{F\}$ - Unbalanced joint load vector arising during an integration step
- V - Base shear derived from the building code, member shear
- v - Member shear due to a unit moment
- V_L, V_R - Shear applied at the centerline of the left (lower) or right (upper) joint
- W - Dead load of a structure
- w - Uniformly distributed load
- $\{\Delta \ddot{y}\}$ - Absolute incremental base acceleration vector
- $\{z\}, \{\dot{z}\}, \{\ddot{z}\}$ - Joint displacement, velocity and acceleration vectors wrt the base
- $\{z_0\}, \{\dot{z}_0\}, \{\ddot{z}_0\}$ - Joint displacement, velocity and acceleration vectors at the start of an integration step
- $\{z_1\}, \{\dot{z}_1\}, \{\ddot{z}_1\}$ - Joint displacement velocity and acceleration vectors at the end of an integration step
- $\{\Delta z\}, \{\Delta \dot{z}\}, \{\Delta \ddot{z}\}$ - Incremental joint displacement, velocity and acceleration vectors
- α - Mass damping constant, spring parameter, inelastic spring rotation
- R - Stiffness damping constant
- Δ - Interstory drift
- Δ_I - Maximum inelastic drift of a structure

- Δ - Drift of a structure at the onset of yield
- Δ_L, Δ_U - Joint translations at the left (lower) and right (upper) end of a member
- δ - Member lateral deformation
- θ_L, θ_U - Joint rotations at the left (lower) and right (upper) end of a member
- θ - Member rotation
- θ_L', θ_U' - Total member rotation at left (lower) and right (upper) centerline of joint
- θ_{max} - Maximum spring rotation
- $\theta_C, \theta_Y, \theta_U$ - Cracking, yielding, and ultimate rotation of a member under antisymmetric flexure
- θ_L, θ_R - Total elastic, or elastic-inelastic rotation of the left (lower) and right (upper) end of a flexible member with simultaneously applied end moments
- θ_L^L, θ_L^R - Elastic rotation at the left (lower) end of a member when subjected to an applied moment at the left (lower) end and right (upper) end
- θ_R^L, θ_R^R - Elastic rotation at the right (upper) end of a member when subjected to moments as above
- θ_L^1, θ_L^2 - Total rotations at the left (lower) end of a member when subjected to moments as above
- θ_R^1, θ_R^2 - Total rotations at the right (upper) end of a member when subjected to moments as above

- ϵ - Axial deformation, strain
- ϵ_c - Concrete strain
- ϵ_s - Reinforcement strain
- ϵ_p - Axial member deformation at the lower and upper end
- λ, μ, ν - Parameters required for deriving the spring primary curve
- η - Ductility demand factor
- σ - stress
- σ_c - Concrete stress
- σ_s - Reinforcement stress
- ϕ - Curvature, section capacity reduction factor
- ϕ_c, ϕ_y, ϕ_u - Cracking, yielding and ultimate curvature of a section in a given direction
- $\phi_{VL}, \phi_{VR}, \phi_{UL}, \phi_{UR}$ - Yield curvature of a section in both loading directions
- ϕ_{UL}, ϕ_{UR} - Ultimate curvature of a section in both loading directions
- ν_L, ν_R - Ratio of rigid left (lower) and right (upper) end length to the flexible length of a member
- ξ_1, ξ_2 - Modal damping factors
- ω_1, ω_2 - Circular modal frequencies
- λ - Computed elastic base shear
- ψ - Shear flexibility factor

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APPENDIX A

TWO COMPUTER PROGRAMS FOR THE
MODAL STATIC AND SEISMIC ANALYSIS
OF DEGRADING AND NON DEGRADING
MULTISTORY RECTANGULAR FRAMES

INSTRUCTIONS FOR USE

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PREFACE

The two programs described in this manual were developed by the author in the course of his research work in the area of seismic analysis. They are in fact two of a growing series of programs available in the Carleton University's scientific program library for the dynamic analysis of multistory frames.

The first program described here (code name; QUAKES*), analyses rigid jointed planar frames. The type of analysis permitted is either a modal analysis, a static analysis, a seismic analysis or a combined static-seismic analysis. In all cases, finite joint dimensions, shear deformations and p-delta effects may be included. For the seismic analysis the member stiffness may be considered as degrading (typical for reinforced concrete frames) or non degrading. Either the linear or the constant acceleration method of numerical integration may be used. The damping matrix may be defined as a linear combination of the mass and instantaneous unreduced stiffness matrices. The simultaneous equations are solved efficiently by a modified form of the Crout reduction technique. Zero multiplications are avoided and only that portion of the structure stiffness matrix that has changed during a time step is decomposed. Step calculations can be retraced over smaller substeps and correction forces are computed when overshoots occur.

*Quick Analyses of Kinetically Energized Structures

The second program (code name; SPINES*), computes the moment-curvature primary curve of a cross section, required to define the stiffness of springs conceptually placed at a flexible elastic member's end, given the member's cross section dimensions, reinforcement areas and axial load.

Although the programs have been successfully used for a substantial number of analyses of building frames, the possibility always remains with programs of this nature that they will fail to function properly under some special set of conditions. It would be therefore appreciated if any real or apparent deficiencies in the programs or the user's manual are brought to the notice of the under signed.

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*Spring Primary Curve Indexes

DISCLAIMER

While every effort has been made to ensure the accuracy of the information presented in this manual, and the reliability of the programs described herein, use of this material can be made only with the understanding that neither the author nor Carleton University will assume any liability of any kind arising from such use.

DISCLAIMER

The National Library of Canada also
does not assume any liability of
anykind arising from the use of
information presented in this manual.

CONTROL CARDS

The following control cards are used to execute the programs on the XDS sigma 9 computer as of May, 1978. With the passage of time, some changes may occur in the control cards. If the control cards do not cause successful execution of the programs, it is suggested that the S.P.L. librarian at the Computing Center be consulted.

EXECUTION OF SPL QUAKES USING THE RBT IN THE COMPUTER ROOM OF C.J. MACKENZIE BUILDING AND CARDS FOR INPUT DATA

```
!JOB XXXXXXXX, XXXXXXXXXXXX, 7
!LIMIT (TIME,2), (LO,200),(CORE,40)
!ASSIGN F:4, (FILE+,XXXXX),(IN),(SAVE)
!ASSIGN F:5, (DEVICE,SI)
!ASSIGN F:6, (DEVICE,LO)
!RUN (LMN,QUAKES,SPL)
!DATA
data cards
!EOD
```

⁺Note: The file name associated with F:4, is the file containing the acceleration records. The user may use a variety of files and hence file names.

EXECUTION OF SPL SPINES

:JOB XXXXXXXX, XXXXXXXXXXXX, 7

:LIMIT (TIME,2),(LO,200)

:RUN (LMN,SPINES,SPL)

:DATA

↑
data cards
↓

:EOD

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1. INTRODUCTION

It is now well recognized that any economical multistory frame structure, designed to withstand a strong earthquake, without undergoing serious damage, must rely upon the ductility of the framing members to absorb the energy input during such an earthquake. An appreciation of the theory of the response of such an inelastic structure is essential to any effort, not only to understand the behaviour of actual structures, but also towards formulating rational guidelines for design.

The analytical study of the inelastic response of a structure is highly complex. Simplifying assumptions have, therefore, to be made to render the problem capable on analytical study even with the aid of high speed digital computers. In spite of this limitation, the results of such an analytical study are very useful.

These computer programs have been developed to permit such analyses, of steel framed or reinforced concrete framed structures.

The major program, QUAKES computes:

1. Modal frequencies and the mode shape for each frequency.
2. Static end moments and rotations for beams and columns, and axial loads for columns.
3. Given a specified ground motion; seismic response histories of floor forces, shears, sways, relative sways, velocities and accelerations. In addition the member and floor response

envelopes are computed. For the beams this includes maximum and minimum end moments, rotations and ductilities. For the column this includes maximum and minimum end moments and rotations, axial loads and ductility factors, and for the floors, this includes maximum and minimum shears, sways, relative sways, and maximum velocities and accelerations. The maximum and minimum overturning moments on the structure are computed too.

The other program, SPINES computes the cracking, yielding, and ultimate moments and the yielding and ultimate curvatures of a reinforced concrete member, and the moment of inertia and area of its transformed section.

Input data for the first program includes beam and column properties, gravity load data, the mass matrix, the acceleration of ground motion defined at evenly spaced intervals of time, and miscellaneous data including material properties, scaling factors and assorted user options. Input data for the second program includes the member cross-section dimensions, the area and location of the reinforcement, and the axial load present (zero for beams).

The procedure QUAKES takes for a modal analysis is to:

1. Read all input data.
2. Output all input data.
3. Assemble the structure stiffness matrix from the beam and column properties, including joint dimensions,

shear deformations and, p-delta effects if required.

4. Reduce the stiffness matrix, eliminating all degrees of freedom other than sway.
5. Invert the reduced stiffness matrix and add the dynamic terms.
6. Compute and output the lowest frequency and associated mode shape. Repeat for all other modes using the matrix iteration method.

The procedure the program takes for a combined static-seismic analysis is to:

1. Read all input data.
2. Output all input data.
3. Process the input data by:
 - (a) computing the spring primary curves,
 - (b) evaluating the joint load vectors due to gravity loads,
 - (c) establishing a look up vector required for the floor force evaluation,
 - (d) deriving the structure overturning arms,
 - (e) evaluating the numerical integration coefficients,
 - (f) assembling the structure stiffness matrix from the member properties, including the joint dimensions, etc as noted for the modal analysis.
4. Perform a static analysis and output the results.

5. Initialize the dynamic response arrays and compute the incremental displacements.
6. Compute the member incremental moments and rotations and thus the current total values.
7. Study the member force levels for change in linearity.
8. If a change is detected, retrace the calculations for the last time step using smaller substeps if desired. Compute the unbalanced joint loads.
9. Compute the current total floor response.
10. Update all maximum and minimum values and return to 6. if not finished, otherwise;
11. Output the response envelopes, and ductility ratios.

A static analysis only, excludes most of step 3 and all of steps 5 through

11. A seismic analysis only, excludes step 4.

If a seismic analysis is undertaken, the status of the springs is examined at the end of each interval and the member stiffness matrices are updated if appropriate. Normally the structural configuration will change during an interval and not at the end of it. The structure stiffness matrix will not, therefore, be constant during the observed interval. The error due to this will not be important if the time interval selected is small.

For cases when the error may be significant, the program provides an option by which whenever a change in structural configuration is detected at the end of an interval of time, the step by step integration reverts back to the status at the

beginning of the interval and then retraces the period of one normal period in smaller substeps. Thus if the data implies 5 subdivisions, the calculations will be retraced over 5 equal substeps. During each substep the base acceleration will be taken as equal to $1/5$ the change during a normal interval.

Retracing the integration over substeps of time will obviously increase the execution time. The extra time will depend on the size of the substep adopted.

To make this manual self contained, a brief description of the method of analysis is presented below.

2. ANALYSIS

2.1 ASSUMPTIONS

The following assumptions are made in the analysis.

ASSUMPTIONS ABOUT A STRUCTURE

1. A structure consists of a set of parallel, rectangular, rigid jointed frames.
2. The frames are linked by floors that are rigid in the horizontal direction, and the height of the floors are the same in each constituent frame.
3. The centroidal axis of the columns must be straight and continuous from the base to the roof whether setback or not.
4. The centroidal axis of the beams must be straight and continuous.
5. The frames are connected (hinged or fixed) to an infinitely rigid foundation.
6. The mass of each story is concentrated at the joints. Vertical torsional and rotational inertial forces are ignored.
7. Mass either dead or live is considered invariant in time.
8. Damping is considered viscous, thus the damping coefficients are set as a linear combination of the mass and stiffness matrix.

9. The base excitation is a horizontal motion in the plane of a frame.
10. The structures resist all loading by essentially flexural behaviour.

ASSUMPTIONS ABOUT MEMBERS AND JOINTS

1. Compatibility of deformations and equilibrium of forces at a joint are satisfied.
2. The effects of axial loads on member stiffness properties are negligible. P-delta effects may be considered however.
3. Local and out of plane buckling is prevented.
4. Axial deformations of the beams are negligible.
5. Beam column joints are entirely rigid.
6. Inelastic flexural deformation of a member is concentrated and approximated by the deformation of two non linear springs placed at the end of the flexible part of the member [1,2]⁺
7. The moment-rotation relationship of the springs are governed (for degrading behaviour), by the rules developed by Takeda et al [3].
8. The above relationship is not practically effected by the effects of shear pinching, strength decay, time dependent axial loads, gravity loads, joint deformations, joint crushing, bond destruction, and other miscellaneous causes of degradation.

⁺The numbers in brackets refer to the reference given at the end of this manual.

9. The inflexion point for beams and columns, lies at the member center throughout the excitation.

ASSUMPTIONS ABOUT A REINFORCED CONCRETE CROSS SECTION

1. Plane sections before bending remain plane after bending.
2. The strains in the reinforcement and the concrete at the same level are identical.
3. The maximum useful compressive concrete strain is 0.004.

2.2. ACCELERATION METHODS [4,5]

Consider the sketch of the multistory frame; Fig. 1. The equation of motion for the structure using matrix algebra notation may be written as:

$$[M] \{\ddot{z}\} + [C] \{\dot{z}\} + [K] \{z\} = \{F\} \quad (1)$$

where $[M]$ is the mass matrix, $[C]$ is the damping matrix, $[K]$ is the structure stiffness matrix, and $\{F\}$ is the exciting force vector.

Assuming then that the parameters $[C]$, and $[K]$ change during the course of excitation, it is convenient to rewrite Eq. 1 in incremental form as follows:

$$[M] \{\Delta\ddot{z}\} + [C] \{\Delta\dot{z}\} + [K] \{\Delta z\} = \{\Delta F\} \quad (2)$$

FIG. 1. MULTISTORY-MULTIBAY FRAME SUBJECTED TO BASE MOTION

If expressions can be found for $\{\Delta\ddot{z}\}$ and $\{\Delta\dot{z}\}$ in terms of $\{\Delta z\}$ then Eq. 2 may be solved. In practice the rate of change of joint acceleration is unknown and some relationship has to be assumed. If the accelerations are assumed to vary linearly over a small interval of time Δt , it can be shown that:

$$\{\Delta\ddot{z}\} = -3 \{\ddot{z}_0\} - \{\dot{z}_0\} \frac{6}{\Delta t} + \{\Delta z\} \frac{6}{\Delta t^2} \quad (3)$$

and

$$\{\Delta\dot{z}\} = -\{\ddot{z}_0\} \frac{\Delta t}{2} - 3 \{\dot{z}_0\} + \{\Delta z\} \frac{3}{\Delta t} \quad (4)$$

where z_0 is the value of time at the start of the interval. According to the assumption that the damping coefficients can be represented as a linear combination of mass and stiffness then:

$$[C] = \alpha [M] + \beta [K] \quad (5)$$

where α and β are constants.

Substituting Eqs. 3, 4, and 5 into Eq. 2 and rearranging, the following incremental forces displacement relationship results:

$$[K^*] \{\Delta z\} = \{\Delta F^*\} \quad (6)$$

where $[K^*]$ and $\{\Delta F^*\}$ frequently referred to as the effective stiffness

matrix and exciting force vector represent:

$$[M] \left(\frac{6}{\Delta t^2} + \frac{3\alpha}{\Delta t} \right) + [K] \left(\frac{3\beta}{\Delta t} + 1 \right)$$

and

$$[M] [-\{\Delta \ddot{y}\} + \{\ddot{z}_0\} (3 + \alpha \frac{\Delta t}{2}) + \{\dot{z}_0\} (\frac{6}{\Delta t} + 3\alpha)] +$$

$$\beta [K] [3 \{\dot{z}_0\} + \frac{\Delta t}{2} \{\ddot{z}_0\}] \quad (7)$$

respectively.

If however it can be assumed that the joint accelerations are constant over the interval under observation, then a similar procedure to above yields for $[K^*]$, and $\{\Delta F^*\}$:

$$[M] \left(\frac{4}{\Delta t^2} + \frac{2\alpha}{\Delta t} \right) + [K] \left(\frac{2\beta}{\Delta t} + 1 \right)$$

and

$$[M] [-\{\Delta \ddot{y}\} + 2 \{\ddot{z}_0\} + \{\dot{z}_0\} (\frac{4}{\Delta t} + 2\alpha)] + 2\beta [K] \{\dot{z}_0\} \quad (8)$$

respectively.

If Eqs. 7 or 8 are defined at the beginning of every step, then Eq. 6 can be solved for the incremental joint displacements. The total response at the end of an interval then is simply the sum of the current total values at the beginning of the step and the incremental values just obtained, that is:

$$\{z_1\} = \{z_0\} + \{\Delta z\}$$

$$\{\dot{z}_1\} = \{\dot{z}_0\} + \{\Delta \dot{z}\} \quad (9)$$

$$\{\ddot{z}_1\} = \{\ddot{z}_0\} + \{\Delta \ddot{z}\}$$

represent the total joint displacements, velocities, and accelerations at the end of the interval.

2.3 EVALUATION OF THE EFFECTIVE STIFFNESS MATRIX

The process of deriving $[K^*]$ may be conveniently divided into 7 steps. They are:

1. Evaluate the moment-axial load-curvature relationship for each section.
2. Hence derive the rotation at the member ends, given a moment pattern.
3. Define the element stiffnesses taking into account the elastic and inelastic effects.
4. Transform the matrices to the member's rigid ends to account for finite joint dimensions.
5. Transform the resulting member matrices into the structural system.
6. Add the p-delta effects if required.
7. Add the dynamic terms.

Assume for the while steps 1. and 2. are complete, then assuming that the curvature diagram (Fig. 2) can be decomposed into 2 elementary curvature diagrams (Fig. 2) then the following matrix expression relating end moments M and end rotations θ results:

$$\begin{Bmatrix} \theta_L \\ \theta_R \end{Bmatrix} = \begin{bmatrix} F_L^1 & F_L^2 \\ F_R^1 & F_R^2 \end{bmatrix}_E + \begin{bmatrix} F_L^1 & F_L^2 \\ F_R^1 & F_R^2 \end{bmatrix}_I \begin{Bmatrix} M_L \\ M_R \end{Bmatrix} \quad (10)$$

where the superscripts refer to the loading case and the subscripts refer to the left or the right end of the member. The left submatrix contains the elastic flexibility coefficients and the right submatrix contains the inelastic coefficients.

For a prismatic element, the elastic coefficients that reflect shear deformations as well as flexural deformations can be shown to be:

$$\begin{bmatrix} 1 + \psi & \psi \\ \psi & 1 + \psi \end{bmatrix} \frac{l}{EI} \begin{bmatrix} 1 & -1/2 \\ -1/2 & 1 \end{bmatrix} \quad (11)$$

where ψ is the shear flexibility coefficient and is given by

$\frac{6EI}{l^2 A_e G}$. E , I , l , A_e , and G represent Young's modulus, the moment of inertia, member length, effective shear area, and shear modulus respectively. For the case when one end is pinned say the left end, Eq. 11 reduces to:

DEFORMED MEMBER

ARBITRARY MOMENT DIAGRAM

CORRESPONDING CURVATURE DIAGRAM

ELEMENTARY CURVATURE DIAGRAMS

FIG. 2 INELASTIC MOMENT-CURVATURE-ROTATION RELATIONSHIP

$$\begin{bmatrix} 0 & 0 \\ 0 & 2 + \psi \end{bmatrix} \frac{l}{3EI} \begin{bmatrix} 0 & 0 \\ 0 & 1/2 \end{bmatrix} \quad (12)$$

At present there is no theoretically exact method for determining the inelastic coefficients. Unfortunately they depend entirely on the history of the loading and are thus indeterminate [6]. However if the inelastic rotations are assumed to be concentrated at the ends, the off diagonal elements become zero, and the main diagonal elements may be found by studying the experimental behaviour of members subject to a fixed loading pattern, say antisymmetric loading. The rotations so obtained may be used to "proxy" for the actual unknown situation.

If the actual moments in a frame member are approximately equal in value and opposite in sign, then the approximations will be quite reasonable. Returning then to Steps 1 and 2, the moment curvature relationship at the primary curve can be readily determined for each member given the section and material properties [7]. The hysteresis behaviour for complex inelastic loading and unloading cycles is assumed to be well represented by Takeda's rules. These rules will be discussed in more detail later.

So far attention has been given only to the flexible portion of a member. If the beam-column joints are of moderate size, their effect is to stiffen the structure, that is consideration of centerline to centre line dimensions underestimates the members (hence structure) stiffness [8]. The transformation matrices required to relate moments and rotations at the joint to moments and rotations at the ends of the flexible member framing into the joints can be shown to be (Fig. 3):

FIG. 3 RIGID END EFFECTS

$$\begin{Bmatrix} M'_L \\ M'_R \end{Bmatrix} = \begin{bmatrix} 1 + \nabla_L & \nabla_L \\ \nabla_R & 1 + \nabla_R \end{bmatrix} \begin{Bmatrix} M_L \\ M_R \end{Bmatrix} \quad (13)$$

$$\begin{Bmatrix} \theta'_L \\ \theta'_R \end{Bmatrix} = \begin{bmatrix} 1 + \nabla_L & \nabla_R \\ \nabla_L & 1 + \nabla_R \end{bmatrix} \begin{Bmatrix} \theta_L \\ \theta_R \end{Bmatrix} \quad (14)$$

The transformation matrices required to define the joint displacements rather than the above member deformations is given as (Fig. 4):

$$\begin{Bmatrix} M_L \\ M_R \\ V_L \\ V_R \end{Bmatrix} = \begin{bmatrix} 1 & 0 \\ 0 & 1 \\ \frac{1}{L} & -\frac{1}{L} \\ \frac{1}{L} & \frac{1}{L} \end{bmatrix} \begin{Bmatrix} M'_L \\ M'_R \end{Bmatrix} \quad (15)$$

and

$$\begin{Bmatrix} \theta'_L \\ \theta'_R \end{Bmatrix} = \begin{bmatrix} 1 & 0 & \frac{1}{L} & \frac{1}{L} \\ 0 & 1 & \frac{1}{L} & \frac{1}{L} \end{bmatrix} \begin{Bmatrix} \theta_L \\ \theta_R \\ \Delta_L \\ \Delta_R \end{Bmatrix} \quad (16)$$

M, Δ, and θ is the joint moment, sway and rotation respectively.

Collecting Eqs. 11, 13, 14, 15, and 16, the member stiffness matrix [K] in global coordinates relating joint moments and joint displacements becomes:

MEMBER DEGREES OF FREEDOM SHOWN
WITH POSITIVE SENSE

FIG. 4 TRANSFORMATION FROM MEMBER TO GLOBAL
COORDINATES

$$[K] = \begin{bmatrix} 1 & 0 \\ 0 & 1 \\ -\frac{1}{L} & -\frac{1}{L} \\ \frac{1}{L} & \frac{1}{L} \end{bmatrix} \begin{bmatrix} 1+\nabla_L & \nabla_L \\ \nabla_R & 1+\nabla_R \end{bmatrix} \begin{bmatrix} 1+\psi & \psi \\ \psi & 1+\psi \end{bmatrix} \frac{\ell}{3 EL} \begin{bmatrix} 1 & -1/2 \\ -1/2 & 1 \end{bmatrix} +$$

$$\begin{bmatrix} F_L^1 & 0 \\ 0 & F_R^2 \end{bmatrix}^{-1} \begin{bmatrix} 1+\nabla_L & \nabla_R \\ \nabla_L & 1+\nabla_R \end{bmatrix} \begin{bmatrix} 1 & 0 & -\frac{1}{L} & \frac{1}{L} \\ 0 & 1 & -\frac{1}{L} & \frac{1}{L} \end{bmatrix} \quad (17)$$

If the columns are considered extensible, then $[K]$ is augmented to⁺:

$$[K]' = \begin{bmatrix} & & 0 & 0 \\ & [K] & 0 & 0 \\ & & 0 & 0 \\ & & 0 & 0 \\ 0 & 0 & 0 & 0 & \frac{AE}{L} & \frac{AE}{L} \\ 0 & 0 & 0 & 0 & -\frac{AE}{L} & -\frac{AE}{L} \end{bmatrix} \quad (18)$$

L is the length of a member from center line to center line of the joints.

The entire structure stiffness matrix is assembled member by member through the direct correspondence between the structural coordinate system and the member degrees of freedom.

When a structure deforms, the gravity loads acting through the displaced joints introduces additional floor shears that must be carried by the structure. Consider Fig. 5, an expression for the additional shear being carried at floor i is:

⁺ with $-\frac{1}{L}$ and $\frac{1}{L}$ interchanged.

FIG. 5 P-DELTA SHEARS

$$F_i = (z_i - z_{i-1}) \frac{P_i}{h_i} - (z_{i+1} - z_i) \frac{P_{i+1}}{h_{i+1}} \quad (19)$$

where i is the level of the floor under consideration, P is the total gravity loads accumulated to the floor indicated by the subscript, and h is the floor to floor height. Eq. 19 can be written more conveniently as:

$$F_i = -\frac{P_i}{h_i} \cdot z_{i-1} + \left(\frac{P_{i+1}}{h_{i+1}} + \frac{P_i}{h_i} \right) \cdot z_i - \frac{P_{i+1}}{h_{i+1}} \cdot z_{i+1} \quad (20)$$

or in matrix notation as:

$$\begin{Bmatrix} \vdots \\ F_i \\ \vdots \end{Bmatrix} = \begin{bmatrix} \vdots & \vdots & \vdots \\ \vdots & -\frac{P_i}{h_i} & \left(\frac{P_{i+1}}{h_{i+1}} + \frac{P_i}{h_i} \right) & -\frac{P_{i+1}}{h_{i+1}} \\ \vdots & \vdots & \vdots & \vdots \end{bmatrix} \begin{Bmatrix} \vdots \\ z_{i-1} \\ z_i \\ z_{i+1} \\ \vdots \end{Bmatrix} \quad (21)$$

The coefficients thus determined are derived for all floors and added with signs reversed to the static structure stiffness matrix. Additional shears developed due to the deformation of the columns are neglected. If the gravity loads remain constant during the loading, then so do the terms of Eq. 21.

Finally the only other consideration is the dynamic terms associated with damping. Hurty and Rubinstein [9] have written an excellent chapter on this matter. At this point in time, the true mechanism of damping is not clearly understood. Fortunately however

its influence is only small. It is usual to represent the effects by an equivalent viscous model. This is actually a mathematical convenience, but apparently reasonable under present circumstances.

The constants α and β can be shown to be:

$$\alpha = \frac{2\omega_1 \omega_2 (\xi_2 \omega_1 - \xi_1 \omega_2)}{\omega_1^2 - \omega_2^2} \quad (22)$$

$$\beta = \frac{2(\xi_1 \omega_1 - \xi_2 \omega_2)}{\omega_1^2 - \omega_2^2}$$

when both mass and stiffness related damping occur, or:

$$\alpha = 0 \quad (23)$$

$$\beta = \frac{2\xi_1}{\omega_1}$$

when stiffness only is considered, or:

$$\alpha = 2\xi_1 \omega_1 \quad (24)$$

$$\beta = 0$$

when mass only is considered, where ω_1 and ω_2 are two adjacent non dimensional modal frequencies, and are usually taken for the first and second modes respectively. ξ_1 and ξ_2 are the associated modal damping ratios expressed as a % of the critical values.

2.4 EVALUATION OF THE EFFECTIVE JOINT LOAD VECTOR

THE DYNAMIC CASE

Most of the terms of Eq. 7 or 8 require no further explanation. However there are several points that should be discussed. The vector $\{\Delta\ddot{y}\}$ represents the incremental base acceleration applied over an interval. The records are usually stored on magnetic tape or disc spaced at discrete equal intervals of time. They may either be "manufactured" from some stochastic process, or the digitized end product of measurements of an actual earthquake. Whatever the source, it is imperative that the structural response obtained from such a record be viewed as having a certain bias, and conclusions drawn from the study may not be entirely true for another record. The other point is the consideration given to changes in linearity within an interval. If a member changes stiffness within a time step, then the member forces predicted on the assumption of invariant stiffness are in error. Much research is presently being conducted into ways of mitigating these effects. A popular method, and the one used here is to assume that the member deformations have been correctly computed but the adjustment of the member forces at the end of the step or substep require that unbalanced joint loads be subtracted from the joint load vector for the next step.

THE STATIC CASE

If the applied loading is static, then the dynamic terms of Eqs. 7, and 8 disappear. If the shear effects are considered

negligible, then the joint loads are found from summation of the fixed end forces at any joint in question using the simple expressions readily supplied in structural analysis or design hand books.

If the shear effects are important such as when there are shear walls or short deep beams present, then it can be shown that the expressions for the fixed end forces; M^F due to a uniformly distributed load w is:

$$M^F = \pm \frac{w \ell^2}{12} \quad (\text{as for zero shear case}) \quad (25)$$

If a concentrated load p acts at distance "a" from the left end and "b" from the right end (Fig. 6) then:

$$M_L^F = - ab \frac{(b + \psi \ell)}{\ell^2 (1 + 2\psi)} p$$

and (26)

$$M_R^F = + ab \frac{(a + \psi \ell)}{\ell^2 (1 + 2\psi)} p$$

Eqs. 25, and 26 have been coded into the program QUAKES.

2.5. MOMENT-LOAD-CURVATURE RELATIONSHIP OF A SECTION

Fig. 7 represents a typical moment-load-curvature relationship of a section of a typical reinforced concrete member. It should be noted that unlike structural steel, increase in axial load

FIG. 6 JOINT LOADS FROM GRAVITY LOADS

FIG. 7 MOMENT-AXIAL LOAD-CURVATURE PRIMARY CURVE

(provided the load is less than the balanced load), causes increase in the ultimate moment. Park and Paulay (7) present an excellent discussion on a mathematical procedure for calculating the cardinal points of the curve, viz the cracking, yielding and ultimate points, given the cross section dimensions, the amount, location, and grade of the reinforcement, Young's Modulus for the concrete and the stress-strain curves of both the concrete and the reinforcement (Fig. 8).

For a beam, the points can be established explicitly and directly, because the axial load is zero, and usually there are only two levels of reinforcement. Columns however are more complex. It is necessary to use an iterative method to find a neutral axis location that produces internal forces that are in balance with the external forces. In addition the reinforcement is usually distributed around all faces, so the point of initial tensile reinforcement yielding is not sharply defined. The smaller of the two programs embodies the theory of Park and Paulay, and outputs for columns, a graph of the moment-load-curvature relationship, thus it becomes an easy matter to define a yield point.

2.6 HYSTERESIS RULES

Experimental work by Clough [6] and Takeda et al [3] indicates that reinforced concrete members do not exhibit a bilinear hysteresis relationship. Rather, the members degrade as a function of their stress history. Characteristically after several load

[illegible]

FIG. 8 STRESS-STRAIN RELATIONSHIPS

cycles, a member will not reload parallel to the initial elastic slope but will aim towards the maximum point attained in that direction at a previous cycle (Fig. 9).

Sozen [10] presented a paper that examined in detail the mechanism of stiffness degradation. He confirmed what Clough had said earlier, that degrading behaviour is closely related to the crack closing phenomenon.

Prior to discussing the various rules governing behaviour it is instructive to derive the spring primary moment-rotation curves. Theoretically, these curves are curvilinear. In practice it is common to treat them as tri-linear or sometimes as bi-linear. Assuming that the moment curvature relationship for a section has been found by application of the program here or by some other method, then with reference to Fig. 10, the following relationships for cracking, yielding and ultimate rotations result.

$$\theta_c = \frac{M_c l}{6 EI}$$

$$\theta_y = \frac{l}{6} [(1 - \lambda_1^3) \phi_y + \lambda_1^2 \phi_c]$$

(27)

$$\theta_u = \frac{l}{12} [(2 + \lambda_2)(1 - \lambda_2)(1 + \lambda_2(\lambda_3 - 1))/\lambda_3$$

$$+ \lambda_2(1 + \lambda_2) - 2\lambda_1^3] \phi_y/\lambda_2 + 2\lambda_1^2 \phi_c]$$

where ϕ represents the curvature, and where

ORDINARY ELASTO-PLASTIC MODEL

DEGRADING STIFFNESS MODEL

FIG. 9 HYSTERESIS MODELS

FIG. 10 DERIVATION OF CRACKING, YIELDING, AND ULTIMATE ROTATIONS

$$\lambda = \frac{M_c}{M_y}, \lambda_1 = \frac{M_c}{M_u}, \lambda_2 = \frac{M_y}{M_u}, \text{ and } \lambda_3 = \frac{(M_u - M_y)}{(\phi_u - \phi_y)} \frac{\phi_y}{M_y}$$

It is assumed above that the tension and compression reinforcement areas are equal and continuous over the entire member length.

Sixteen rules that represent practical application at Takeda's work are (Fig. 11) [2]:

- Rule 1: Loading on the primary curve before cracking.
- Rule 2: Loading on the primary curve before yielding.
- Rule 3: Loading on the primary curve after yielding.
- Rule 4: Unloading from the maximum point attained during Rule 2.
- Rule 5: Unloading from the maximum point attained during Rule 3.
- Rule 6: Reloading after Rules 4 or 5 towards the maximum point on the primary curve.
- Rule 7: Unloading without reattaining the moment level of Rule 6.
- Rule 8: Reloading after Rule 7 towards the maximum point on the primary curve.
- Rule 9: Unloading without reattaining the moment level of Rule 8.
- Rule 10: Reloading towards the maximum point attained during Rule 6.
- Rule 11: Unloading from the maximum point attained during Rule 10.
- Rule 12: Loading after Rule 11 towards the maximum point attained during Rule 8.
- Rule 13: Unloading from the maximum point attained during Rule 12.
- Rule 14: Loading in the uncracked direction after cracking in the other direction.
- Rule 15: Loading in the uncracked direction after yielding in the other direction.
- Rule 16: Loading towards the yield point after Rule 15.

- 1: Rule 1
- 2: Rule 2
- 3: Rule 4
- 4: Rule 14
- 5: Rule 2
- 6: Rule 3
- 7: Rule 5
- 8: Rule 6

- 1: Rule 1
- 2: Rule 2
- 3: Rule 3
- 4: Rule 5
- 5: Rule 15
- 6: Rule 16
- 7: Rule 3
- 8: Rule 5
- 9: Rule 6

SEVERAL INITIAL POSSIBILITIES

FIG. 11 HYSTERESIS RULES

1: Rule 1	8: Rule 9
2: Rule 2	9: Rule 10
3: Rule 3	10: Rule 11
4: Rule 5	11: Rule 12
5: Rule 6	12: Rule 13
6: Rule 7	13: New Rule 10
7: Rule 8	

Note: Slopes 6, 8, 10, and 12 are a user controlled multiple of slope 4.

LOOPS REDUCING

1: Rule 1	7: Rule 8
2: Rule 2	8: Rule 9
3: Rule 3	9: Rule 10
4: Rule 5	10: New Rule 6
5: Rule 6	11: New Rule 7
6: Rule 7	12: New Rule 8

LOOPS REDUCING THEN INCREASING
- SEVERAL LATER POSSIBILITIES -

FIG. 11 (CONT'D) HYSTERESIS RULES

- NON DEGRADING MODEL -

FIG. 11-(CONT'D) HYSTERESIS RULES

Non degrading behaviour (typical for structural steel members) can be achieved by applying only Rules 1, 2, 3, 4, 5, and 15. Obviously the computational effort involved with the former model is much larger than the latter because the chances of linearity changes occurring during a time step are so much greater. The inelastic spring rotations are as shown in Fig. 12. The influence of shear on the spring properties can be included by specifying the shear modulus and member shear areas.

2.7 DUCTILITY DEMAND

A structure must be able to redistribute loads accumulated at critical points by inelastic action. This requires that the members undergo substantial inelastic deformations without failure. The deformations should not be that severe so as to render the building unusable however. Thus the ductility demand must be measured and compared to a tolerable level.

The best measure of ductility demand is the maximum strain. In practice however, ductility is defined in terms of either floor displacements, member curvature, or spring rotations. Displacement ductility demand can be a useful criterion but it masks the demand of the members. It is also difficult to define. As the inelastic curvatures are assumed as concentrated, the only method available here is the hinge rotations. The program keeps track of the maximum and minimum rotations of each spring. Finally these values are compared to their yield rotations and output as ductility ratios. No modification is made for the presence of gravity loads however.

SPRING PRIMARY CURVE WITH ELASTIC AND INELASTIC ROTATIONS

SPRING PRIMARY CURVE WITH INELASTIC ROTATIONS ONLY

FIG. 12 REPRESENTATION OF SPRING INELASTIC PROPERTIES

2.8 INTEGRATION INTERVAL

Considerable care must be exercised when selecting an integration interval. Difficulties that include numerical stability, truncation and other errors, convergency, disturbances due to abrupt changes of stiffness, and computational efficiency are a function of the size of the interval.

It has been suggested [11] that the interval should not exceed the smallest of:

1. One tenth of the period of the highest mode that has a significant effect on the elastic internal forces.
2. One fortieth of the first mode period.
3. The discretization interval of the earthquake records.

3. PROGRAM SPECIFICATIONS

3.1 PROGRAM STRUCTURE

The dynamic analysis program consists of approximately 3700 executable and non executable statements. The statements are organized into a main program, one sub-program and nine subroutines. All arguments are passed between the main program, the sub-program, and the subroutines through Labelled Common. From a staging point of view the program is divided into 3 stages:

1. Preliminary work.
2. Static analysis.
3. Dynamic analysis.

Each of the 11 program elements identified above has been separately stored as a ROM (Relocatable Object Module). Thus the load module is simply a linkage of all the ROMS. In the event that the program has to be altered, only the particular ROM or ROMS requiring change have to be recompiled. Thus an updated load module is very easy to build.

The following table identifies the program element, its ROM designation and its function. It should be noted that the order of sequence of the first two elements is mandatory.

PROGRAM ELEMENT NAME	ROM NAME	ELEMENT TYPE	COMPILED SEQUENCE STATUS	FUNCTION
BLOCK DATA	Q1	Sub-program	As shown.	Initialize all inter program element data.
MAIN	Q32	Main program	As shown.	Program control.
SOLVER	Q33	Subroutine	Optional.	Compute joint displacements.
SKCOPY	Q34	do	do	Transfer updated stiffness information from storage to the working area.
FORMSK	Q37	do	do	Build or update the structure stiffness matrix.
FORCES	Q8	do	do	Compute beam and column actions and deformations.
MOTION	Q9	do	do	Compute velocities, accelera- tions, sways, relative sways, floor shears, and overturning moments.
MINMAX	Q10	do	do	Update seismic response envel- opes.
MRHYST	Q11	do	do	Examine springs for change in linearity.
SWITCH	Q12	do	do	Interchange positive and nega- tive spring indexes.
EQMCOR	Q13	do	do	Compute unbalanced joint loads.

The other program is made up of approximately 300 executable and non executable statements. There is only a main program, however there are two distinct sections. Their function in order of coding is:

1. Compute the beam and column transformed section properties and the beam moment-curvature primary curve.
2. Compute the column moment-axial load-curvature primary curve.

3.2 FORTRAN VARIABLES

Should the user be interested in the program variable names used and their meaning, he should refer to a copy of a current program listing. The number of variables used is many and the logic employed is quite complex.

To assist the user when reading the program however, considerable effort has been directed towards making the program a self contained, readily understood document. This is achieved by adding considerable comment throughout the program that states the intention of the various passages of coding and the theoretical background as well. All major variables and many minor variables are defined by comments where they first appear. Also variable names have been chosen to bear a strong resemblance to the intended meaning, e.g. OTM is a variable name representing the over turning moments in a structure, BR () is a matrix that represents beam rotations etc. In fact the characters B, C, and F prefix most variables associated with beams, columns and floors respectively.

3.3 INTERNAL STORAGE OF THE STRUCTURE STIFFNESS MATRICES

Two structural stiffness matrices are used [12]. They are identical in size. The matrix referenced in the simultaneous equation solving process referred to as the working matrix, is at any time either partly or completely decomposed. On the other hand the other matrix referred to as the storage matrix is always in its original form. If a member stiffness matrix changes because of non linearities, the stiffness contributions of that member are removed from the storage matrix and replaced by updated values. As the p-delta effects are defined at the structure rather than the member level their contributions to the storage matrix once added, remain unchanged for the duration of the study.

Thus if a number of members change stiffness during a time step, only that area of the storage matrix that is updated is read to the working matrix. This area is defined by the structural coordinate corresponding to the lowest rotational degree of freedom affected in the updating process, and the structural coordinate corresponding to the total number of degrees of freedom. Because the working matrix is decomposed column by column from left to right [13], it is sufficient then to start decomposition only in that column that is the lower bound of the area defined above.

Considerable saving in computational effort is thus possible. Further savings can be realized if the half band width of each row is known in advance. This feature eliminates zero multiplications and is used in this program. As the greater portion of seismic

computations stem from the assembly and solving of the simultaneous equations, it follows that processing costs would be kept to minimum.

Consider Figs. 13, 14, it is seen that the structural stiffness matrix is symmetrical and banded. All elements outside the band will be zero, and if the elements within one half band including the diagonal are stored in the computer, all relevant information will be available. In keeping with the Crout reduction method, only the upper band and diagonal are stored in a special rectangular matrix of size NDF by $MBAND$. Thus the diagonal elements of the square form occupy the first column in the rectangular form. Hence, if the address of any element is given as i, j in square form; the corresponding address in rectangular form is $i, j - i + 1$. Fig. 14, shows the relationship of the working and the storage matrices in square form, and indicates the area of the storage matrix read to the working matrix for decomposition.

3.4 DATA LIMITATIONS

Both programs have been written in standard Fortran IV and with slight modifications, will run on any machine that stores a Fortran compiler. Since the dynamic analysis program computes incremental displacements, velocities, accelerations, and forces, all real variables have been compiled in double precision (64 bits) to keep the truncation errors to a minimum.

FIG. 13 STIFFNESS MATRICES CONTRIBUTING TO THE
WORKING STIFFNESS MATRIX

MAXIMUM
HALF BAND

1	2	3	4	5	6
7	8	9	10	11	0
12	13	14	15	16	
17	18	19	20	21	22
23	24	25	26	27	
28	29	30	31		
32	33	34			
35	36			0	
37					

$(i, j-i+1)$

NDF

1	2	3	4	5	6			
2	7	8	9	10	11		0	
3	8	12	13	14	15	16		
4	9	13	17	18	19	20	21	22
5	10	14	18	23	24	25	26	27
6	11	15	19	24	28	29	30	31
		16	20	25	29	32	33	34
	0		21	26	30	33	35	36
			22	27	31	34	36	37

NDF

$(i, j) = (j, i)$

ELEMENT ADDRESS RELATIONSHIP

REGION TO BE
DECOMPOSED

- WORKING MATRIX -

STORED FORM

SQUARE FORM

- STORAGE MATRIX -

FIG. 14 STIFFNESS MATRICES : RELATIONSHIP BETWEEN
AREA TO BE READ AND AREA TO BE REDUCED

The limiting dimensions of the present version of the dynamic analysis program are as follows:

No. of degrees of freedom (NDF)	95
Band width (MBAND)	15
No. of stories (NS)	12
No. of beams (NB)	32
No. of columns (NC)	42

The core requirements in decimal words can be estimated from⁺:

$$\text{Core} = \text{NB} (90p + 10) + \text{NC} (100p + 10) + \text{NS} ((2\text{NS} + 15)p + 2) \\ + \text{NDF} ((2\text{MBAND} + 4)p + 1) + 360p + 11200 + \text{OSR}$$

where $p = 2$ if all real variables are compiled in double precision.

$p = 1$ if all real variables are compiled in single precision.

$1 < p < 2$ if a mixture of single and double precision is used,

and

$\text{OSR} = 2100$ and represents the sigma 9's operating system requirements.

A 10 story, 3 bay frame can be analysed within these dimensions.

The limitations of the other program are:

Levels of reinforcement	5
Units	kips, inches
28 day compressive strength of concrete	4 k.s.i
Yield strength of the reinforcement	60 k.s.i
Young's modulus for the reinforcement	29000 k.s.i
Young's modulus for the concrete	3605 k.s.i
Modular ratio	8

⁺This value is 1000 times the value coded for the core option on the limit card.

3.5 NUMBERING CONVENTIONS

The dynamic analysis program will analyse any right angled multistory frame with rigid joints. The columns may be pinned or fixed to the rigid base. Frames with setbacks as shown in Fig. 15 can be handled.

Before preparing the data, the structural degrees of freedom have to be numbered. A specific scheme of numbering has been adopted to take advantage of the banding of the structural stiffness matrix. As shown in Fig. 15, starting from the lowest story, the story rotational degrees of freedom are numbered first, then the degree of freedom corresponding to the side sway deflection of the story is numbered and lastly the degrees of freedom relating to column extensions are numbered. The positive sense of the degrees of freedom is also shown in Fig. 15.

The above scheme ensures minimum band width for the structural stiffness matrix, the value of band width being equal to the maximum difference between a rotational degree of freedom in one story and the degree of freedom corresponding to side sway of the story above or below-plus one. Thus using Fig. 15, the maximum difference between the above said degrees of freedom is $20 - 1 = 19$, and thus the maximum band width is $19 + 1 = 20$.

The convention adopted for numbering the element degrees of freedom is shown in Fig. 4. It will be observed that there are

FIG. 15 EXAMPLE FRAME FOR CODING CONVENTION

four degrees of freedom associated with a beam and six degrees of freedom associated with a column.

The beams and columns also have to be assigned numbers by which they may be identified at input and output. The only restriction on the numbering scheme is that the first number of each member type is 1, and that all other numbers follow consecutively. It will be found convenient to, say, start from the lowest story and proceed from left to right in a story.

Having numbered the degrees of freedom of the beams and columns, it is straight forward to work out the structural coordinate numbers that correspond to element degrees of freedom. Thus in Fig. 15, for column 7,

$$\text{NDFC}(7,1) = 1$$

$$\text{NDFC}(7,2) = 14$$

$$\text{NDFC}(7,3) = 7$$

$$\text{NDFC}(7,4) = 20$$

$$\text{NDFC}(7,5) = 8$$

$$\text{NDFC}(7,6) = 21$$

where NDFC is the name of the array storing the structural coordinate numbers corresponding to column degrees of freedom. For columns like 1, 2, 3, 4, 5, and 6, it will be observed that there are no structural coordinates corresponding to element degrees of freedom 1, 3, and 5.

In such cases, corresponding structure coordinates default to zero.

If a column is pinned to the base however, the structural coordinate associated with the rotation degree of freedom 1 must be assigned the value '99999', by the user.

3.6 SIGN CONVENTION

Joint rotations, displacements and moments are positive in the positive directions of structural degrees of freedom shown in Fig. 4. The axial force is considered positive when compressive.

Uniformly distributed or concentrated loads acting on a beam are considered positive when acting downwards.

If the user requests a static analysis or a combined static-seismic analysis and he elects to input the static fixed end forces directly rather than details of discrete gravity loads, then it is imperative that the proper sign of the force be attached.

3.7 UNITS

Any set of consistent units may be used in the dynamic analysis program. The program does not carry out any transformation. A typical set of units is shown below.

Length	Inches (in)
Moment of inertia	In ⁴
Area	In ²
Acceleration	In/sec ²
Velocity	In/sec

Modulus of elasticity	kips/in ²
Uniformly distributed load	kips/in
Concentrated load	kips

3.8 OUTPUT FORMAT

The bulk of the results of the modal, static, and dynamic analyses are in tabular form with appropriate page and other headings. Individual beams, columns, or floors are identified sequentially to the left of the page with all associated data tabulated to the right.

Moment-curvature relationships for beams are presented in tabular form, first for the case when the compression reinforcement is at the top of the beam, and then for the case when it is at the bottom. That is the primary curve is established for both loading directions.

The entire relationship for the columns is not given explicitly however. Columns are usually heavily reinforced and so their yield point is not always clear. Thus the information is presented for a set of curvatures ranging, from zero to the ultimate value, in both tabular and plotted form. Hence some judgement must be exercised when selecting the yield point. The cracking and ultimate points are explicitly given however. It is strongly recommended that for the case where tension and compression reinforcements of a beam are equal, the user treat the beam as a column with an

extremely small value of axial load (e.g. 1 kip). The program coding for the columns is more suitable in this case. Note that a maximum of 10 moment-curvature points are given. If none are given, it is to be understood that the column is heavily loaded and completely in compression.

The user may output story responses at any desired interval of time. This is useful if the history of the responses are required from the beginning of the earthquake simulation to its termination. If however only the response envelopes are required then this can be achieved by simply leaving the appropriate field on the data card blank.

To allow some flexibility in the QUAKE's output format, the number of printed lines per listing page have been set as a user controlled variable. The default value is 43.

3.9 DIAGNOSTICS

Neither of the programs provide error checks at this stage. If a run time problem is encountered, it is suggested that the output listing of the input data be thoroughly checked for errors.

If the constant acceleration method is used, then the damping constants α and β can be zero or non zero. If however the linear acceleration method is used, then β must be zero, but the value of α is optional. The reason for the latter requirements is that the unreduced stiffness matrix is used, therefore the frequencies

associated with axial extensions are extremely high. (Vertical inertial mass is zero), thus instability occurs, and the computations rapidly increase to very large numbers.

There are several eventualities that might arise when computing column moment-load-curvature relationships. Normally 10 moment-curvature points are output per column. Sometimes however the number may be less than 10. This is due to the fact that difference between the internal and external forces exceeded the allowable tolerance coded into the program. This should not present a problem however. If on the other hand no points are output this should be taken to mean that the neutral axis lies beyond the confines of the section, that is, the axial load in the member is quite significant. As the spring properties are defined at the working level however, this situation will rarely if ever happen.

If the user intends to perform a static or a static-seismic analysis he is cautioned to ensure that the frame is capable of responding completely elastically. Thus there should not be a problem for a simple static analyses when gravity loads defined at working level are applied. Similarly there should be no difficulty when performing an elastic seismic analysis or an elastic-plastic seismic analysis. However if the primary curve of the hysteresis relationship is trilinear as may be the case when concrete members are used, then at present there is no adequate method in the program of adequately predicting the internal response, if cracking occurs,

and so inclusion of gravity loads may severely disturb the seismic response rendering the computed results meaningless.

4.0 DESCRIPTION OF INPUT DATA

The following tables provide comprehensive information on the identification, format and sequence of the data required to undertake any particular analysis.

PROGRAM: QUAKE

CARD TYPE	VARIABLE DESCRIPTION	VARIABLE TYPE	FORTRAN FORMAT	CARDS REQUIRED PER CARD TYPE	ANALYSIS PURPOSE				COMMENTS
					MODAL	STATIC	SEISMIC	STATIC-SEISMIC	
A	Job identification.	Alphanumeric	20A4	1	✓	✓	✓	✓	Can be blank.
B	Earthquake record.	"	"	1	x	x	✓	✓	" " "
C	Consistent physical units of mass, force, length, & time.	"	4(A3,2X)		✓	✓	✓	✓	" " "
	Maximum number of printed lines per page.	Integer	I5	1	✓	✓	✓	✓	Default = 43.
	Interval between output of seismic response.	"	"		x	x	✓	✓	Eg., code 10 for output each 10th cycle.
D	Young's Modulus.	Real	F10		✓	✓	✓	✓	Leave any value blank if appropriate.
	Shear Modulus.	"	"		✓	✓	✓	✓	If space for shear modulus is left blank, deformations due to shear will not be considered.
	Mass related damping factor.	"	"		x	x	✓	✓	
	Stiffness related damping factor.	"	"		x	x	✓	✓	
	Gravity constant.	"	"						

CARD TYPE	VARIABLE DESCRIPTION	VARIABLE TYPE	FORTRAN FORMAT	CARDS REQUIRED PER CARD TYPE	ANALYSIS PURPOSE			COMMENTS
					MODAL	STATIC	SEISMIC	
E	Number of;	Integer	I5		✓	✓	✓	Number of loaded beams may be blank.
	Beams	"	"		x	✓	✓	
	Loaded beams.	"	"		✓	✓	✓	
	Columns.	"	"		✓	✓	✓	
	Stories.	"	"		✓	✓	✓	
	Kinematic degrees of freedom.	"	"	1	✓	✓	✓	
	Maximum half band width including the diagonal.	"	"		✓	✓	✓	
	Analysis purpose.	"	"		✓	x	x	
F	Earthquake duration.	Real	F10	1	x	x	✓	Values can be blank as appropriate.
	Acceleration scaling factor.	"	"		x	x	✓	Linear method =6 Constant method=4
	Reciprocal of Newmark's beta.	"	"		x	x	✓	

CARD TYPE	VARIABLE DESCRIPTION	VARIABLE TYPE	FORTRAN FORMAT	CARDS REQUIRED PER CARD TYPE	ANALYSIS PURPOSE ✓ = required x = blank * = card not required			COMMENTS
					MODAL	STATIC	SEISMIC	
F (cont)	Time step of numerical integration (eg, .001 sec)	Real	F10		x	x	✓	Computations retraced if step not equal to sub-step.
	Sub time step (eg, .0001 sec)	"	"		x	x	✓	
	Initial base velocity.	"	"		x	x	✓	
	P-Delta scaling factor.	"	"		✓	✓	✓	
	Acceleration record interval (eg, .01 secs)	"	"		x	x	✓	Assumed constant with records on disc.
G	Option for defining spring spine.	Integer	I5	1	x	x	✓	Curvature; blank rotations; = 1
	Spring unloading factor.	Real	F10		x	x	✓	Degrading model only (eg, 1.).
	Spring reloading factor wrt elastic flexibility.	"	"		x	x	✓	Degrading; blank, non-degrading; value as reqd (eg, 1.).
	Option for requesting printing of hinge yielding and unyielding	Integer	I5		x	x	✓	Required; = 1 not required; blank.

CARD TYPE	VARIABLE DESCRIPTION	VARIABLE TYPE	FORTRAN FORMAT	CARDS REQUIRED PER CARD TYPE	ANALYSIS PURPOSE ✓ = required x = blank * = card not required				COMMENTS
					MODAL	STATIC	SEISMIC	STATIC-SEISMIC	
H ₁	Beams:	Integer	I5	See comment for card type: H4	✓	✓	✓	✓	
	Identification.								
	Structural coordinate:								
	Rotation left end.				✓	✓	✓	✓	
	Rotation right end.				✓	✓	✓	✓	
	Uplift left end.				✓	✓	✓	✓	
	Uplift right end.				✓	✓	✓	✓	
	Rigid length left end.	Real	F5		✓	✓	✓	✓	Can be blank.
	Clear span.	"	"		✓	✓	✓	✓	
	Rigid length right end.	"	"		✓	✓	✓	✓	" "
	Effective shear area.	"	"		✓	✓	✓	✓	Can leave blank if shear not to be considered.
	Moment of inertia.	"	F10		✓	✓	✓	✓	

CARD TYPE	VARIABLE DESCRIPTION	VARIABLE TYPE	FORTRAN FORMAT	CARDS REQUIRED PER CARD TYPE	ANALYSIS PURPOSE				COMMENTS
					MODAL	STATIC	SEISMIC	STATIC-SEISMIC	
H ₂	Beam Spring Spine:								
	+cracking moment-left end.	Real	F10	See comment for card type: H4	*	*	✓	✓	Rotations generated automatically. For steel beams, input yield moment.
	- " " "	"	"		*	*	✓	✓	
	+cracking moment-right end.	"	"		*	*	✓	✓	
H ₃	- " " "				*	*	✓	✓	
	+yielding moment-left end.	"	"	"	*	*	✓	✓	For steel beams, input plastic moment.
	- " " "	"	"		*	*	✓	✓	
	+yielding moment-right end.	"	"		*	*	✓	✓	
	- " " "	"	"		*	*	✓	✓	
	+yielding rotation-left end.	"	"		*	*	✓	✓	Rotation or curvature. For steel beams use rotations only.
	- " " "	"	"		*	*	✓	✓	
	+yielding rotation-right end.	"	"		*	*	✓	✓	
	- " " "	"	"		*	*	✓	✓	
	+yielding rotation-right end.	"	"		*	*	✓	✓	

CARD TYPE	VARIABLE DESCRIPTION	VARIABLE TYPE	FORTRAN FORMAT	CARDS REQUIRED PER CARD TYPE	ANALYSIS PURPOSE				COMMENTS
					MODAL	STATIC	SEISMIC	STATIC-SEISMIC	
H ₄	+ ultimate moment-left end.	Real	F10	1 card each of card types H ₁ , H ₂ , H ₃ , and H ₄ per beam	*	*	✓	✓	For steel beams with quasi elastic-plastic spine add 10(Say) to value of plastic moment.
	- " " "	"	"		*	*	✓	✓	
	+ ultimate moment-right end.	"	"		*	*	✓	✓	
	- " " "	"	"		*	*	✓	✓	
	+ ultimate rotation-left end.	"	"		*	*	✓	✓	
	- " " "	"	"		*	*	✓	✓	
	+ ultimate rotation-right end.	"	"		*	*	✓	✓	
	- " " "	"	"		*	*	✓	✓	
					*	*	✓	✓	
I ₁	Column: Identification	Integer	I5	See comment for card type I4	✓	✓	✓	✓	
	Structural coordinate rotation-lower end	"	"		✓	✓	✓	✓	

CARD TYPE	VARIABLE DESCRIPTION	VARIABLE TYPE	FORTRAN FORMAT	CARDS REQUIRED PER CARD TYPE	ANALYSIS PURPOSE			COMMENTS
					MODAL	STATIC	SEISMIC	
I ₁	Rotation-upper end.	Integer	I5		✓	✓	✓	For columns attached to foundation leave lower values blank. If pinned to foundation; then as above but input 99999 for rotational degree of freedom no : 1.
	Sway-lower end.	"	"		✓	✓	✓	
	Sway-upper end	"	"		✓	✓	✓	
	Axial deformation-lower end.	"	"		✓	✓	✓	
	Axial deformation-upper end.	"	"		✓	✓	✓	
	Rigid length-lower end.	Real	F5		✓	✓	✓	Can be blank.
	Clear span.	"	"		✓	✓	✓	"
	Rigid length-upper end.	"	"		✓	✓	✓	"
	Cross section area.	"	"		✓	✓	✓	"
	Effective shear area.	"	"		✓	✓	✓	"
	Moment of inertia	"	F10		✓	✓	✓	"

CARD TYPE	VARIABLE DESCRIPTION	VARIABLE TYPE	FORTRAN FORMAT	CARDS REQUIRED PER CARD TYPE	ANALYSIS PURPOSE				COMMENTS
					MODAL	STATIC	SEISMIC	STATIC-SEISMIC	
I ₂	Column spring spine;	Real	F10	See comment card type I ₄	*	*	✓	✓	See comments for beam springs.
	+ cracking moment-lower end.	"	"		*	*	✓	✓	
	- " " "	"	"		*	*	✓	✓	
	+ cracking moment-upper end.	"	"		*	*	✓	✓	
I ₃	- " " "	"	"		*	*	✓	✓	For both beam and column springs, absolute values for cracking yielding and ultimate values required.
	+ yielding moment-lower end.	"	"		*	*	✓	✓	
	- " " "	"	"		*	*	✓	✓	
	+ yielding moment-upper end.	"	"		*	*	✓	✓	
	- " " "	"	"		*	*	✓	✓	
	+ yielding moment-lower end.	"	"		*	*	✓	✓	
	- " " "	"	"		*	*	✓	✓	
	+ yielding rotation-lower end.	"	"		*	*	✓	✓	

CARD TYPE	VARIABLE DESCRIPTION	VARIABLE TYPE	FORTRAN FORMAT	CARDS REQUIRED PER CARD TYPE	ANALYSIS PURPOSE			COMMENTS
					MODAL	STATIC	SEISMIC	
I ₃	+ yielding rotation-upper end	Real	F10	See comment for card type: I ₄	*	*	✓	
I ₄	- " " "	"	"	1 card each of card types I ₁ , I ₂ , I ₃ and I ₄ per column.	*	*	✓	
	+ ultimate moment-lower end	"	"		*	*	✓	
	- " " "	"	"		*	*	✓	
	+ ultimate moment-upper end	"	"		*	*	✓	
	- " " "	"	"		*	*	✓	
	+ ultimate rotation-lower end	"	"		*	*	✓	
	- " " "	"	"		*	*	✓	
	+ ultimate rotation-upper end	"	"		*	*	✓	
	- " " "	"	"		*	*	✓	
J	Gravity loading option.	Integer	I5	1	x	✓	x	No loads; blank, fixed end forces; 1, input loads; 2

CARD TYPE	VARIABLE DESCRIPTION	VARIABLE TYPE	FORTRAN FORMAT	CARDS REQUIRED PER CARD TYPE	ANALYSIS PURPOSE			COMMENTS
					MODAL	STATIC	SEISMIC	
K	Loaded beams; Identification.	Integer	I5	1 card per loaded beam under this option	*	✓	*	The user must supply the appropriate signs. Shear deformation effects are assumed to be included. Omit if gravity load option #1.
	Fixed end moment-left end.	Real	F10		*	✓	*	
	Fixed end moment-right end.	"	"		*	✓	*	
	Fixed end shear-left end.	"	"		*	✓	*	
	Fixed end shear-right end.	"	"		*	✓	*	
K	Loaded beams; Identification.	Integer	I5	1 card per loaded beam under this option	*	✓	*	Can be blank. " " " Repeat as necessary. Maximum=3 Loads. Omit gravity load option #2.
	Number of concentrated loads.	"	"		*	✓	*	
	Value of uniformly distributed load	Real	F10		*	✓	*	
	Location of concentrated load wrt the left end (face of joint)	"	6F10		*	✓	*	
	Magnitude of load	"			*	✓	*	

CARD TYPE	VARIABLE DESCRIPTION	VARIABLE TYPE	FORTRAN FORMAT	CARDS REQUIRED PER CARD TYPE	ANALYSIS PURPOSE ✓ = required x = blank * = card not required				COMMENTS
					MODAL	STATIC	SEISMIC	STATIC-SEISMIC	
L	Story Identification	Integer	I5	One card per story	✓	✓	✓	✓	
	Structural coordinate associated with sway	"	"		✓	✓	✓	✓	
	Mass	Real	F10		✓	x	✓	✓	

7. PROGRAM APPLICATION EXAMPLES

EXAMPLE 1:

A modal analysis is required for the frame shown on the coding form. Joint dimensions are to be included in the assembly of the structural stiffness matrix but neither the $P-\Delta$ nor the shear deformation effects are to be included. The units selected are kips and inches.

The input data is shown on pages 69 to 75, and the results of the analysis including mode shapes and frequencies are given on pages 76 to 82.

IBM

A statement was filed with the Board on May 1987 in compliance for purchasing statements from this paper.

FORTRAN CODING FORM

$$\begin{aligned} & \text{For } x \in \mathbb{R}^n, \text{ let } \mathcal{H}_x = \{y \in \mathbb{R}^n : y \cdot x = 0\} \\ & \text{and } \mathcal{H}_x^\perp = \{y \in \mathbb{R}^n : y \cdot x = 0\}^\perp. \end{aligned}$$

DATE _____

PUNCHING INSTRUCTIONS

GRAPHIC
PINCH

2. Results

FORTRAN STATEMENT

STATEMENT NUMBER		FORTNIGHT STATEMENT																														TOTAL																																																	
NO.	DATE	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40	41	42	43	44	45	46	47	48	49	50	51	52	53	54	55	56	57	58	59	60	61	62	63	64	65	66	67	68	69	70	71	72	73	74	75	76	77	78	79	80
6	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40	41	42	43	44	45	46	47	48	49	50	51	52	53	54	55	56	57	58	59	60	61	62	63	64	65	66	67	68	69	70	71	72	73	74	75	76	77	78	79	80												
7	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40	41	42	43	44	45	46	47	48	49	50	51	52	53	54	55	56	57	58	59	60	61	62	63	64	65	66	67	68	69	70	71	72	73	74	75	76	77	78	79	80																			
8	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40	41	42	43	44	45	46	47	48	49	50	51	52	53	54	55	56	57	58	59	60	61	62	63	64	65	66	67	68	69	70	71	72	73	74	75	76	77	78	79	80																				
9	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40	41	42	43	44	45	46	47	48	49	50	51	52	53	54	55	56	57	58	59	60	61	62	63	64	65	66	67	68	69	70	71	72	73	74	75	76	77	78	79	80																					
10	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40	41	42	43	44	45	46	47	48	49	50	51	52	53	54	55	56	57	58	59	60	61	62	63	64	65	66	67	68	69	70	71	72	73	74	75	76	77	78	79	80																						
11	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40	41	42	43	44	45	46	47	48	49	50	51	52	53	54	55	56	57	58	59	60	61	62	63	64	65	66	67	68	69	70	71	72	73	74	75	76	77	78	79	80																							
12	30	31	32	33	34	35	36	37	38	39	40	41	42	43	44	45	46	47	48	49	50	51	52	53	54	55	56	57	58	59	60	61	62	63	64	65	66	67	68	69	70	71	72	73	74	75	76	77	78	79	80																														
13	37	38	39	40	41	42	43	44	45	46	47	48	49	50	51	52	53	54	55	56	57	58	59	60	61	62	63	64	65	66	67	68	69	70	71	72	73	74	75	76	77	78	79	80																																					
14	38	39	40	41	42	43	44	45	46	47	48	49	50	51	52	53	54	55	56	57	58	59	60	61	62	63	64	65	66	67	68	69	70	71	72	73	74	75	76	77	78	79	80																																						
15	39	40	41	42	43	44	45	46	47	48	49	50	51	52	53	54	55	56	57	58	59	60	61	62	63	64	65	66	67	68	69	70	71	72	73	74	75	76	77	78	79	80																																							
16	46	47	48	49	50	51	52	53	54	55	56	57	58	59	60	61	62	63	64	65	66	67	68	69	70	71	72	73	74	75	76	77	78	79	80																																														
17	51	52	53	54	55	56	57	58	59	60	61	62	63	64	65	66	67	68	69	70	71	72	73	74	75	76	77	78	79	80																																																			

IBM

FORTRAN CODING FORM

FORTRAN CODING FORM

STATEMENT NUMBER	DATE	PUNCHING INSTRUCTIONS	GRAPHIC PUNCH	CLASSIFICATION NUMBER	REVISION NUMBER
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75
DATA

A. Standard Form 100 (Rev. 10-1-60) is available for purchasing information from the Bureau.

PAGE 1

*** QUAKE ***

PROGRAM FOR THE ELASTIC-INELASTIC SEISMIC RESPONSE
OF DEGRADING REINFORCED CONCRETE STRUCTURES.
A.L. MORRIS OTTAWA 1977.

10 STORY RETRACK CONCRETE FRAME/MODAL ANALYSIS- CRACKED I, JOINT DIMS REAL/AM, 78
WITHOUT P-Delta EFFECTS.

** INPUT DATA **

BEAM PROPERTIES

MEM.	STRUCTURAL COORDS.				<	RIGID		LENGTH CLEAR SPAN	>	RIGID RIGHT	SHEAR AREA	MOMENT OF INERTIA
	1	2	3	4		LEFT	RIGHT					
1	1	2	3	4		.95000E 01	.12000E 02	.26650E 03		.12000E 02	.00000E 00	.76300E 04
2	2	3	4	5		.12000E 02	.12000E 02	.26400E 03		.12000E 02	.00000E 00	.67900E 04
3	3	4	5	6		.12000E 02	.12000E 02	.26650E 03		.12000E 02	.00000E 00	.76300E 04
4	10	11	12	13		.95000E 01	.12000E 02	.26650E 03		.12000E 02	.00000E 00	.73000E 04
5	11	12	13	14		.12000E 02	.12000E 02	.26400E 03		.12000E 02	.00000E 00	.61500E 04
6	12	13	14	15		.12000E 02	.12000E 02	.26650E 03		.12000E 02	.00000E 00	.73000E 04
7	19	20	21	22		.95000E 01	.12000E 02	.26650E 03		.12000E 02	.00000E 00	.73000E 04
8	20	21	22	23		.12000E 02	.12000E 02	.26400E 03		.12000E 02	.00000E 00	.61500E 04
9	21	22	23	24		.12000E 02	.12000E 02	.26650E 03		.12000E 02	.00000E 00	.73000E 04
10	28	29	30	31		.95000E 01	.12000E 02	.26650E 03		.12000E 02	.00000E 00	.73000E 04
11	29	30	31	32		.12000E 02	.12000E 02	.26400E 03		.12000E 02	.00000E 00	.61500E 04
12	30	31	32	33		.12000E 02	.12000E 02	.26650E 03		.12000E 02	.00000E 00	.73000E 04
13	37	38	39	40		.95000E 01	.12000E 02	.26650E 03		.12000E 02	.00000E 00	.73000E 04
14	38	39	40	41		.12000E 02	.12000E 02	.26400E 03		.12000E 02	.00000E 00	.61500E 04
15	39	40	41	42		.12000E 02	.12000E 02	.26650E 03		.12000E 02	.00000E 00	.73000E 04
16	46	47	48	49		.95000E 01	.12000E 02	.26650E 03		.12000E 02	.00000E 00	.73000E 04
17	51	52	53	54		.12000E 02	.12000E 02	.26400E 03		.12000E 02	.00000E 00	.61500E 04
18	56	57	58	59		.95000E 01	.12000E 02	.26650E 03		.12000E 02	.00000E 00	.73000E 04
19	61	62	63	64		.12000E 02	.12000E 02	.26400E 03		.12000E 02	.00000E 00	.61500E 04
20	66	67	68	69		.95000E 01	.12000E 02	.26650E 03		.12000E 02	.00000E 00	.73000E 04

100 STORY SETBACK CONCRETE FRAME/MODAL ANALYSIS- CRACKED I, JOINT DIMS REAL/AM, 7R
WITHOUT P-DELTA EFFECTS.

** INPUT DATA **

NO.	STRUCTURAL COORDINATE							COLUMN PROPERTIES				AREA	SHEAR	MOMENT OF INERTIA	
	<	1	0	5	3	0	6	RIGID LOWER	CLEAR	LENGTH	RIGID UPPER				>
1	0	1	0	5	3	0	6	.00000E 00	.13070E 03		.13300E 02		.36100E 03	.00000E 00	.38900E 04
2	0	2	0	5	0	7		.00000E 00	.13070E 03		.13300E 02		.57600E 03	.00000E 00	.15550E 05
3	0	3	0	5	0	8		.00000E 00	.13070E 03		.13300E 02		.57600E 03	.00000E 00	.15550E 05
4	0	4	0	5	0	9		.00000E 00	.13070E 03		.13300E 02		.36100E 03	.00000E 00	.38900E 04
5	1	10	5	14	4	15		.13300E 02	.11790E 03		.12800E 02		.36100E 03	.00000E 00	.38900E 04
6	2	11	5	14	7	16		.13300E 02	.11790E 03		.12800E 02		.57600E 03	.00000E 00	.15550E 05
7	3	12	5	14	8	17		.13300E 02	.11790E 03		.12800E 02		.57600E 03	.00000E 00	.15550E 05
8	4	13	5	14	9	18		.13300E 02	.11790E 03		.12800E 02		.36100E 03	.00000E 00	.38900E 04
9	10	19	14	23	15	24		.12800E 02	.11840E 03		.12800E 02		.36100E 03	.00000E 00	.38900E 04
10	11	20	14	23	16	25		.12800E 02	.11840E 03		.12800E 02		.57600E 03	.00000E 00	.15550E 05
11	12	21	14	23	17	26		.12800E 02	.11840E 03		.12800E 02		.57600E 03	.00000E 00	.15550E 05
12	13	22	14	23	18	27		.12800E 02	.11840E 03		.12800E 02		.36100E 03	.00000E 00	.38900E 04
13	19	28	23	32	24	33		.12800E 02	.11840E 03		.12800E 02		.36100E 03	.00000E 00	.38900E 04
14	20	29	23	32	25	34		.12800E 02	.11840E 03		.12800E 02		.57600E 03	.00000E 00	.15550E 05
15	21	30	23	32	26	35		.12800E 02	.11840E 03		.12800E 02		.57600E 03	.00000E 00	.15550E 05
16	22	31	23	32	27	36		.12800E 02	.11840E 03		.12800E 02		.36100E 03	.00000E 00	.38900E 04
17	28	37	32	41	33	42		.12800E 02	.11900E 03		.12200E 02		.36100E 03	.00000E 00	.38900E 04
18	29	38	32	41	34	43		.12800E 02	.11900E 03		.12200E 02		.57600E 03	.00000E 00	.15550E 05
19	30	39	32	41	35	44		.12800E 02	.11900E 03		.12200E 02		.57600E 03	.00000E 00	.15550E 05
20	31	40	32	41	36	45		.12800E 02	.11900E 03		.12200E 02		.36100E 03	.00000E 00	.38900E 04
21	38	46	41	48	43	49		.12200E 02	.12080E 03		.11000E 02		.57600E 03	.00000E 00	.15550E 05
22	39	47	41	48	44	50		.12200E 02	.12080E 03		.11000E 02		.57600E 03	.00000E 00	.15550E 05
23	46	51	48	53	49	54		.11000E 02	.12200E 03		.11000E 02		.32400E 03	.00000E 00	.25200E 04
24	47	52	48	53	50	55		.11000E 02	.12200E 03		.11000E 02		.32400E 03	.00000E 00	.25200E 04
25	51	56	53	58	54	59		.12270E 03	.12270E 03		.10300E 02		.32400E 03	.00000E 00	.25200E 04

INPUT DATA **

NO.	< STRUCTURAL COORDINATE >			COLUMN PROPERTIES							AREA	SHEAR	>	MOMENT OF INERTIA
	<	X	>	<	RIGID LOWER	CLEAR SPAN	>	RIGID UPPER	<	FULL				
26	52	57	53	58	55	50	.12270E 03	.10300E 02	.32400E 03	.00000E 00	.25200E 04			
27	56	61	58	63	59	54	.12340E 03	.10300E 02	.32400E 03	.00000E 00	.25200E 04			
28	57	62	58	63	60	55	.12340E 03	.10300E 02	.32400E 03	.00000E 00	.25200E 04			
29	61	66	63	68	64	59	.12420E 03	.95000E 01	.32400E 03	.00000E 00	.25200E 04			
30	62	67	63	68	65	60	.12420E 03	.95000E 01	.32400E 03	.00000E 00	.25200E 04			

PAGE 6

10 STORY RETRACK CONCRETE FRAME/MODAL ANALYSIS- CRACKED I, JOINT DIMS REAL/AM, 78
WITHOUT P-DELTA EFFECTS.

** INPUT DATA **

STORY DATA		
NO.	COORD.	MASS
1	5	.47850E 00
2	14	.47850E 00
3	23	.47850E 00
4	32	.47850E 00
5	41	.43850E 00
6	48	.15950E 00
7	53	.15950E 00
8	58	.15950E 00
9	63	.15950E 00
10	68	.11930E 00

10 STORY SETBACK CONCRETE FRAME/MODAL ANALYSIS- CRACKED I,JOINT DIMS REAL/AM,78
WITHOUT P-DELTA EFFECTS.

** OUTPUT DATA **

RESULTS OF MODAL ANALYSIS.

MODE NO.	MODAL FREQUENCY	STORY 1	2	3	4	5	6	7	8
1	.35991F 01 .18739E 02	.10000E 01 .18739E 02	.25041E 01 .20070E 02	.40550E 01	.55555E 04	.70884E 01	.93848E 01	.13120E 02	.16428E 02
2	.70340F 01 .55157E 01	.10000E 01 .55157E 01	.23941E 01 .5192E 01	.35842E 01	.43109E 01	.43513E 01	.32818E 01	.44788E 00	.28324E 01
3	.193001E 02 .14992E 01	.10000E 01 .14992E 01	.20146E 01 .4209E 01	.21405E 01	.11409E 01	.74550E 00	.35276E 01	.59235E 01	.38035E 01
4	.196133F 02 .55289E 00	.10000E 01 .55289E 00	.16848E 01 .55713E 01	.10876E 01	.46426E 00	.17172E 01	.14137E 01	.10665E 01	.24255E 01
5	.427414F 02 .28031E 01	.10000E 01 .28031E 01	.11777E 01 .2377E 01	.10782E 00	.12857E 01	.53656E 00	.22382E 01	.29886E 01	.21278E 01

10 STORY RETRACK CONCRETE FRAME/MODAL ANALYSIS- CRACKED I, JOINT DIMS REAL/AM,78
WITHOUT P-DELTA EFFECTS.

** OUTPUT DATA **

RESULTS OF MODAL ANALYSIS.

MODE NO.	MODAL FREQUENCY	STORY 1	2	3	4	5	6	7	8
6	.3527185F 02	.10000E 01 .16823E 01	.49644E 00 -.11617E 01	.79813E 00	-.85824E 00	.82963E 00	.2123E 01	-.17527E 01	-.49399E 00
7	.374224F 02	.10000E 01 -.29558E 01	.2573E 00 .14911E 01	-.10325E 01	.84220E-01	.10027E 01	-.16059E 01	-.13015E 01	.31732E 01
8	.527277F 02	.10000E 01 .7134E 00	-.22943E 00 -.28744E 00	-.83608E 00	.87703E 00	.47135E-01	-.17553E 01	.16540E 01	-.12381E 01
9	.644444F 02	.10000E 01 -.20347E 00	-.74247E 00 .46345E-01	-.13120E 00	.91656E 00	-.11776E 01	.21194E 01	-.10203E 01	.48888E 00
10	.804224F 02	.10000E 01 .21515E-01	-.12846E 01 -.47948E-02	.11982E 01	-.86394E 00	.51714E 00	-.55078E 00	.18116E 00	-.65840E-01

STOP

EXAMPLE 2:

The frame shown on the coding form is required to be analysed for its response to the SCEL-5⁺ earthquake records. Joint dimensions and the P-Δ effects are to be considered. Mass related damping is used. Units are in kips and inches. The initial base velocity is selected as zero, the modulus of elasticity is 3605. The number of substeps is set at 10, and degrading behaviour is required for 1 second.

The input data is shown on pages 85 to 90 and the results of the analyses include the static loading element forces, rotations, floor sways and parts of the dynamic response history and the member and floor response envelopes.

⁺El Centro NS component, filtered to achieve a design spectrum @ 90% probability level of inexcurrence and @ 5% critical damping.

FORTRAN CODING FORM

1. The first part of the paper is devoted to the study of the asymptotic behavior of the solutions of the system (1) as $\epsilon \rightarrow 0$. It is shown that the solutions of the system (1) converge to the solutions of the system (2) as $\epsilon \rightarrow 0$.

PROGRAM PROGRAMMER	DATE	PUNCHING INSTRUCTIONS	GRAPHIC PUNCH	PAGE PAGE 3	ADDRESSES NUMBER 07
5900.	5900.	5900.	5900.	5900.	5900.
6217.	6217.	6217.	6217.	6217.	6217.
2996.	2996.	2996.	2996.	2996.	2996.
12500.	12500.	12500.	12500.	12500.	12500.
13239.	13239.	13239.	13239.	13239.	13239.
20	20	20	20	20	20
1993.	1993.	1993.	1993.	1993.	1993.
5200.	5200.	5200.	5200.	5200.	5200.

IBM

FORTRAN CODING FORM

FORM NO. 100-10000
Printed in U.S.A.

STATEMENT NUMBER	PROGRAM PROGRAMMER	DATE	PUNCHING INSTRUCTIONS		GRAPHIC PUNCH	PALL 5 CARD ELECTRIC NUMBER	ORIGINATOR JAN 1961	
			1	2				
1	11	10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29 30 31 32 33 34 35 36 37 38 39 40 41 42 43 44 45 46 47 48 49 50 51 52 53 54 55 56 57 58 59 60 61 62 63 64 65 66 67 68 69 70 71 72 73 74 75 76 77 78 79 80						
2	12							
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PAGE 1

*** QUAKE ***
PROGRAM FOR THE ELASTIC-INELASTIC SEISMIC RESPONSE
OF DEGRADING REINFORCED CONCRETE STRUCTURES.
A.L. MORRIS OTTAWA 1977.

GENERAL DATA

INPUT CONTROL:

KPS
IPS
SfC

MAXIMUM LINES
RESPONSE INTERVAL

0.50

ADDENDUM 1:

YOUNG, S.
SHEAR

• 3605E 04 •
• 0000E 00 •

MASS
STIFFNESS

• 55.99E 00
• 00000E 00

NUMBER OF:

BEAMS
LOADED BEAMS
COLUMNS
STORIES
DEGREES OF F

15 15 20 5 45

EARTHQUAKE:

DURATION	.1000E 01
SCALING FACTOR	.1000E 00
RECORD INTERVAL	.1000E-01

NEWARK'S BETA
TIME STEP
SUR. TIME STEP
INITL BASE VEL

• 10000E-01
• 10000E-02
• 00000E 00

SPRING PARAMETERS:

```

DEFORMATION SP115N      0
UNLOADING FACT    .10000E 01
RELOADING FACT    .00000E 00
HINGE PRINT SP115N      1

```

MISCELLANEOUS:

GRAVITY CONST.	.38640E 03
P-DELTA FACTOR	.00000E 00
MAX 1/2 BANDWIDTH	10
LOADING OPTION	0
MODAL INFO OPTION	0

5 STORY UNIFORM CONCRETE FRAME/SEISMIC ANALYSIS-UNCRAKED 1 JOINT DIMS REAL/AM,78
EARTH SHAKE RECORD: SCAL-5 (MASS CONSISTS OF DEAD LOAD ONLY)

** INPUT DATA **

BEAM PROPERTIES

No.	STRUCTURAL COORDS.		<		LENGTH		>		SHEAR AREA	MOMENT OF INERTIA
			RIGID	LEFT	CLEAR	SPAN	RIGID	RIGHT		
1	1	2	1	1	26650E	03	1	1	00000E	00
2	2	3	1	1	26650E	03	1	1	00000E	00
3	3	4	1	1	26650E	03	1	1	00000E	00
4	10	11	1	1	26650E	03	1	1	00000E	00
5	11	12	1	1	26650E	03	1	1	00000E	00
6	12	13	1	1	26650E	03	1	1	00000E	00
7	19	20	1	1	26650E	03	1	1	00000E	00
8	20	21	1	1	26650E	03	1	1	00000E	00
9	21	22	1	1	26650E	03	1	1	00000E	00
10	28	29	1	1	26650E	03	1	1	00000E	00
11	29	30	1	1	26650E	03	1	1	00000E	00
12	30	31	1	1	26650E	03	1	1	00000E	00
13	37	38	1	1	26650E	03	1	1	00000E	00
14	38	39	1	1	26650E	03	1	1	00000E	00
15	39	40	1	1	26650E	03	1	1	00000E	00

PAGE

5 STORY UNIFORM CONCRETE FRAME/SEISMIC ANALYSIS-UNCRACKED 1, JOINT DIMS REAL/AM, 78
EARTH SHAKE RECORD: SCEL-5 (MASS CONSISTS OF DEAD LOAD ONLY)

** INPUT DATA **

BEAM SPRING PROPERTIES

BN No.	MOMENTS CRACKING YIELDING ULTIMATE				ROTATIONS- CRACKING YIELDING ULTIMATE			
	←	→	←	→	←	→	←	→
	←	→	←	→	←	→	←	→
1	.76500E 03 .45000E 04 .45060E 04 .46150E 04	.76500E 03 .45040E 04 .45060E 04 .46150E 04	.76500E 03 .45040E 04 .45060E 04 .46150E 04	.76500E 03 .45040E 04 .45060E 04 .46150E 04	.13400E-03 .12200E-02 .13400E-03 .12200E-02	.13400E-03 .12200E-02 .13400E-03 .12200E-02	.13400E-03 .12200E-02 .13400E-03 .12200E-02	.13400E-03 .12200E-02 .13400E-03 .12200E-02
2	.70800E 03 .42870E 04 .43750E 04	.70800E 03 .42870E 04 .43750E 04	.70800E 03 .42870E 04 .43750E 04	.70800E 03 .42870E 04 .43750E 04	.14100E-03 .12200E-02 .14100E-03 .12200E-02	.14100E-03 .12200E-02 .14100E-03 .12200E-02	.14100E-03 .12200E-02 .14100E-03 .12200E-02	.14100E-03 .12200E-02 .14100E-03 .12200E-02
3	.76500E 03 .45060E 04 .46150E 04	.76500E 03 .45040E 04 .45060E 04 .46150E 04	.76500E 03 .45040E 04 .45060E 04 .46150E 04	.76500E 03 .45040E 04 .45060E 04 .46150E 04	.13400E-03 .12200E-02 .13400E-03 .12200E-02	.13400E-03 .12200E-02 .13400E-03 .12200E-02	.13400E-03 .12200E-02 .13400E-03 .12200E-02	.13400E-03 .12200E-02 .13400E-03 .12200E-02
4	.76500E 03 .45060E 04 .46150E 04	.76500E 03 .45040E 04 .45060E 04 .46150E 04	.76500E 03 .45040E 04 .45060E 04 .46150E 04	.76500E 03 .45040E 04 .45060E 04 .46150E 04	.13400E-03 .12200E-02 .13400E-03 .12200E-02	.13400E-03 .12200E-02 .13400E-03 .12200E-02	.13400E-03 .12200E-02 .13400E-03 .12200E-02	.13400E-03 .12200E-02 .13400E-03 .12200E-02
5	.70800E 03 .42870E 04 .43750E 04	.70800E 03 .42870E 04 .43750E 04	.70800E 03 .42870E 04 .43750E 04	.70800E 03 .42870E 04 .43750E 04	.14100E-03 .12200E-02 .14100E-03 .12200E-02	.14100E-03 .12200E-02 .14100E-03 .12200E-02	.14100E-03 .12200E-02 .14100E-03 .12200E-02	.14100E-03 .12200E-02 .14100E-03 .12200E-02
6	.76500E 03 .45060E 04 .46150E 04	.76500E 03 .45040E 04 .45060E 04 .46150E 04	.76500E 03 .45040E 04 .45060E 04 .46150E 04	.76500E 03 .45040E 04 .45060E 04 .46150E 04	.13400E-03 .12200E-02 .13400E-03 .12200E-02	.13400E-03 .12200E-02 .13400E-03 .12200E-02	.13400E-03 .12200E-02 .13400E-03 .12200E-02	.13400E-03 .12200E-02 .13400E-03 .12200E-02

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** INPUT DATA **

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BEAM SPRING PROPERTIES

RM NO.	MOMENTS CRACKING YIELDING ULTIMATE			-ROTATIONS- CRACKING YIELDING ULTIMATE		
	+LEFT	-LEFT	+RIGHT	+LEFT	-LEFT	+RIGHT
7	.76500E-03 .45060E-04 .46150E-04	.76500E-03 .45060E-04 .46150E-04	.76500E-03 .45060E-04 .46150E-04	.13400E-03 .12200E-02	.13400E-03 .12200E-02	.13400E-03 .12200E-02
8	.70800E-03 .42870E-04 .43750E-04	.70800E-03 .42870E-04 .43750E-04	.70800E-03 .42870E-04 .43750E-04	.14100E-03 .12200E-02	.14100E-03 .12200E-02	.14100E-03 .12200E-02
9	.76500E-03 .45060E-04 .46150E-04	.76500E-03 .45060E-04 .46150E-04	.76500E-03 .45060E-04 .46150E-04	.13400E-03 .12200E-02	.13400E-03 .12200E-02	.13400E-03 .12200E-02
10	.76500E-03 .45060E-04 .46150E-04	.76500E-03 .45060E-04 .46150E-04	.76500E-03 .45060E-04 .46150E-04	.13400E-03 .12200E-02	.13400E-03 .12200E-02	.13400E-03 .12200E-02
11	.70800E-03 .42870E-04 .43750E-04	.70800E-03 .42870E-04 .43750E-04	.70800E-03 .42870E-04 .43750E-04	.14100E-03 .12200E-02	.14100E-03 .12200E-02	.14100E-03 .12200E-02
12	.76500E-03 .45060E-04 .46150E-04	.76500E-03 .45060E-04 .46150E-04	.76500E-03 .45060E-04 .46150E-04	.13400E-03 .12200E-02	.13400E-03 .12200E-02	.13400E-03 .12200E-02

5 STORY UNIFORM CONCRETE FRAME/SEISMIC ANALYSIS-UNCRACKED I, JOINT DIMS REAL/AM, 7R
 EARTH QUAKE RECORD: SCEL-5 IMASS CONSISTS OF DEAD LOAD ONLY

** INPUT DATA **

DEAD SPRING PROPERTIES

BH NO.	MOMENTS			ROTATIONS		
	CRACKING	YIELDING	ULTIMATE	CRACKING	YIELDING	ULTIMATE
	-LEFT	+RIGHT	-RIGHT	+LEFT	+RIGHT	-RIGHT
13	.57600E 03 .32480E 04 .33250E 04	.57600E 03 .72480E 04 .33750E 04	.57600E 03 .32480E 04 .33250E 04	.15400E-03 .13300E-02	.15400E-03 .13300E-02	.15400E-03 .13300E-02
14	.52000E 03 .29880E 04 .30580E 04	.52000E 03 .29880E 04 .30580E 04	.52000E 03 .29880E 04 .30580E 04	.16600E-03 .12800E-02	.16600E-03 .12800E-02	.16600E-03 .12800E-02
15	.57600E 03 .32480E 04 .33250E 04	.57600E 03 .72480E 04 .33750E 04	.57600E 03 .32480E 04 .33250E 04	.15400E-03 .13300E-02	.15400E-03 .13300E-02	.15400E-03 .13300E-02

5 STORY UNIFORM CONCRETE FRAME/SEISMIC ANALYSIS-UNCRACKED, 1 JOINT DIMS REAL/4M, 78
EARTH SHAKE RECORD: SCAL-5 (MASS CONSISTS OF DEAD LOAD ONLY)

** INPUT DATA **

COLUMN SPRING PROPERTIES

COL NO.	MOMENTS CRACKING & YIELDING ULTIMATE				ROTATIONS CRACKING YIELDING ULTIMATE			
	LOWER	UPPER	LOWER	UPPER	LOWER	UPPER	LOWER	UPPER
1	.12760E-04 .59000E-04 .62170E-04	.12760E-04 .59000E-04 .62170E-04	.12760E-04 .59000E-04 .62170E-04	.12760E-04 .59000E-04 .62170E-04	.25000E-03 .65200E-03 .65200E-03	.25000E-03 .65200E-03 .65200E-03	.25000E-03 .65200E-03 .65200E-03	.25000E-03 .65200E-03 .65200E-03
2	.29960E-04 .12500E-05 .13239E-05	.29960E-04 .12500E-05 .13239E-05	.29960E-04 .12500E-05 .13239E-05	.29960E-04 .12500E-05 .13239E-05	.20000E-03 .80000E-03 .80000E-03	.20000E-03 .80000E-03 .80000E-03	.20000E-03 .80000E-03 .80000E-03	.20000E-03 .80000E-03 .80000E-03
3	.29960E-04 .12500E-05 .13239E-05	.29960E-04 .12500E-05 .13239E-05	.29960E-04 .12500E-05 .13239E-05	.29960E-04 .12500E-05 .13239E-05	.20000E-03 .80000E-03 .80000E-03	.20000E-03 .80000E-03 .80000E-03	.20000E-03 .80000E-03 .80000E-03	.20000E-03 .80000E-03 .80000E-03
4	.12760E-04 .59000E-04 .62170E-04	.12760E-04 .59000E-04 .62170E-04	.12760E-04 .59000E-04 .62170E-04	.12760E-04 .59000E-04 .62170E-04	.25000E-03 .65200E-03 .65200E-03	.25000E-03 .65200E-03 .65200E-03	.25000E-03 .65200E-03 .65200E-03	.25000E-03 .65200E-03 .65200E-03
5	.11570E-04 .57000E-04 .60640E-04	.11570E-04 .57000E-04 .60640E-04	.11570E-04 .57000E-04 .60640E-04	.11570E-04 .57000E-04 .60640E-04	.25000E-03 .70000E-03 .70000E-03	.25000E-03 .70000E-03 .70000E-03	.25000E-03 .70000E-03 .70000E-03	.25000E-03 .70000E-03 .70000E-03
6	.26690E-04 .12000E-05 .12964E-05	.26690E-04 .12000E-05 .12964E-05	.26690E-04 .12000E-05 .12964E-05	.26690E-04 .12000E-05 .12964E-05	.20000E-03 .51300E-03 .51300E-03	.20000E-03 .51300E-03 .51300E-03	.20000E-03 .51300E-03 .51300E-03	.20000E-03 .51300E-03 .51300E-03

5 STORY UNIFORM CONCRETE FRAME/SEISC ANALYSIS-UNCRACKED J, JOINT DIMS REAL/AM, 78
EARTH QUAKE RECORD: SCEL-5 (MASS CONSISTS OF DEAD LOAD ONLY)

** INPUT DATA **

COLUMN SPRING PROPERTIES

COL NO.	MOMENTS CRACKING YIELDING			ROTATIONS: CRACKING YIELDING		
	ULTIMATE			ULTIMATE		
	LOWER	UPPER	UPPER	LOWER	UPPER	UPPER
7	.26690E 04 .12000E 05 .12964E 05	.26690E 04 .12000E 05 .12964E 05	.26690E 04 .12000E 05 .12964E 05	.20000E-03 .51300E-03 .51300E-03	.20000E-03 .51300E-03 .51300E-03	.20000E-03 .51300E-03 .51300E-03
8	.11570E 04 .57000E 04 .60640E 04	.11570E 04 .57000E 04 .60640E 04	.11570E 04 .57000E 04 .60640E 04	.25000E-03 .70000E-03 .70000E-03	.25000E-03 .70000E-03 .70000E-03	.25000E-03 .70000E-03 .70000E-03
9	.10360E 04 .55000E 04 .59040E 04	.10360E 04 .55000E 04 .59040E 04	.10360E 04 .55000E 04 .59040E 04	.24000E-03 .75200E-03 .75200E-03	.24000E-03 .75200E-03 .75200E-03	.24000E-03 .75200E-03 .75200E-03
10	.23460E 04 .11750E 05 .12575E 05	.23460E 04 .11750E 05 .12575E 05	.23460E 04 .11750E 05 .12575E 05	.19000E-03 .56300E-03 .56300E-03	.19000E-03 .56300E-03 .56300E-03	.19000E-03 .56300E-03 .56300E-03
11	.23460E 04 .11750E 05 .12575E 05	.23460E 04 .11750E 05 .12575E 05	.23460E 04 .11750E 05 .12575E 05	.19000E-03 .56300E-03 .56300E-03	.19000E-03 .56300E-03 .56300E-03	.19000E-03 .56300E-03 .56300E-03
12	.10360E 04 .55000E 04 .59040E 04	.10360E 04 .55000E 04 .59040E 04	.10360E 04 .55000E 04 .59040E 04	.24000E-03 .75200E-03 .75200E-03	.24000E-03 .75200E-03 .75200E-03	.24000E-03 .75200E-03 .75200E-03

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5 STORY UNIFORM CONCRETE FRAME/SEISMIC ANALYSIS-UNCRAKED I, JOINT DIMS REAL/AM, JR
 EARTH QUAKE RECORD: SCAL-5 (MASS CONSISTS OF DEAD LOAD ONLY)

** INPUT DATA **

COLUMN SPRING PROPERTIES

COL NR.	<	>	MOMENTS		ROTATIONS	
			CRACKING YIELDING ULTIMATE	CRACKING YIELDING ULTIMATE	CRACKING YIELDING ULTIMATE	CRACKING YIELDING ULTIMATE
	LOWER	UPPER	LOWER	UPPER	LOWER	UPPER
13	.91600E 03 .52000E 04 .57350E 04	.91600E 03 .52000E 04 .57350E 04	.91600E 03 .52000E 04 .57350E 04	.91600E 03 .52000E 04 .57350E 04	.23000E-03 .81000E-03 .81000E-03	.23000E-03 .81000E-03 .81000E-03
14	.20220E 04 .11500E 05 .12130E 05	.20220E 04 .11500E 05 .12130E 05	.20220E 04 .11500E 05 .12130E 05	.20220E 04 .11500E 05 .12130E 05	.19000E-03 .62000E-03 .62000E-03	.19000E-03 .62000E-03 .62000E-03
15	.20220E 04 .11500E 05 .12130E 05	.20220E 04 .11500E 05 .12130E 05	.20220E 04 .11500E 05 .12130E 05	.20220E 04 .11500E 05 .12130E 05	.19000E-03 .62000E-03 .62000E-03	.19000E-03 .62000E-03 .62000E-03
16	.91600E 03 .52000E 04 .57350E 04	.91600E 03 .52000E 04 .57350E 04	.91600E 03 .52000E 04 .57350E 04	.91600E 03 .52000E 04 .57350E 04	.23000E-03 .81000E-03 .81000E-03	.23000E-03 .81000E-03 .81000E-03
17	.79300E 03 .52000E 04 .55400E 04	.79300E 03 .52000E 04 .55400E 04	.79300E 03 .52000E 04 .55400E 04	.79300E 03 .52000E 04 .55400E 04	.23000E-03 .85300E-03 .85300E-03	.23000E-03 .85300E-03 .85300E-03
18	.17000E 04 .11000E 05 .11661E 05	.17000E 04 .11000E 05 .11661E 05	.17000E 04 .11000E 05 .11661E 05	.17000E 04 .11000E 05 .11661E 05	.19000E-03 .68300E-03 .68300E-03	.19000E-03 .68300E-03 .68300E-03

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5 STORY UNIFORM CONCRETE FRAME/SEISMIC ANALYSIS-UNCRACKED I, JOINT DIMS REAL/AM, 78
 PART 5 QUAKE RECORD: SEEL-5 (MASS CONSISTS OF DEAD LOAD ONLY)

** INPUT DATA **

COLUMN SPRING PROPERTIES

COL NR.	<	MOMENTS			>	<	ROTATIONS			>
		-LOWER	+UPPER	-UPPER			-LOWER	+LOWER	-UPPER	
19		.17000E 04	.17000E 04	.17000E 04			.19000E-03	.19000E-03	.19000E-03	
		.11000E 05	.11000E 05	.11000E 05			.68300E-03	.68300E-03	.68300E-03	
		.11661E 05	.11661E 05	.11661E 05						
20		.79300E 03	.79300E 03	.79300E 03			.23000E-03	.23000E-03	.23000E-03	
		.52000E 04	.52000E 04	.52000E 04			.85300E-03	.85300E-03	.85300E-03	
		.55400E 04	.55400E 04	.55400E 04						

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5 STORY UNIFORM CONCRETE FRAME/SEISMIC ANALYSIS-UNPACKED 1, JOINT DIMS REAL/AM, 78
EARTH QUAKE RECORD: SCAL-5 (MASS CONSISTS OF DEAD LOAD ONLY)

** INPUT DATA **

GRAVITY LOADS

BN NO.	CONC LOADS	UOL	LOCATION FROM LEFT.	MAGNITUDE.	LOCATION FROM LEFT.	MAGNITUDE.	LOCATION FROM LEFT.	MAGNITUDE
1	0	.28600E 00						
2	0	.28600E 00						
3	0	.28600E 00						
4	0	.28600E 00						
5	0	.28600E 00						
6	0	.28600E 00						
7	0	.28600E 00						
8	0	.28600E 00						
9	0	.28600E 00						
10	0	.28600E 00						
11	0	.28600E 00						
12	0	.28600E 00						
13	0	.23200E 00						
14	0	.23200E 00						
15	0	.23200E 00						

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5 STORY UNIFORM CONCRETE FRAME/SEISMIC ANALYSIS-UNCRAKED I,JOINT DIMS REAL/AM,78
EARTH QUAKE RECORD: SCEL=5 (MASS CONSISTS OF DEAD LOAD ONLY)

** INPUT DATA **

STORY DATA		
NO.	COORD	MASS
1	5	.47850E 00
2	14	.47850E 00
3	23	.47850E 00
4	32	.47850E 00
5	41	.35780E 00

5 STORY UNIFORM CONCRETE FRAME/SEISMIC ANALYSIS-UNCRACKED 1, JOINT DIMS REAL/AM, 78
EARTH SHAKE RECORD: SCAL-5 (MASS CONSISTS OF DEAD LOAD ONLY)

** OUTPUT DATA **

RESULTS OF STATIC ANALYSIS.

RM NO.	BEAM RESPONSE			
	ELEMENTS (INS-KPS)		ROTATIONS (RADIAN)	
	LEFT	RIGHT	LEFT	RIGHT
1	-.1019E-04	.1223E 04	.32136E-03	-.24623E-04
2	-.11281E 04	.11281E 04	-.17162E-04	.17162E-04
3	-.12231E 04	.10198E 04	.24623E-04	-.32136E-03
4	-.10770E 04	.11238E 04	.19692E-03	-.26409E-04
5	-.11248E 04	.11248E 04	-.15323E-05	.15323E-05
6	-.11938E 04	.10770E 04	.26409E-04	-.19692E-03
7	-.10619E 04	.11946E 04	.23979E-03	-.46071E-04
8	-.11271E 04	.11270E 04	-.11955E-04	.11955E-04
9	-.11946E 04	.10619E 04	.46071E-04	-.23979E-03
10	-.11031E 04	.11771E 04	.14525E-03	-.37129E-04
11	-.11232E 04	.11232E 04	.60324E-05	-.60324E-05
12	-.11771E 04	.11031E 04	.37129E-04	-.14525E-03
13	-.77241E 03	.94356E 03	.47035E-03	-.68759E-04
14	-.85395E 03	.85395E 03	-.31798E-04	.31798E-04
15	-.94356E 03	.77261E 03	.68759E-04	-.47035E-03

5 STORY UNIFORM CONCRETE FRAME/SEISMIC ANALYSIS-UNCRACKED 1,JOINT DIMS REAL/AM,7R
EARTH QUAKE RECORD: SEEL-5 INASS CONSISTS OF DEAD LOAD ONLY

*** OUTPUT DATA ***

RESULTS OF STATIC ANALYSIS.

COL NR.	MOMENTS (INR-KPS)		ROTATIONS (RADIANS)		SWAY (INS)		AXIAL LOAD (KPS)
	LOWER	UPPER	LOWER	UPPER	LOWER	UPPER	
1	.32059E 03	.47150E 03	.30437E-04	.35926E-03	.97917E-18	.97917E-18	.17712E 03
2	-.43478E 02	-.77507E 02	-.15886E-05	-.18751E-04	.97917E-18	.97917E-18	.37121E 03
3	.43478E 02	.77507E 02	.15886E-05	.18751E-04	.97917E-18	.97917E-18	.37121E 03
4	-.32059E 03	-.47150E 03	-.30437E-04	-.35926E-03	.97917E-18	.97917E-18	.17712E 03
5	.32059E 03	.47150E 03	.30437E-04	.35926E-03	.97917E-18	.97917E-18	.17712E 03
6	-.32059E 03	-.47150E 03	-.30437E-04	-.35926E-03	.97917E-18	.97917E-18	.17712E 03
7	.32059E 03	.47150E 03	.30437E-04	.35926E-03	.97917E-18	.97917E-18	.17712E 03
8	-.32059E 03	-.47150E 03	-.30437E-04	-.35926E-03	.97917E-18	.97917E-18	.17712E 03
9	.32059E 03	.47150E 03	.30437E-04	.35926E-03	.97917E-18	.97917E-18	.17712E 03
10	-.32059E 03	-.47150E 03	-.30437E-04	-.35926E-03	.97917E-18	.97917E-18	.17712E 03
11	.32059E 03	.47150E 03	.30437E-04	.35926E-03	.97917E-18	.97917E-18	.17712E 03
12	-.32059E 03	-.47150E 03	-.30437E-04	-.35926E-03	.97917E-18	.97917E-18	.17712E 03
13	.32059E 03	.47150E 03	.30437E-04	.35926E-03	.97917E-18	.97917E-18	.17712E 03
14	-.32059E 03	-.47150E 03	-.30437E-04	-.35926E-03	.97917E-18	.97917E-18	.17712E 03
15	.32059E 03	.47150E 03	.30437E-04	.35926E-03	.97917E-18	.97917E-18	.17712E 03
16	-.32059E 03	-.47150E 03	-.30437E-04	-.35926E-03	.97917E-18	.97917E-18	.17712E 03
17	.32059E 03	.47150E 03	.30437E-04	.35926E-03	.97917E-18	.97917E-18	.17712E 03
18	-.32059E 03	-.47150E 03	-.30437E-04	-.35926E-03	.97917E-18	.97917E-18	.17712E 03
19	.32059E 03	.47150E 03	.30437E-04	.35926E-03	.97917E-18	.97917E-18	.17712E 03
20	-.32059E 03	-.47150E 03	-.30437E-04	-.35926E-03	.97917E-18	.97917E-18	.17712E 03

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5 STORY UNIFORM CONCRETE FRAME/SEISC ANALYSIS-UNCRACKED I, JOINT DIMS REAL/AM, 7R,
EARTH QUAKE RECORD; SCAL=5, MASS CONSISTS OF DEAD LOAD ONLY)

** OUTPUT DATA **

RESULTS OF DYNAMIC ANALYSIS.

STORY NO.	FORCES (KPS)		SWAY (INS)		VELOCITY (INS/SEC)		ACCELN (INS/SEC/SEC)		TIME (SEC)
	INERTIAL	SHEAR	ABSOLUTE	RELATIVE	FLOOR	BASE	FLOOR	BASE	
1	.65294E 00	.48603E 00	-.50052E-02	-.50052E-02	-.54468E 00	-.14574E 02	-.14574E 02	.13434E 02	.500
2	.78962E 01	-.69349E-02	-.20436E-01	-.15431E-01	-.69403E 00	-.29544E 02	-.29544E 02		
3	-.18726E 01	-.79031E 01	-.48951E-01	-.28515E-01	-.12187E 00	-.17274E 02	-.17274E 02		
4	-.58535E 01	-.57757E 01	-.80295E-01	-.31345E-01	-.42002E 00	-.20596E 01	-.20596E 01		
5	-.42222E 01	-.42222E 01	-.10180E 00	-.23507E-01	.51635E 00	-.19191E 01	-.19191E 01		
1	-.20944E 02	-.37514E 02	-.49452E-01	-.49452E-01	-.50418E 00	.28563E 02	.28563E 02	.15489E 02	1.000
2	-.13475E 02	-.16572E 02	-.99466E-01	-.50114E-01	-.65819E 00	.17221E 02	.17221E 02		
3	.26485E 01	-.10964E 01	-.12411E 00	-.25548E-01	-.29510E 00	-.20859E 02	-.20859E 02		
4	.44179E 01	-.37457E 01	-.14439E 00	-.20278E-01	.32236E-02	-.24723E 02	-.24723E 02		
5	-.44163E 01	-.41634E 01	-.16442E 00	-.24029E-01	-.14218E 00	.74069E 01	.74069E 01		

5 STORY UNIFORM CONCRETE FRAME/SEISMIC ANALYSIS-UNCRAKED, 1 JOINT DIMS REAL/AM, 78
EARTH QUAKE RECORD: SCAL-5 (MASS CONSISTS OF DEAD LOAD ONLY)

** OUTPUT DATA **

RESULTS OF DYNAMIC ANALYSIS.

BEAM RESPONSE ENVELOPES

BM NO.	DUCTILITY RATIOS		MOMENTS (LBS-KIPS)		RIGHT	
	MAXIMUM	MINIMUM	MAXIMUM	MINIMUM	MAXIMUM	MINIMUM
1	.000	-.226	.000	-.000	.00000E 00	.00000E 00
2	.000	-.270	.000	-.000	.00000E 00	.00000E 00
3	.000	-.277	.000	-.000	.00000E 00	.00000E 00
4	.000	-.232	.000	-.000	.00000E 00	.00000E 00
5	.000	-.262	.000	-.000	.00000E 00	.00000E 00
6	.000	-.262	.000	-.000	.00000E 00	.00000E 00
7	.000	-.235	.000	-.000	.00000E 00	.00000E 00
8	.000	-.268	.000	-.000	.00000E 00	.00000E 00
9	.000	-.268	.000	-.000	.00000E 00	.00000E 00
10	.000	-.224	.000	-.000	.00000E 00	.00000E 00
11	.000	-.249	.000	-.000	.00000E 00	.00000E 00
12	.000	-.245	.000	-.000	.00000E 00	.00000E 00
13	.000	-.194	.000	-.000	.00000E 00	.00000E 00
14	.000	-.256	.000	-.000	.00000E 00	.00000E 00
15	.000	-.262	.000	-.000	.00000E 00	.00000E 00

5 STORY UNIFORM CONCRETE FRAME/SEISMIC ANALYSIS-UNCRACKED 1-JOINT DIMS REAL/AM, 78
EARTH SHAKE RECORD: SCEL-5 (MASS CONSISTS OF DEAD LOAD ONLY)

** OUTPUT DATA **

RESULTS OF DYNAMIC ANALYSIS.

COLUMN RESPONSE ENVELOPES

COL NO.	DUCTILITY RATIOS				MOMENTS (INS-KIPS)				AXIAL LOAD (KIPS)			
	MAXIMUM	MINIMUM	UPPER	LOWER	MAXIMUM	MINIMUM	UPPER	LOWER	MAXIMUM	MINIMUM	UPPER	LOWER
1	.045	.000	.062	.000	.8107E 03	.0000E 00	.7723E 03	.0000E 00	.1874E 03	.0000E 00	.1723E 03	.0000E 00
2	.047	.024	.016	.015	.1210E 04	.6216E 03	.4112E 03	.3905E 03	.3716E 03	.3708E 03	.3708E 03	.3708E 03
3	.050	.021	.022	.009	.1294E 04	.5365E 03	.5612E 03	.5612E 03	.3718E 03	.3710E 03	.3710E 03	.3710E 03
4	.014	.044	.000	.056	.1393E 03	.5446E 03	.0000E 00	.0000E 00	.1816E 03	.1664E 03	.1664E 03	.1664E 03
5	.044	.000	.079	.000	.1072E 04	.0000E 00	.9945E 03	.0000E 00	.1428E 03	.1354E 03	.1354E 03	.1354E 03
6	.014	.013	.017	.013	.4057E 03	.3379E 03	.4454E 03	.3354E 03	.2936E 03	.2932E 03	.2932E 03	.2932E 03
7	.022	.006	.021	.009	.5824E 03	.1611E 03	.5549E 03	.2271E 03	.2940E 03	.2939E 03	.2939E 03	.2939E 03
8	.000	.078	.000	.071	.3040E 00	.9746E 03	.0000E 00	.8903E 03	.1454E 03	.1321E 03	.1321E 03	.1321E 03
9	.078	.000	.041	.000	.3364E 03	.0000E 00	.9793E 03	.0000E 00	.1056E 03	.1001E 03	.1001E 03	.1001E 03
10	.018	.021	.019	.016	.4442E 03	.5265E 03	.4692E 03	.3989E 03	.2169E 03	.2169E 03	.2169E 03	.2169E 03
11	.021	.018	.024	.011	.5234E 03	.4480E 03	.5801E 03	.2741E 03	.2170E 03	.2167E 03	.2167E 03	.2167E 03
12	.000	.077	.000	.074	.0000E 00	.3225E 03	.0000E 00	.8912E 03	.1015E 03	.0976E 03	.0976E 03	.0976E 03
13	.077	.000	.077	.000	.8844E 03	.0000E 00	.8852E 03	.0000E 00	.7375E 03	.6430E 03	.6430E 03	.6430E 03
14	.013	.003	.019	.016	.3252E 03	.3440E 03	.4750E 03	.3887E 03	.1392E 03	.1395E 03	.1395E 03	.1395E 03
15	.014	.011	.019	.016	.4034E 03	.2633E 03	.4764E 03	.3859E 03	.1399E 03	.1395E 03	.1395E 03	.1395E 03
16	.000	.074	.000	.073	.0000E 00	.5536E 03	.0000E 00	.8423E 03	.6901E 03	.6289E 03	.6289E 03	.6289E 03
17	.100	.000	.128	.000	.9418E 03	.0000E 00	.1071E 04	.0000E 00	.3001E 02	.2880E 02	.2880E 02	.2880E 02
18	.013	.015	.008	.015	.3115E 03	.3641E 03	.2074E 03	.3619E 03	.6328E 02	.6311E 02	.6311E 02	.6311E 02

5 STORY UNIFORM CONCRETE FRAME/SEISC ANALYSIS-UNCRAKED 1 JOINT DIMS REAL/AM,78
EARTH SHAKE RECORD: SCEL-5 (MASS CONSISTS OF DEAD LOAD ONLY)

** OUTPUT DATA **

RESULTS OF DYNAMIC ANALYSIS.

COLUMN RESPONSE ENVELOPES

COL NO.	DUCTILITY RATIOS				MOMENTS (INS-KPS)				AXIAL LOAD (KPS)			
	MAXIMUM	MINIMUM	MAXIMUM	UPPER	LOWER	MINIMUM	MAXIMUM	UPPER	MINIMUM	MAXIMUM	MINIMUM	(TENSION--)

19	.017	-.010	.020	-.003	.42789E 03	-.24925E 03	.48999E 03	-.79487E 02	.63314E 02	.29764E 02	.63114E 02	
20	.000	-.095	.000	-.116	.00000E 00	-.31752E 03	.00000E 00	-.10176E 04	.29764E 02	.28543E 02		

5 STORY UNIFORM CONCRETE FRAME/SEISMIC ANALYSIS-UNCRAKED 1 JOINT DIMS REAL/AM,7M
EARTH QUAKE RECORD: SCAL-5 (MASS CONSISTS OF DEAD LOAD ONLY)

** OUTPT DATA **

RESULTS OF DYNAMIC ANALYSIS:

STORY NO.	SWAY (INS)		RELATIVE SWAY (INS)		SHEAR (KPS)		MAX. VEL (INS/SEC)		MAX. ACCLN (GINS/SEC/SEC)	
	MAXIMUM	MINIMUM	MAXIMUM	MINIMUM	MAXIMUM	MINIMUM	MAXIMUM	MINIMUM	MAXIMUM	MINIMUM
1	.20363E-01	-.49452E-01	.20343E-01	-.49452E-01	.19296E 02	-.37516E 02	.58453E 00	-.37516E 02	.30866E 02	-.37516E 02
2	.93360E-01	-.99566E-01	.17485E-01	-.50691E-01	.12132E 02	-.17153E 02	.11072E 01	.11072E 01	.36377E 02	.11072E 01
3	.79107E-01	-.17482E 00	.29179E-01	-.63259E-01	.18906E 02	-.23979E 02	.16707E 01	.16707E 01	.42107E 02	.16707E 01
4	.56607E-01	-.19001E 00	.26145E-01	-.55806E-01	.13456E 02	-.17790E 02	.11866E 01	.11866E 01	.36028E 02	.11866E 01
5	.56276E-01	-.23238E 00	.23876E-01	-.47846E-01	.12741E 02	-.17566E 02	.13287E 01	.13287E 01	.48696E 02	.13287E 01

STRUCTURE OVERTURNING MOMENTS (INS-KPS)

	MAXIMUM	MINIMUM
STORY 0	.46877E 04	-.10672E 05

EXAMPLE 3:

The moment-axial load-curvature relationships for two beams and one column are required. The input data is given on page 112 and output is given on pages 113 to 116.

104000

DEMONSTRATION APPLICATION OF THE PROGRAM TO PRODUCE SPRING PRIMARY CURVES A.M.78

INPUT DATA: 1.24 .88 1.58 2.37 2.25 1.75 2.0 2.37512. 26.5

*CRACKING MOMENT, YBOTTOM, -CRACKING MOMENT, YTOP, GROSS MOMENT OF INERTIA
844.2 12.90 800.8 13.60 22976.0

YIELD MOMENT, YIELD CURVATURE, KD, MAX CONCRTE STRESS
4683.2 .12597E-03 7.703 3.498

ULTIMATE MOMENT, ULTIMATE CURVATURE
4989.6 .89539E-03

YIELD MOMENT, YIELD CURVATURE, KD, MAX CONCRTE STRESS
2620.9 .11082E-03 5.581 2.230

ULTIMATE MOMENT, ULTIMATE CURVATURE
2888.3 .89539E-03

INPUT DATA: 2.0 1.58 1.80 2.5 2.0 2.37512. 22.5

*CRACKING MOMENT, YBOTTOM, -CRACKING MOMENT, YTOP, GROSS MOMENT OF INERTIA
529.7 11.04 377.4 11.46 13963.8

YIELD MOMENT, YIELD CURVATURE, KD, MAX CONCRTE STRESS
3227.5 .15019E-03 6.348 3.437

ULTIMATE MOMENT, ULTIMATE CURVATURE
3504.0 .10840E-02

YIELD MOMENT, YIELD CURVATURE, KD, MAX CONCRTE STRESS
2161.3 .13988E-03 5.209 2.624

ULTIMATE MOMENT, ULTIMATE CURVATURE
2002.8 .10840E-02

INPUT DATA: 3.0 1.2 1.2 1.2 3.0 2.3753.3133.3133.3132.37518. 18. 108.27

GROSS, MOMENT OF INERTIA, AREA, CRACKING MOMENT

	391.2	900.1	
MOMENT, CURVATURE, KD, POINT			
10776.4	391.2	900.1	
1092.34	146495E-04		8.603 1
MOMENT, CURVATURE, KD, POINT			
2109.74	11456E-03		6.983 2
MOMENT, CURVATURE, KD, POINT			
3044.16	18069E-03		6.641 3
MOMENT, CURVATURE, KD, POINT			
3749.16	24972E-03		6.407 4
MOMENT, CURVATURE, KD, POINT			
4087.56	33272E-03		6.011 5
MOMENT, CURVATURE, KD, POINT			
4213.41	42604E-03		5.633 6
MOMENT, CURVATURE, KD, POINT			
4323.90	51517E-03		5.435 7
MOMENT, CURVATURE, KD, POINT			
4392.22	60894E-03		5.255 8
MOMENT, CURVATURE, KD, POINT			
4416.57	7118E-03		5.057 9
MOMENT, CURVATURE, KD, POINT			
4442.70	81416E-03		4.913 10

[illegible]

INPUT DATA:

EX11

7. REFERENCES

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PROGRAM LISTING

- LMN: SPINES -

PCL
COPY MONCURV TO LP(ING)

COMPUTE SPRING SYSTEMESIS SPINES GIVEN MEMBER X-SECTIN DETAILS.

A. MORRIS MARCH 1978

UNITS ARE IN KIIPS AND INCHES, FIC=4, FIS=60, EC=3605, ES=29000, N=8
FIVE LEVELS OF REINFORCEMENT ARE PERMITTED WITH AT LEAST TWO
LAYERS REQUIRED, ONE AT THE TOP SURFACE THE OTHER AT THE BOTTOM
SURFACE.

```

STEEL(I)..STEEL AREA AT LEVEL I=1,5
SPACNG(I)..SPACING OF REINFT TO TO BOTTOM OF MEMBER, I=1,6
W.....WIDTH OF MEMBER
H.....D/ALL DEPTH OF MEMBER
RII).....STRESS /STRAIN MOMENT AREA FACTOR, I=1,10
GII).....STRESS SHAPE FACTOR, I=1,10
GROSSA.....GROSS AREA OF MEMBER INCLUDING REINFORCEMENT
GROSSI.....GROSS MOMENT OF INERTIA OF MEMBER INCLUDING REINFT
DATA( )...INPUT DATA
ARM.....MOMENT ARM OF REINFT ABOUT MEMBER TOP
CENTRO.....DIST BETWEEN HALF DPTH AND AREA CENTROID
MOMENT.....MOMENT OF STEEL ABOUT REFERENCE OR O/ALL SECTION
MOMENT CAPACITY
YBAR.....CENTROIDAL DIST TO MEMBER TOP
AXLOAD.....AXIAL LOAD
KD,JD,A,D. AS PER THE USUAL REINFORCEMENT CONVENTION
STEELI...STRAIN OF STEEL AT LEVEL I,ETC
STC.....MAX CONCRETE STRAIN
FSI,ETC...FORCE IN STEEL AT LEVEL I,ETC
CHKST.....CRITERIA STRAIN AT ULTIMATE STRENGTH
AOD.....RATIO OF A TO D
AREA .....AREA UNDER STRESS STRAIN CURVE (CONCRETE)
GAP.....SMALL INTERVAL OF STRAIN (CONCRETE)
STRAIN.....STRAIN MOMENT ARM FOR FINDING R( )
TOTA.....ACCUMULATED AREA UNDER STRESS STRAIN CURVE (CONC)
TOTMOM.....ACCUMULATED MOMENTS OF SMALL AREAS UNDER STRESS
STRAIN CURVE (CONCRETE)
DAREA.....SUB AREA UNDER STRESS STRAIN CURVE (CONCRETE)

```

FOR SUPPORTING THEORY REFER TO "REINFORCED CONCRETE
STRUCTURES" BY R. PARK AND T. RAULAY, PAGES 195 ONWARDS.


```

1      DATA(K),K=1,20)
3  FORMAT (13F5.2,F6.2,T1,20A4)
  WRITE(108,4) (DATA(I),I=1,20)
4  FORMAT (1X,1 INPUT DATA: ',20A4/)
  IF (STEEL(1).EQ.0.) CALL EXIT

```

BEAMS AND COLUMNS:-

CRACKING POINT/

FLAG=0.

REINFT=0.

DO 5 I=1,5

REINFT=REINFT+STEEL(I)

5 CONTINUE

GROSSA=M*H/47.*REINFT

MOMENT=M*H*H*.5

ARM=SPACNG(1)

IF (STEEL(2).EQ.0..AND.8TEEL(3).EQ.0..AND.8TEEL(4).EQ.0.) GOTO 50

IF (STEEL(2).EQ.0..AND.8TEEL(3).EQ.0.) GOTO 51

IF (STEEL(3).EQ.0..AND.8TEEL(4).EQ.0.) GOTO 52

IF (STEEL(3).EQ.0.) GOTO 53

GOTO 6

50 SPACNG(2)=(H*.5-SPACNG(1))*5

SPACNG(3)=SPACNG(2)

SPACNG(4)=(H*.5-SPACNG(6))*5

SPACNG(5)=SPACNG(4)

GOTO 6

51 SPACNG(2)=(H*.5-SPACNG(1))*5

SPACNG(3)=SPACNG(2)

SPACNG(4)=(H*.5-SPACNG(5))-8PACNG(6)

GOTO 6

52 SPACNG(3)=H*.5-SPACNG(1)-SPACNG(2)

SPACNG(4)=(H*.5-SPACNG(6))*5

SPACNG(5)=SPACNG(4)

GOTO 6

53 SPACNG(3)=H*.5-SPACNG(1)-SPACNG(2)

SPACNG(4)=H*.5-SPACNG(6)-SPACNG(5)

DO 7 I=1,5

MOMENT=MOMENT+7.*STEEL(I)*ARM

ARM=ARM+SPACNG(I+1)

7 CONTINUE

YBAR=MOMENT/GROSSA

```

CENTRD=H*.5-YBAR
GROSSI=W*H*H/12.+.H*H*CENTRD*CENTRD
GROSSI=GROSSI+(STEEL(1)*(SPACNG(2)+SPACNG(3)-CENTRD)**2
1      + STEEL(2)*(SPACNG(3)-CENTRD)**2
2      + STEEL(3)*CENTRD**2
3      + STEEL(4)*(SPACNG(4)+CENTRD)**2
4      + STEEL(5)*(SPACNG(4)+SPACNG(5)+CENTRD)**2)*7.
IF (AXLOAD,NE.0.) GOTO 29
YT=SPACNG(1)+SPACNG(2)+SPACNG(3)-CENTRD
YB=SPACNG(4)+SPACNG(5)+SPACNG(6)+CENTRD
TMP=.474*GROSSI
CMPOS=TMP/YB
CMNEG=TMP/YT
WRITE (108,8)
8 FORMAT (1X,1+CRACKING MOMENT,YBOTTOM,-CRACKING MOMENT,YTOP,1
1      ,GROSS MOMENT OF INERTIA,1)
WRITE (108,9) CMPOS,YB,CMNEG,YT,GROSSI
9 FORMAT (5X,2(F7.1,3X,F5.2),5X,F7.1//)

```

BEAMS:-

YEILDING POINT:

```

11 B=2.+.15.+(STEEL(1)+STEEL(2))+8.+(STEEL(3)+STEEL(4)+STEEL(5))/W
C=2.+.18.+(H-SPACNG(6))+8.+(STEEL(3)+STEEL(4)+STEEL(5))
1 -8.+(STEEL(4)+SPACNG(5)+STEEL(3)+SPACNG(5)+STEEL(3)*SPACNG(4))
2 +17.+(STEEL(1)+SPACNG(1)+STEEL(2)+SPACNG(1)
3 +STEEL(2)+SPACNG(2))/W
D=B*8+.5C
KD=(.8+SQRT(D))*.5
KDD=H-KD*SPACNG(6)
ST5=YLDST
ST4=ST5+(KD-SPACNG(5))/KDD
ST3=ST5+(KDD-SPACNG(5)-SPACNG(4))/KDD
STC=ST5+KD/KDD
ST1=STC+(KD-SPACNG(1))/KD
ST2=STC+(KD-SPACNG(1)-SPACNG(2))/KD
CSTRSS=STC+3605.
FC=.5*W*KD+CSTRSS
FS1=STEEL(1)*ST1+29000.
FS2=STEEL(2)*ST2+29000.
FS3=STEEL(3)*ST3+29000.
FS4=STEEL(4)*ST4+29000.

```



```

FSS=STEEL(5)*STB*29000.
FCOMP=FC+FS1+FS2
FTENS=FS3+FS4+FS5
YCOMP=(FC+KD/3.+FS1*SPACNG(1)+FS2*(SPACNG(1)+SPACNG(2)))/FCOMP
YTENS=(FSS+SPACNG(6)+FS4*(SPACNG(6)+SPACNG(5))
1      +FS3*(SPACNG(6)+SPACNG(5)+SPACNG(4)))/FTENS
JD=N-YCOMP-YTENS
YMON=FTENS*JD
YCUR=ST5/KOD
WRITE (108,12)
12 FORMAT (1X,'YIELD MOMENT,YIELD CURVATURE,KD,MAX CONCRETE STRESS')
WRITE (108,13) YMON,YCUR,KD,CSTRSS
43 FORMAT (5X,F7.1,5X,E12.5,5X,F6.3,5X,F5.3//)
C
C      ULTIMATE POINT
C
DD=SPACNG(1)+SPACNG(2)*STEEL(2)/(STEEL(1)+STEEL(2))
D=H-SPACNG(6)-SPACNG(5)*STEEL(4)/(STEEL(4)+STEEL(5))
IF (FLAG.EQ.1) UMON=(STEEL(4)+STEEL(5))*60.*(D-DD)
IF (FLAG.EQ.1) GOTO 70
FCONG=(STEEL(4)+STEEL(1)-STEEL(2))*60.
IF (FCONG.EQ.0.) GOTO 15
CHKST=.00*(1.-.85+.85*(H+D)/FCONG)
IF (CHKST.GE.YLDBT) GOTO 15
D=2.*(STEEL(1)+STEEL(2))*OD*.29000.-(STEEL(4)+STEEL(5))*60.
1 / (1.7*(H+D))
C=2.*(STEEL(1)+STEEL(2))*OD*.29000.+.85*DD/(1.7*(H+D)*D)
TMP=8+.4*.C
ADD=(8+SORT(TMP))*5
X=ADD*D
UMON=.85+.4*.A*(D-A*.5)
1 +STEEL(1)+STEEL(2))*29000.+.00*(A-.85*DD)/A*(D-DD)
GOTO 17
15 A=FCONG/(.85+.4*.H)
UMON=.85+.4*.A*(D-A*.5)*(STEEL(1)+STEEL(2))*60.*(D-DD)
17 IF (FCONG.EQ.0.) UCUR=YLDBT/(H+.5*SPACNG(1))
IF (FCONG.NE.0.) UCUR=.00+.85/A
70 WRITE (108,18)
18 FORMAT (1X,'ULTIMATE MOMENT,ULTIMATE CURVATURE')
WRITE (108,19) UMON,UCUR
19 FORMAT (5X,F7.1,5X,E12.5//)
IF (FLAG.EQ.1) GOTO 1
DO 21 1-176

```

TEMP(I)=SPACNG(I)

21 CONTINUE

J=0

DO 23 I=1,6

SPACNG(I)=TEMP(6-J)

J=J+1

23 CONTINUE

DO 25 I=1,5

TEMP(I)=STEEL(I)

25 CONTINUE

J=0

DO 27 I=1,5

STEEL(I)=TEMP(5-J)

J=J+1

27 CONTINUE

FLAG=1.

GOTO 11

C

C

C

C

C

COLUMNS1=

MOMENT-AXIAL LOAD-CURVATURE PLOT

29 DO 30 I=1,22

PLOT(I)=0.

30 CONTINUE

CMCH=2.*GROSS1/H*(.475+AXLOAD/GROSS1)

VLAX=0.

XLAX=0.

TOL=500.*AXLOAD

IF (TOL.LT.1.5) TOL=2.5

IF (FLAG.EQ.1.) GOTO 33

FLAG=1.

CM1=.0004

GAP=.00004

AREA=0.

MOMENT=0.

TOTL=0.

TOTMCH=0.

STRAIN=.00002

DO 32 I=1,10

DO 31 J=1,10

RATIO=STRAIN/.002

DAREA=GAP+.12.*RATIO-RATIO*RATIO

4 OF 4

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```

IF (STRAIN.GE..00202) DAREA=GAP***(1.-100.*(STRAIN-.002))
AREA=AREA+DAREA
MOMENT=MOMENT+DAREA*STRAIN
STRAIN=STRAIN+.00004

```

31 CONTINUE

```

TOTAT=TOTAT+AREA
TOTMOM=TOTMOM+MOMENT
Q(I)=TOTAT/(4.*CST)
R(I)=1.-TOTMOM/(TOTAT*CST)
CST=CST+.0004

```

```

AREA=0.
MOMENT=0.

```

32 CONTINUE

33 CST=.0004

```

DO 41 I=1,10
KD=H

```

ICOUNT=0

```

34 ICOUNT=ICOUNT+1
IF (ICOUNT.EQ.1000) GO TO 40
ST1=CST*(KD-SPACNG(1))/KD
ST2=CST*(KD-SPACNG(1))-SPACNG(2))/KD
ST3=CST*(KD-SPACNG(1))-SPACNG(2))-SPACNG(3))/KD
ST4=CST*(KD-SPACNG(1))-SPACNG(2))-SPACNG(3))-SPACNG(4))/KD
ST5=CST*(KD-SPACNG(1))-SPACNG(2))-SPACNG(3))-SPACNG(4))-SPACNG(5))/KD
FS1=STEEL(1)*ST1*29000.
IF (ST1.GT.YLDS1) FS1=STEEL(1)*YLDS1*29000.
FS2=STEEL(2)*ST2*29000.
IF (ST2.GT.YLDS1) FS2=STEEL(2)*YLDS1*29000.
FS3=STEEL(3)*ST3*29000.
IF (ST3.LT.-YLDS1) FS3=-STEEL(3)*YLDS1*29000.
FS4=STEEL(4)*ST4*29000.
IF (ST4.LT.-YLDS1) FS4=-STEEL(4)*YLDS1*29000.
FS5=STEEL(5)*ST5*29000.
IF (ST5.LT.-YLDS1) FS5=-STEEL(5)*YLDS1*29000.
P=Q(I)+.4*W*KD+FS1+FS2+FS3+FS4+FS5
TMP=AXLOAD-P
DIFF=ABS(TMP)
IF (DIFF.LE.TOL) GO TO 35
KD=KD-H*.001
GO TO 34

```

35 MOMENT=Q(I)+.4*W*KD*(H*.5-R(I)*KD)

```

1 +FS1*(H*.5-SPACNG(1))
2 +FS2*(H*.5-SPACNG(1))-SPACNG(2))

```

```

3   +FS3*(H*.5-SPACNG(1)-SPACNG(2)-SPACNG(3))
4   +FS4*(H*.5-SPACNG(1)-SPACNG(2)-SPACNG(3)-SPACNG(4))
5   +FS5*(H*.5-SPACNG(1)-SPACNG(2)-SPACNG(3)-SPACNG(4)
6   -SPACNG(5))
    CURV=CST/KD
    PLOT(I+1)=CURV
    PLOT(I+12)=MOMENT
    IF (MOMENT.GT.YLAX) YLAX=MOMENT
    XLAX=CURV
    IF (FLAG.EQ.0.) WRITE (108,36)
36  FORMAT (1X,'GROSS' MOMENT OF INERTIA,AREA,CRACKING MOMENT')
37  FORMAT (10X,F7.1,5X,F5.1,5X,F7.1)
    FLAG=1.
    WRITE (108,38)
38  FORMAT (10X,'MOMENT',CURVATURE,KD,POINT')
    WRITE (108,39) MOMENT,CURV,KD,I
39  FORMAT (10X,F8.2,5X,E12.5,5X,F6.3,5X,I2)
40  CST=CST+.0004
41  CONTINUE
    CALL PLOT(PLOT,N,M,XLAX,XLIN,YLAX,YLIN)
    WRITE (108,54)
    GOTO 1
    END
PCL PROCESSING TERMINATED

```

- LMN: QUAKES -

PCL
COPY QUAKER TO LP(NC)
BLOCK DATA

C	THIS SUBPROGRAM INITIALIZES ALL ARRAYS PASSED BETWEEN THE	QUAKE
C	MAIN PROGRAM AND THE SUBROUTINES.	QUAKE
C		QUAKE
	REAL JLU,JO,MASS	QUAKE
	COMMON /STIFFS/ SKI 95,151,SKKOPY(95,15),GSK(12),SWK(3,3),	QUAKE
	BKI 32,3),BEF(32),BFI 32,2),BSF(32),	QUAKE
1	CKI 42,3),CEF(42),CIFI 42,2),CSF(42)	QUAKE
2	BMI 32,2),BMO(32,2),BRI 32,2),BRO(32,2),	QUAKE
1	CHI 42,2),CMO(42,2),CRI 42,2),CRO(42,2),	QUAKE
2	PI(42),PO(42),OTH	QUAKE
	COMMON /JOINTS/ JLU(95),JD(95)	QUAKE
	COMMON /FLOORS/ VEL(95),ACC(95),FF(12),FV(12),SWAY(12),RSWY(12)	QUAKE
	COMMON /MEMBER/ NDFBI 32,4),BRZL(32),BCL(32),BRZR(32),LBM(32),	QUAKE
1	NDFCI 42,6),CRZL(42),CCL(42),CRZU(42),LCM(42),	QUAKE
2	CAI(42)	QUAKE
	COMMON /ENVLP/ BMX(32,2),BMN(32,2),BRX(32,2),BRN(32,2),	QUAKE
1	CMX(42,2),CMN(42,2),CRX(42,2),CRN(42,2),	QUAKE
2	PMX(42),PMN(42),VELX(12),ACCX(12),FVX(12),	QUAKE
3	FVN(12),SWYX(12),SWYN(12),RSWX(12),RSWN(12),	QUAKE
4	OTMX,OTMN	QUAKE
	COMMON /MISCEL/ F7,DF,F8,DF8,F9,DF9,F10,DF10,F11,DF11,F12,DF12,E,	QUAKE
1	LSC(42),LVL(96),NDFH(12),MASS(12),ARM(12),GRF	QUAKE
	COMMON /SPRING/ BCM(32,4),BCR(32,4),BYM(32,4),BYR(32,4),	QUAKE
1	BUM(32,4),BURI 32,4),BUMPI 32,4),BURPI 32,4),	QUAKE
2	BUM7(32,2),BUR7(32,2),BUM9(32,2),BUR9(32,2),	QUAKE
3	BUM11(32,2),BUR11(32,2),BUM12(32,2),	QUAKE
4	BUM13(32,2),B16(32,2),B18(32,2),B110(32,2),	QUAKE
5	B112(32,2),BF1(32,2),BF2(32,2),BSR(32,2),	QUAKE
6	LBK(32,2),	QUAKE
7	CCM(42,4),CCR(42,4),CYM(42,4),CYR(42,4),	QUAKE
8	CUM(42,4),CUR(42,4),CUMPI 42,4),CURPI 42,4),	QUAKE
9	CUM7(42,2),CUR7(42,2),CUM9(42,2),CUR9(42,2),	QUAKE
1	CUM11(42,2),CUR11(42,2),CUM13(42,2),	QUAKE
2	CUM13(42,2),CI6(42,2),CI8(42,2),CI10(42,2),	QUAKE
3	CI12(42,2),CF1(42,2),CF2(42,2),CSR(42,2),	QUAKE
4	LCK(42,2),UF,RF	QUAKE
	DATA SK,SKKOPY,GSK,SWK,BK,BEF,BIF,BSF,BK,CEF,CIF,CSF / 1425*0.,	QUAKE
1	1425*0.,12*0., 9*0., 96*0., 32*0., 64*0., 32*0.,126*0.,	QUAKE
2	42*0., 84*0., 42*0. /	QUAKE

C LINEAR SPRINGS CONCEPTUALLY LOCATED AT THE MEMBER ENDS, THEIR
C STIFFNESS IS DEFINED IN TERMS OF ANTISYMMETRICAL MEMBER FLEXURE.
C RULES USED TO MANIPULATE THE POSSIBLE STIFFNESS REGIMES ARE
C DERIVED FROM THE WORK OF I. TAKEDA ET AL AT THE UNIVERSITY OF
C ILLINOIS.
C THE DYNAMIC EQUATION DEFINED IN TERMS OF JOINT DISPLACEMENTS
C IS NUMERICALLY INTEGRATED OVER AN INTERVAL OF TIME WITHIN WHICH IT
C IS ASSUMED THAT THE STRUCTURE STIFFNESS MATRIX AND HENCE STIFFNESS
C DEPENDENT DAMPING REMAIN CONSTANT WHILST THE JOINT ACCELERATIONS
C VARY LINEARLY OR REMAIN CONSTANT DEPENDING ON USER PREFERENCE.
C FINALLY, IT IS ASSUMED THAT THE GROUND AND STRUCTURE MOTION DO NOT
C INTERACT, IN LIEU OF A SEISMIC EVALUATION HOWEVER, THE PROGRAM CAN
C BE DIRECTED TO PERFORM A MODAL ANALYSIS.
C THE PROGRAM IS WRITTEN IN FORTRAN IV FOR THE XEROX COMPUTERS
C SERIES SIGMA 5, 7, AND 9 AND THE IBM COMPUTERS SERIES 370/168 AND
C 360/85 AND WITH MINOR CHANGES MAY BE ADAPTED TO OTHER MACHINES. IT
C IS ORGANIZED INTO THREE STAGES, PRELIMINARY WORK, STATIC ANALYSIS,
C AND DYNAMIC ANALYSIS. EACH STAGE IS SUBDIVIDED INTO AN APPROPRIATE
C NUMBER OF 'TASK MODULES' AND SUBROUTINES.
C IN ESSENCE THE PROGRAM MAY PERFORM ANY OF THE FOLLOWING TASKS
C (1) A MODAL ANALYSIS OR,
C (2) A STATIC ANALYSIS OR,
C (3) A SEISMIC ANALYSIS OR,
C (4) A COMBINED STATIC-SEISMIC ANALYSIS.
C THE TOTAL CORE REQUIREMENTS IN DECIMAL WORDS TO STORE THE
C DATA, GENERATED CODE, OPERATING SYSTEM REQUIREMENTS ETC CAN BE
C ESTIMATED FROM THE FOLLOWING FORMULA,
C
C
$$NB*(90*P+10)+NC*(100*P+10)+NS*((2*NS+15)*P+2)$$

C
$$+NDF*((2*MBAND+4)*P+1)+360*P+8+11200$$

C
C WHERE P=2 IF ALL REAL VARIABLES COMPILED IN DOUBLE PRECISION,
C P=1 IF ALL REAL VARIABLES COMPILED IN SINGLE PRECISION,
C NB =S THE NUMBER OF BEAMS,
C NC =S THE NUMBER OF COLUMNS,
C NS =S THE NUMBER OF STORIES,
C NDF =S THE NUMBER OF DEGREES OF FREEDOM,
C MBAND =S THE MAXIMUM HALF BAND WIDTH, AND
C 0 =S THE OPERATING SYSTEM REQUIREMENTS (=2100 FOR THE
C XEROX COMPUTERS). NOTE THAT FOR A MIXTURE OF DOUBLE AND
C SINGLE PRECISION REAL VARIABLES, THE CORE REQUIREMENTS WILL LIE
C BETWEEN THE VALUES PRECICIED FOR PURE DOUBLE AND SINGLE PRECISION.
C IT IS REPORTED THAT UPTO 80 PERCENT OF THE EXECUTION TIME OF

COMMON /ENVELOP/	BMA(32,2),BMN(32,2),BRX(32,2),BRN(32,2),	QUAKE
1	CMX(42,2),CMN(42,2),CRX(42,2),CRN(42,2),	QUAKE
2	PMX(42,2),PMN(42,2),VELX(12),ACCX(12),FVX(12),	QUAKE
3	FVN(12),SWYX(12),SMYN(12),RSMX(12),RSMN(12),	QUAKE
4	OTMX,OTMN	QUAKE
COMMON /ACTION/	BM(32,2),BMO(32,2),BR(32,2),BRO(32,2),	QUAKE
1	CM(42,2),CMO(42,2),CR(42,2),CRO(42,2),	QUAKE
2	P(42,2),PO(42,2),BOM	QUAKE
COMMON /MEMBER/	NDP8(32,2),BRZL(32,2),BCL(32,2),BRZR(32,2),LBM(32,2),	QUAKE
1	NDPC(42,2),CRZL(42,2),CCL(42,2),CRZU(42,2),LCM(42,2),	QUAKE
2	CAI(42,2)	QUAKE
COMMON /MISCEL/	F7,DF7,F8,DF8,F9,DF9,F10,DF10,F11,DF11,F12,DF12,E,	QUAKE
1	LSC(42,2),LVL(96,2),NDFM(12),MASS(12),ARM(12),GRF	QUAKE
COMMON /SPRING/	BCH(32,2),BCR(32,2),BYM(32,2),BYR(32,2),	QUAKE
1	BUM(32,2),BUR(32,2),BUMP(32,2),BURP(32,2),	QUAKE
2	BUM7(32,2),BUR7(32,2),BUM9(32,2),BUR9(32,2),	QUAKE
3	BUM11(32,2),BUR11(32,2),BUM13(32,2),	QUAKE
4	BUM13(32,2),B16(32,2),B18(32,2),B110(32,2),	QUAKE
5	B112(32,2),BF1(32,2),BF2(32,2),BSR(32,2),	QUAKE
6	LBK(32,2),	QUAKE
7	CCM(42,2),CCR(42,2),CYM(42,2),CYR(42,2),	QUAKE
8	CUM(42,2),CUR(42,2),CUMPI(42,2),ACURPI(42,2),	QUAKE
9	CUM7(42,2),CUR7(42,2),CUM9(42,2),CUR9(42,2),	QUAKE
1	CUM11(42,2),CUR11(42,2),CUM13(42,2),	QUAKE
2	CUM13(42,2),CI6(42,2),CI8(42,2),CI10(42,2),	QUAKE
3	CI12(42,2),CF1(42,2),CF2(42,2),CSR(42,2),	QUAKE
4	LCK(42,2),UF,KF	QUAKE
EQUIVALENCE	(FEM,BM),(FEV,BR),(GRL,SK),(BSA,BSF),(BI,BEF),	QUAKE
1	(BUR,BYR),(CSA,CSF),(CI,CEF),(CDR,CYR),(FFH,FF)	QUAKE

TASK MODULE 1-2... READ THE FOLLOWING DATA FROM CARDS.

NOTE THAT AN EXPLANATION OF VARIABLES DEFINED ABOVE AND ALL MAJOR LOCAL AND COMMON VARIABLES DEFINED SUBSEQUENTLY IS GIVEN THROUGHOUT THE PROGRAM WHERE APPROPRIATE.

FOLLOWS DATA FOR PAGE HEADINGS AND OUTPUT CONTROL:

JOB(*),...JOB IDENTIFICATION.

EON(*),...NAME OF THE EARTHQUAKE RECORD.

UNITS(*),...CONSISTENT UNITS OF MASS,FORCE,LENGTH,AND TIME.

MLP.....MAXIMUM NUMBER OF PRINTED LINES REQUIRED PER PAGE.

IPR.....INTERVAL BETWEEN OUTPUT OF DYNAMIC RESPONSE.

1	READ (IR,1) JOB	QUAKE
1	FORMAT (20A4)	QUAKE
2	READ (IR,3) EUN	QUAKE
3	FORMAT (20A4)	QUAKE
4	READ (IR,5) UNITS,MLP,IPR	QUAKE
5	FORMAT (4(A3,2X),2I5)	QUAKE
C	FOLLOWS QUALITATIVE, AND QUANTITATIVE DATA:	QUAKE
C	E.....YOUNG'S MODULUS.	QUAKE
C	G.....SHEAR MODULUS.	QUAKE
C	ALFA.....MASS RELATED DAMPING FACTOR.	QUAKE
C	BETA.....STIFFNESS RELATED DAMPING FACTOR.	QUAKE
C	GRK.....GRAVITY CONSTANT.	QUAKE
C	NB.....NUMBER OF BEAMS.	QUAKE
C	NLB.....NUMBER OF LOADED BEAMS.	QUAKE
C	NC.....NUMBER OF COLUMNS.	QUAKE
C	NS.....NUMBER OF STORIES.	QUAKE
C	NDF.....NUMBER OF KINEMATIC DEGREES OF FREEDOM.	QUAKE
C	MBAND.....MAXIMUM 1/2 BAND WIDTH OF STRUCTURE STIFFNESS	QUAKE
C	MATRIX INCLUDING THE MAIN DIAGONAL.	QUAKE
C	FLAG9.....IF NATURAL FREQUENCIES REQUIRED (=1), IF NOT (=5).	QUAKE
C	EQD.....REQUIRED DURATION OF EARTHQUAKE SIMULATION.	QUAKE
C	ASF.....EARTHQUAKE ACCELERATION SCALING FACTOR.	QUAKE
C	BTA.....RECIPROCAL OF NEWMARK'S BETA (EITHER 6. OR 4.).	QUAKE
C	DT.....TIME STEP OF NUMERICAL INTEGRATION.	QUAKE
C	DDT.....TIME SUB-STEP OF NUMERICAL INTEGRATION.	QUAKE
C	BVO.....INITIAL BASE VELOCITY.	QUAKE
C	PDSF.....P-DELTA SCALING FACTOR.	QUAKE
C	GAP.....INTERVAL BETWEEN CONSECUTIVE INPUT ACCELERATION	QUAKE
C	RECORDS.	QUAKE
C	READ (IR,7) E,G,ALFA,BETA,GRK	QUAKE
7	FORMAT (5F10.0)	QUAKE
8	READ (IR,9) NB,NLB,NC,NS,NDF,MBAND,FLAG9	QUAKE
9	FORMAT (7I5)	QUAKE
11	READ (IR,11) EQD,ASF,BTA,UT,DDT,BVO,PDSF,GAP	QUAKE
11	FORMAT (8F10.0)	QUAKE
C	FOLLOWS MEMBER DATA:	QUAKE
C	FLAG1.....INDICATES OPTION SELECTED FOR DEFINING SPRING DISP-	QUAKE
C	-LACEMENTS; CURVATURE (=0), ROTATIONS (=1).	QUAKE
C	UF.....SPRING UNLOADING FACTOR.	QUAKE
C	RF.....SPRING RELOADING FACTOR (FOR NONDEGRADING SPRINGS).	QUAKE

DB 21 K=1,NB		QUAKE
READ (IR,15) I,(NDP,B(I,J),J=1,4),BRZ(I),BCL(I),BRZR(I),BSA(I),		QUAKE
BI(I)		QUAKE
15 FORMAT (5I5,4F5.0,4F10.0)		QUAKE
IF (EQD.EQ.0.) GOTO 21		QUAKE
IF (FLAG9.EQ.1) GOTO 21		QUAKE
READ (IR,17) (BCM(I,J),J=1,4)		QUAKE
17 FORMAT (4F10.0)		QUAKE
READ (IR,19) (BYM(I,J),J=1,4), (BYR(I,J),J=1,4)		QUAKE
19 FORMAT (8F10.0)		QUAKE
READ (IR,19) (BUM(I,J),J=1,4), (BUR(I,J),J=1,4)		QUAKE
21 CONTINUE		QUAKE
C		QUAKE
C COLUMNS-		QUAKE
C I.....IDENTIFICATION.		QUAKE
C KINEMATIC STRUCTURAL COORDINATES-		QUAKE
C NDFC(I,1).ROTATION-LOWER END.		QUAKE
C NDFC(I,2).ROTATION-UPPER END.		QUAKE
C NDFC(I,3).SWAY-LOWER END.		QUAKE
C NDFC(I,4).SWAY-UPPER END.		QUAKE
C NDFC(I,5).AXIAL DEFORMATION-LOWER END.		QUAKE
C NDFC(I,6).AXIAL DEFORMATION-UPPER END.		QUAKE
C CRZL(I).....RIGID LENGTH-LOWER END.		QUAKE
C CCL(I).....CLEAK LENGTH.		QUAKE
C CRZUI(I).....RIGID LENGTH-UPPER END.		QUAKE
C CAI(I).....AREA.		QUAKE
C CSA(I).....EFFECTIVE SHEAR AREA.		QUAKE
C CI(I).....MOMENT OF INERTIA.		QUAKE
C COLUMN SPRINGS-		QUAKE
C CCM(I,1).....POSITIVE CRACKING MOMENT-LOWER END.		QUAKE
C CCM(I,2).....NEGATIVE CRACKING MOMENT-LOWER END.		QUAKE
C CCM(I,3).....POSITIVE CRACKING MOMENT-UPPER END.		QUAKE
C CCM(I,4).....NEGATIVE CRACKING MOMENT-UPPER END.		QUAKE
C CYM(I,1).....POSITIVE YIELDING MOMENT-LOWER END.		QUAKE
C CYM(I,2).....NEGATIVE YIELDING MOMENT-LOWER END.		QUAKE
C CYM(I,3).....POSITIVE YIELDING MOMENT-UPPER END.		QUAKE
C CYM(I,4).....NEGATIVE YIELDING MOMENT-UPPER END.		QUAKE
C CYR(I,1).....POSITIVE YIELDING ROTATION-LOWER END.		QUAKE
C CYR(I,2).....NEGATIVE YIELDING ROTATION-LOWER END.		QUAKE
C CYR(I,3).....POSITIVE YIELDING ROTATION-UPPER END.		QUAKE
C CYR(I,4).....NEGATIVE YIELDING ROTATION-UPPER END.		QUAKE
C CUM(I,1).....POSITIVE ULTIMATE MOMENT-LOWER END.		QUAKE
C CUM(I,2).....NEGATIVE ULTIMATE MOMENT-LOWER END.		QUAKE


```

31 DO 35 I=1,NLB
  READ (IR,33) ID(I),NCL(ID(I)),GR(ID(I),J),J=1,7)
33 FORMAT (215,F10.0)
35 CONTINUE

C
C
C STORIES-
C I.....IDENTIFICATION,FIRST STORY ABOVE GROUND IS 1.
C NDFM(I)....KINEMATIC STRUCTURAL COORDINATE ASSOCIATED WITH
C STORY SWAY.
C MASS(I)....MASS.
C

37 DO 41 J=1,NS
  READ (IR,39) I,NDFM(I),MASS(I)
39 FORMAT (215,F10.0)
41 CONTINUE

C
C TASK MODULE 1-3 ... LIST THE INPUT DATA.
C
C IP.....PAGE NUMBER
C
C IP=1
C WRITE (IW,43) IP
43 FORMAT (1H1,120X,'PAGE ',I2,
1 60X,'*** QUAKE ***')
2 42X,'PROGRAM FOR THE ELASTIC-INELASTIC SEISMIC RESPONSE'
3 45X,'OF DEGRADING REINFORCED CONCRETE STRUCTURES.'
4 55X,'A.L.TORRIS OTTAWA 1977.'//)

C
C PRINT A LOGO IN THE FORM OF A FOUR STORY FRAME SWAYING TO ITS
C RIGHT.
C
C WRITE (IW,45)
45 FORMAT (55X,32,'/55X,32,'/55X,4(2,'/8X)/55X,4(2,'/8X)/
1 54X,4(2,'/8X)/53X,4(2,'/8X)/53X,4(2,'/8X)/
2 53X,32,'/53X,32,'/53X,4(2,'/8X)/53X,4(2,'/8X)/
3 52X,4(2,'/8X)/51X,4(2,'/8X)/51X,4(2,'/8X)/
4 51X,32,'/51X,32,'/51X,4(2,'/8X)/51X,4(2,'/8X)/
5 50X,4(2,'/8X)/49X,4(2,'/8X)/49X,4(2,'/8X)/
6 49X,32,'/49X,32,'/49X,4(2,'/8X)/49X,4(2,'/8X)/
7 48X,4(2,'/8X)/47X,4(2,'/8X)/47X,4(2,'/8X)/
8 47X,4(2,'/8X)/45X,36,'-')
C IP=IP+1
C WRITE (IW,47) IP,JOB,EUN

```

81 P

47 FORMAT (1H1, 120X, 'PAGE ', I2/27X, 20A4/27X, 20A4//
1 59X, '** INPUT DATA **', I7//)

GENERAL DATA:

WRITE (IW,49) UNITS(1),MLP,E,UNITS(2),IPR,G,UNITS(3),UNITS(4),
1 ALFA,NB,EQD,BETA,MLB,ASF,NC,GAP,NS,NDF

49 FORMAT (/61X, 'GENERAL DATA', //10X, UNITS OF:')

1 34X, 'OUTPUT CONTROL:', 28X, 'MODULI:', //11X, 'MASS', 20X, A3,

2 16X, 'MAXIMUM LINES', 12X, 12, 16X, 'YOUNG', 'S', 8X, E12.5/

3 11X, 'FORCE', 19X, A3, 16X, 'RESPONSE INTERVAL', 8X, I2,

4 16X, 'SHEAR', 10X, E12.5/11X, 'LENGTH', 18X, A3/11X, 'TIME',

5 20X, A3//10X, 'DAMPING FACTORS:', 27X, 'NUMBER OF:',

6 33X, 'EARTHQUAKE:', //11X, 'MASS', 11X, E12.5, 16X, 'BEAMS',

7 17X, 15, 16X, 'DURATION', 7X, E12.5/11X, 'STIFFNESS', 6X, E12.5,

8 16X, 'LOADED BEAMS', 10X, 15, 16X, 'SCALING FACTOR', 1X, E12.5/

9 54X, 'COLUMNS', 15X, 15, 16X, 'RECORD INTERVAL', E12.5/

1 54X, 'STORIES', 15X, 15/54X, 'DEGREES OF FREEDOM', 4X, I5//11X,

WRITE (IW,51) BTA,LAG1,GMK,DT,UF,PDSF,DDT,RF,MBAND,BVO,NOMN3E,

1 FLAG2,FLAG9

51 FORMAT (10X, 'MOTION EQUATION', 27X, 'SPRING PARAMETERS:',

1 25X, 'MISCELLANEOUS:', //11X, 'NEWMARK'S BETA', 11X, F2.0,

2 16X, 'DEFORMATION OPTION', 8X, I1, 16X, 'GRAVITY CONST',

3 2X, E12.5/11X, 'TIME STEP', 6X, E12.5, 16X, 'UNLOADING FACT',

4 1X, E12.5, 16X, 'P-DELTA FACTOR', 1X, E12.5/

5 11X, 'SUB TIME STEP', 2X, E12.5, 16X, 'RELOADING FACT',

6 1X, E12.5, 16X, 'MAX 1/P BANDWIDTH', 5X, I5,/

7 11X, 'INITL BASE VEL', X, E12.5, 16X, 'HINGE PRINT OPTION',

8 8X, 11, 16X, 'LOADING OPTION', 12X, I1/

9 97X, 'MODAL INFO OPTION', 9X, I1)

IP=IP+1

WRITE (IW,47) IP, JOB, EUN

MEMBER DATA:

WRITE (IW,53)

53 FORMAT (58X, 'BEAM PROPERTIES', //12X, NG, 125X, 'STRUCTURAL COORDS.',

1 7X, 'K', 18X, 'LENGTH', 18X, 13, 16X, 'SHEAR AREA',

2 7X, 'MOMENT OF', 45X, 'RIGID LEFT', 5X, 'CLEAR SPAN',

3 6X, 'RIGID RIGHT', 25X, 'INERTIA', /)

SET MLP TO DEFAULT VALUE IF INPUT VALUE BLANK. REDUCE THE

INPUT OR DEFAULT VALUE TO PROVIDE A HUFFER AGAINST MISCALCULATION


```

C      DO 99 I=1,NS
C      IF (NLP.LT.MLR) GO10 96
C      IP=IP+1
C      WRITE (IW,47) IP,JOB,EUN
C      WRITE (IW,95)
C      NLP=12
C      96 WRITE (IW,97) I,NDP*(I),MASS(I)
C      97 FORMAT (54X,2(13,5X),E12.5)
C      NLP=NLP+1
C      99 CONTINUE
C      101 CONTINUE
C
C      TASK MODULE 1-4 ... EDIT THE INPUT DATA. IF ERRORS ARE FOUND
C      LIST THEM AND TERMINATE THE PROCESSING.
C
C      FLAG3.....INDICATES DATA ERROR FREE (=0),OR WITH ERROR(S) (=1)
C      FLAG3=0
C
C      ----- TO BE DEVELOPED LATER, -----
C
C      TEST FOR ERROR(S).
C
C      IF (FLAG3.NE.0) GO10 513
C
C      TASK MODULE 1-5 ... PROCESS THE DATA AS REQUIRED.
C
C      DEFINE THE BASIC MOMENT-ROTATION RELATIONSHIP OF EACH SPRING.
C      NOTE THAT THIS RELATIONSHIP IS DEFINED W.R.T. THE ANTISYMMETRIC
C      MONOTONIC FLEXURE OF A PRISMATIC LINE MEMBER AND IS CONSIDERED
C      HERE TO BE INDIFFERENT TO THE PRESENCE OF AXIAL AND GRAVITY LOADS.
C      AUTOMATIC GENERATION OF THE CRACKING, YIELDING AND THE
C      ULTIMATE ROTATIONS FROM INPUT CURVATURES CAN BE UNDERTAKEN FOR THE
C      THREE FOLLOWING CASES ASSUMING THAT BOTH THE TOP AND BOTTOM
C      REINFORCEMENT AREAS ARE CONSTANT ALONG THE LENGTH OF THE MEMBER.
C      (1) TOP AND BOTTOM AREAS EQUAL OR,
C      (2) TOP AREA LESS THAN THE BOTTOM AREA OR,
C      (3) TOP AREA GREATER THAN THE BOTTOM AREA.
C
C      COMPUTE THE CRACKING ROTATIONS (M*L/6*E*I), TAKING INTO
C      CONSIDERATION ANY MEMBER SHEAR DEFORMATIONS.
C
C      BSF(I).....SHEAR FLEXIBILITY FACTOR (=6*E*I/L*L*AE+G)=BEAM I.

```

```

C      DO 99 I=1,NS
C      IF (NLP.LT.MLR) GO 10 96
C      IP=IP+1
C      WRITE (IW,47) IP,JOB,EWN
C      WRITE (IW,95)
C      NLP=12
C      96 WRITE (IW,97) I,NDP,M(I),MASS(I)
C      97 FORMAT (54X,2(I3,5X),E12.5)
C      NLP=NLP+1
C      99 CONTINUE
C      101 CONTINUE
C
C      TASK MODULE 1-4 ... EDIT THE INPUT DATA. IF ERRORS ARE FOUND
C      LIST THEM AND TERMINATE THE PROCESSING.
C
C      FLAG3.....INDICATES DATA ERROR FREE (=0),OR WITH ERROR(S) (=1)
C      FLAG3=0
C
C      -----7- TO BE DEVELOPED LATER. -----
C
C      TEST FOR ERROR(S).
C
C      IF (FLAG3.NE.0) GO 10 513
C
C      TASK MODULE 1-5 ... PROCESS THE DATA AS REQUIRED.
C
C      DEFINE THE BASIC MOMENT-ROTATION RELATIONSHIP OF EACH SPRING.
C      NOTE THAT THIS RELATIONSHIP IS DEFINED W,R,I. THE ANTISYMMETRIC
C      MONOTONIC FLEXURE OF A PRISMATIC LINE MEMBER AND IS CONSIDERED
C      HERE TO BE INDIFFERENT TO THE PRESENCE OF AXIAL AND GRAVITY LOADS.
C      AUTOMATIC GENERATION OF THE CRACKING, YIELDING AND THE
C      ULTIMATE ROTATIONS FROM INPUT CURVATURES CAN BE UNDERTAKEN FOR THE
C      THREE FOLLOWING CASES ASSUMING THAT BOTH THE TOP AND BOTTOM
C      REINFORCEMENT AREAS ARE CONSTANT ALONG THE LENGTH OF THE MEMBER.
C      (1) TOP AND BOTTOM AREAS EQUAL OR,
C      (2) TOP AREA LESS THAN THE BOTTOM AREA OR,
C      (3) TOP AREA GREATER THAN THE BOTTOM AREA.
C
C      COMPUTE THE CRACKING ROTATIONS (M*L/6*E*I), TAKING INTO
C      CONSIDERATION ANY MEMBER SHEAR DEFORMATIONS.
C      BSF(I).....SHEAR FLEXIBILITY FACTOR (=6*E*I/L*L*AE*G)-BEAM I.

```

```

C      CSF(I).....SHEAR FLEXIBILITY FACTOR (=6*E*I/L*L*AE*G)-COLUMN I.      QUAKE
C      BEF(I).....FLEXURAL FLEXIBILITY FACTOR (=L/6*E*I)-BEAM I.              QUAKE
C      CEF(I).....FLEXURAL FLEXIBILITY FACTOR (=L/6*E*I)-COLUMN I.              QUAKE
C
C      CONST=6.*E
C      DO 207 I=1,NB
C      EI=E*BI(I)
C      BEF(I)=BCL(I)/(CONST*BI(I))
C      IF (BSA(I).EQ.0..0X.G.EQ.0.) GOTO 201
C      BSF(I)=1./(BEF(I)*BCL(I)*BSA(I)*G)
C      V=1.+2.*BSF(I)
C      GOTO 203
C
C      201 V=1.
C      203 BCR(I,1)=BCM(I,1)*BEF(I)*V
C      BCR(I,2)=BCM(I,2)*BEF(I)*V
C      BCR(I,3)=BCR(I,2)
C      BCR(I,4)=BCR(I,1)
C      207 CONTINUE
C
C      DO 215 I=1,NC
C      EI=E*CI(I)
C      CEF(I)=CCL(I)/(CONST*CI(I))
C      IF (CSA(I).EQ.0..0X.G.EQ.0.) GOTO 209
C      CSF(I)=1./(CEF(I)*CCL(I)*CSA(I)*G)
C      V=1.+2.*CSF(I)
C      GOTO 211
C
C      209 V=1.
C      211 CCR(I,1)=CCM(I,1)*CEF(I)*V
C      CCR(I,2)=CCM(I,2)*CEF(I)*V
C      CCR(I,3)=CCR(I,2)
C      CCR(I,4)=CCR(I,1)
C      215 CONTINUE
C      IF (EQD.EQ.0.) GOTO 234
C      IF (FLAG9.EQ.1) GOTO 254
C      IF (FLAG1.EQ.1) GOTO 225
C
C      CONVERT INPUT CURVATURES TO ROTATIONS.
C      EI.....MEMBER RIGIDITY.
C      YC1,YC2...YIELD CURVATURES (TEMPARY LOCATIONS).
C
C      DO 219 I=1,NB
C      EI=BCL(I)/(6.*BEF(I))
C      YC1=BYR(I,1)
C      BYR(I,1)=.166/*BCL(I)*((1.-(BCM(I,1)/BYM(I,1))*3)*YC1

```

```

1      + (BCM(I,1)/BYM(I,1))*2*BCM(I,1)/EI)
      YC2=BYR(I,2)
      BYR(I,2)=.1667*BCCL(I)*((1.-(BCM(I,2)/BYM(I,2))*3)*YC2
1      + (BCM(I,2)/BYM(I,2))*2*BCM(I,2)/EI)
      BYR(I,3)=BYR(I,2)
      BYR(I,4)=BYR(I,1)
      V1=BCM(I,1)/BUM(I,1)
      V2=V1*V1
      V3=BYM(I,1)/BUM(I,1)
      V4=(BUM(I,1)-BYM(I,1))/(BUR(I,1)-YC1)*YC1/BYM(I,1)
      BUR(I,1)=.0883*BCCL(I)*((1.2.+V3)*(1.-V3)*(1.+V3*(V4-1.))/V4
1      +V3*(1.+V3)-2.*V1*V2)*YC1/V3+2.*V2*BCM(I,1)/EI)
      V1=BCM(I,2)/BUM(I,2)
      V2=V1*V1
      V3=BYM(I,2)/BUM(I,2)
      V4=(BUM(I,2)-BYM(I,2))/(BUR(I,2)-YC2)*YC2/BYM(I,2)
      BUR(I,2)=.0883*BCCL(I)*((1.2.+V3)*(1.-V3)*(1.+V3*(V4-1.))/V4
1      +V3*(1.+V3)-2.*V1*V2)*YC2/V3+2.*V2*BCM(I,2)/EI)
      BUR(I,3)=BUR(I,2)
      BUR(I,4)=BUR(I,1)
219  CONTINUE
      DO 223 I=1,NC
      EI=CCL(I)/(6.*CF(I))
      YC1=CYR(I,1)
      CYR(I,1)=.1667*CCCL(I)*((1.-(CCM(I,1)/CYM(I,1))*3)*YC1
1      + (CCM(I,1)/CYM(I,1))*2*CCM(I,1)/EI)
      YC2=CYR(I,2)
      CYR(I,2)=.1667*CCCL(I)*((1.-(CCM(I,2)/CYM(I,2))*3)*YC2
1      + (CCM(I,2)/CYM(I,2))*2*CCM(I,2)/EI)
      CYR(I,3)=CYR(I,2)
      CYR(I,4)=CYR(I,1)
      V1=CCM(I,1)/CUM(I,1)
      V2=V1*V1
      V3=CYM(I,1)/CUM(I,1)
      V4=(CUM(I,1)-CYM(I,1))/(CUR(I,1)-YC1)*YC1/CYM(I,1)
      CUR(I,1)=.0883*CCCL(I)*((1.2.+V3)*(1.-V3)*(1.+V3*(V4-1.))/V4
1      +V3*(1.+V3)-2.*V1*V2)*YC1/V3+2.*V2*CCM(I,1)/EI)
      V1=CCM(I,2)/CUM(I,2)
      V2=V1*V1
      V3=CYM(I,2)/CUM(I,2)
      V4=(CUM(I,2)-CYM(I,2))/(CUR(I,2)-YC2)*YC2/CYM(I,2)
      CUR(I,2)=.0883*CCCL(I)*((1.2.+V3)*(1.-V3)*(1.+V3*(V4-1.))/V4
1      +V3*(1.+V3)-2.*V1*V2)*YC2/V3+2.*V2*CCM(I,2)/EI)

```

```

CUR(I,3)=CUR(I,2)
CUR(I,4)=CUR(I,1)
223 CONTINUE
C
C      CORRECT THE YIELD AND ULTIMATE ROTATIONS TO ACCOUNT FOR ANY
C      SHEAR DEFORMATIONS. IT IS ASSUMED THAT THE RATIO OF THE FLEXURAL
C      ROTATION TO THE SUM OF THE FLEXURAL AND SHEAR ROTATION (1+2*BSF(I)
C      ,OR 1+2*CSF(I) ) REMAINS CONSTANT UP TO AND INCLUDING THE ULTIMATE
C      MOMENT.
C
225 DO 229 I=1,NB
V=1.+2.*BSF(I)
DO 227 J=1,4
BYR(I,J)=BYR(I,J)+V
BUR(I,J)=BUR(I,J)+V
227 CONTINUE
229 CONTINUE
DO 233 I=1,NC
V=1.+2.*CSF(I)
DO 231 J=1,4
CYR(I,J)=CYR(I,J)+V
CUR(I,J)=CUR(I,J)+V
231 CONTINUE
233 CONTINUE
C
C      EVALUATE THE JOINT LOADS REFLECTING IF REQUIRED THE EFFECTS
C      OF BOTH BEAM SHEAR DEFORMATION AND FINITE JOINT DIMENSIONS.
C
C      JLD(*)...JOINT LOAD ASSOCIATED WITH STRUCTURAL COORDINATE *.
C
234 IF (FLAG2.EQ.0) GO TO 247
IF (FLAG2.EQ.2) GO TO 237
C
C      USER SUPPLIED FIXED END FORCES. THIS OPTION PERMITS THE INPUT
C      OF THE EFFECTS OF IRREGULAR (OR SIMPLE) LOADING PATTERNS. THE
C      EFFECTS OF ANY SHEAR DEFORMATIONS ARE ASSUMED TO BE INCLUDED.
C
DO 235 I=1,NLB
BN=ID(I)
JLD(NDFB(BN,1))=JLV(NDFB(BN,1))-FEM(BN,1)+FEV(BN,1)*BRZL(BN)
JLD(NDFB(BN,2))=JLV(NDFB(BN,2))-FEM(BN,2)+FEV(BN,2)*BRZR(BN)
JLD(NDFB(BN,3))=JLV(NDFB(BN,3))-FEM(BN,3)+FEV(BN,1)
JLD(NDFB(BN,4))=JLV(NDFB(BN,4))-FEV(BN,2)

```


FEV(BN,1)=0.
FEV(BN,2)=0.
235 CONTINUE
GO TO 247

USER SUPPLIED GRAVITY LOADS.

```

      XL.....LOCATION OF LOAD WRT LFT END.
      XR.....LOCATION OF LOAD WRT RIGHT END.

```

237 CONST=1./12.
D0 245 I=1,NLB
BN=ID(I)
BFS=BSF(BN)
V1=1+2.*BFS
BLC=BCL(BN)
BL2=BLC*BLC
V4=BLC*BFS
UDL=GRL(BN,1)

F4 = (4.+2.*ALFA*DT)/DT	QUAKE
DF4 = (4.+2.*ALFA*DDI)/DDT	QUAKE
F5 = 2.	QUAKE
DF5 = 2.	QUAKE
F6 = 0.	QUAKE
DF6 = 0.	QUAKE
FF6 = BETA/F2	QUAKE
DDF6 = BETA/DF2	QUAKE
C	QUAKE
C	QUAKE
CONSTANTS ASSOCIATED WITH THE INCREMENTAL JOINT VELOCITIES	QUAKE
C	QUAKE
F7 = 0.	QUAKE
DF7 = 0.	QUAKE
F8 = -2.	QUAKE
DF8 = -2.	QUAKE
F9 = 2./DT	QUAKE
DF9 = 2./DDT	QUAKE
C	QUAKE
C	QUAKE
CONSTANTS ASSOCIATED WITH THE INCREMENTAL JOINT ACCELERATIONS	QUAKE
C	QUAKE
F10 = -2.	QUAKE
DF10 = -2.	QUAKE
F11 = 4./DT	QUAKE
DF11 = 4./DDT	QUAKE
F12 = F11/DT	QUAKE
DF12 = DF11/DDT	QUAKE
GOTO 261	QUAKE
C	QUAKE
C	QUAKE
LINEAR ACCELERATION METHOD	QUAKE
C	QUAKE
260 F1 = (6./DT+3.*ALFA)/DT	QUAKE
DF1 = (6./DDT+3.*ALFA)/DDT	QUAKE
F2 = (3.*BETA+DT)/DI	QUAKE
DF2 = (3.*BETA+DDT)/DDT	QUAKE
FF2 = F2/DF2	QUAKE
C	QUAKE
F3 = 3.+5.*ALFA*DT	QUAKE
DF3 = 3.+5.*ALFA*DDT	QUAKE
F4 = (6.+3.*ALFA*DT)/DT	QUAKE
DF4 = (6.+3.*ALFA*DDT)/DDT	QUAKE
F5 = 3.	QUAKE
DF5 = 3.	QUAKE
F6 = .5*DT	QUAKE


```

SK(I,J)=0.
265 CONTINUE
266 CALL FORMSK
IF (POSF.EQ.0.) GO TO 269
IF (NS.EQ.1) GO TO 268
DO 267 I=1,NS1
N1=NDFM(I)
N2=NDFM(I+1)
SKCOPY(N1,1)=SKCOPY(N1,1)+GSK(I)
SKCOPY(N1,N2-N1+1)=SKCOPY(N1,N2-N1+1)+GSK(I+1)
267 CONTINUE
SKCOPY(NDFM(NS),1)=SKCOPY(NDFM(NS),1)+GSK(NS)
GO TO 269
268 SKCOPY(NDFM(1),1)=SKCOPY(NDFM(1),1)+GSK(1)
C
C      UPDATE SKCOPY.
C
269 CALL SKCOPY
C
C      DETERMINE IF MODAL OR STATIC ANALYSIS IS REQUIRED.
C
IF (FLAG9.EQ.1) GO TO 289
IF (FLAG2.EQ.0) GO TO 289
C
C      STAGE 2... STATIC ANALYSIS.
C
TASK MODULE 2... COMPUTE THE JOINT DISPLACEMENTS AND MEMBER
DEFORMATIONS AND FORCES AT THE FLEXIBLE ENDS. REZERO THE JOINT LOAD
VECTOR. CHECK WHETHER THE CRACKING MOMENTS HAVE BEEN EXCEEDED AND
IF SO UPDATE SKCOPY. INITIALIZE THE DYNAMIC FLOOR RESPONSE ARRAYS
WITH THE STATIC ANALYSIS RESULTS AND INITIALIZE THE MINIMUM AND
MAXIMUM RESPONSE VALUES.
C
FLAG4.....STATIC ANALYSIS OR IF DT=DDT (=1), IF DT.NE.DDT (=2).
FLAG5.....FOR CASE WHEN DT.NE.DDT, PRELIMINARY SPRING
EXAMINATION ONLY (=1), COMPLETE EXAMINATION (=2).
FLAG6.....CHANGE IN LINEARITY DETECTED (=1), NO CHANGE (=0).
C
CALL SOLVER
CALL FORCES
IF (EQD.EQ.0.) GO TO 2/1

```

DO 270 I=1,NDF

JLD(I)=0.

270 CONTINUE

FLAG4=1

FLAG5=2

CALL MRHYST

IF (FLAG6.EQ.0) GO TO 271

LEVEL=10000

KEY=2

CALL FORMSK

IF (NDFM(1).LT.LEVEL) LEVEL=NDFM(1)

GO TO 272

271 LEVEL=NDFM(1)

272 CALL SKCOPY

CALL MOTION

CALL MINMAX

C

TASK MODULE 2-2... OUTPUT THE RESULTS OF THE STATIC ANALYSIS.

C

IP=IP+1

WRITE (IW,273) IP,JOB,EQN

273 FORMAT (1H1,120X,'PAGE ',13/27X,20A4/27X,20A4//

1 59X,'** OUTPUT DATA **'//)

WRITE (IW,275)

275 FORMAT (52X,'RESULTS OF STATIC ANALYSIS.'//)

WRITE (IW,277) UNITS(3),UNITS(2)

277 FORMAT (61X,'BEAM RESPONSE'//32X,'BM',7X,'<','>X,MOMENTS ('',A3,

1 '-',A3,''),5X,'>','<','>X,ROTATIONS (RADIANS)'),

2 4X,'>','<','>X,LEFT',13X,'RIGHT',12X,'LEFT',

3 13X,'RIGHT'//)

NLP=18

DO 281 I=1,NB

IF (NLP.LT.MLP) GO TO 278

IP=IP+1

WRITE (IW,273) IP,JOB,EQN

WRITE (IW,275)

WRITE (IW,277) UNITS(3),UNITS(2)

NLP=18

278 WRITE (IW,279) I,(BM(1,J),J=1,2),(BR(1,J),J=1,2),

279 FORMAT (32X,13,5X,'(E12.5,5X)')

NLP=NLP+1

281 CONTINUE

IP=IP+1

QUAKE

QUAKE

QUAKE

QUAKE

QUAKE

QUAKE

QUAKE

QUAKE

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QUAKE

QUAKE

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QUAKE


```

WRITE (IW,273) IP,JOB,EQN
WRITE (IW,275)
WRITE (IW,283) UNITS(3),UNITS(2),UNITS(3),UNITS(2)
283 FORMAT (59X, 'COLUMN RESPONSE',14X,COL,6X,'C',4X, 'MOMENTS (',A3
1 , '-',A3,')',5X,'>',6X,'<',3X, 'ROTATIONS (RADIANS)',
2 4X, '2',6X, 'SWAY (',A3,')',7X, 'AXIAL LOAD',14X, 'NO.',
3 9X, 'LOWER',12X, 'UPPER',12X, 'LOWER',12X, 'UPPER',
4 12X, 'UPPER',12X, '(',A3,')',11X,
NLP=18
00 287 J=1,NC
IF (NLP.LT.NLP) GO10 284
IP=IP+1
WRITE (IW,273) IP,JOB,EQN
WRITE (IW,275)
WRITE (IW,283) UNITS(3),UNITS(2),UNITS(3),UNITS(2)
NLP=18
284 WRITE (IW,285) J,LCM(1,J),J=1,2),CR(1,J),J=1,2),JD(NDFC(1,4)),
1 P(1)
285 FORMAT (14X,I3,5X,6(E12.5,5X))
NLP=NLP+1
287 CONTINUE
IF (EQD.EQ.0.) GO10 513

STAGE 3... DYNAMIC ANALYSIS.

TASK MODULE 3-1... COMPUTE THE NATURAL FREQUENCIES, AND MODE
SHAPES OR PREPARE FOR THE APPLICATION OF SEISMIC LOADING.

289 IP=IP+1
WRITE (IW,273) IP,JOB,EQN
IF (FLAG9.EQ.0) GO10 413
WRITE (IW,291)
291 FORMAT (54X, 'RESULTS OF MODAL ANALYSIS',11X)
NLP=13

DETERMINE THE NATURAL FREQUENCIES AND MODE SHAPE VECTORS.

CONDENSE THE STRUCTURE STIFFNESS MATRIX FROM SIZE NDF*NDF TO
NS*NS BY RECURSIVE ELIMINATION AT SUB MATRIX LEVEL OF ALL DEGREES
OF FREEDOM OTHER THAN THOSE ASSOCIATED WITH SWAY.

```


IF (NK.GT.MBAND) GOTO 303
 A1(I,J)=SK(NJ,NK)
 A1(J,I)=A1(I,J)
 303 CONTINUE

QUAKE
 QUAKE
 QUAKE
 QUAKE

C

DO 307 I=1,LMT
 NI=L1(I)
 DO 307 J=1,NS
 NJ=NDFM(J)
 NIJ=NI-NJ

QUAKE
 QUAKE
 QUAKE
 QUAKE
 QUAKE
 QUAKE
 QUAKE

NK=IABS(NIJ)+1
 IF (NK.GT.MBAND) GOTO 307
 C1(I,J)=SK(NI,NK)
 IF (NIJ.GE.0) C1(I,J)=SK(NJ,NK)
 307 CONTINUE

QUAKE
 QUAKE
 QUAKE
 QUAKE
 QUAKE
 QUAKE
 QUAKE
 QUAKE

C

309 K=LMT-1
 L=LMT+1
 A1(1,L)=1.
 DO 311 I=2,LMT
 A1(I,L)=0.

QUAKE
 QUAKE
 QUAKE
 QUAKE
 QUAKE
 QUAKE
 QUAKE
 QUAKE

311

CONTINUE
 DO 317 I=1,LMT
 DO 313 J=1,LMT
 A1(L,J)=A1(1,J+1)/A1(1,1).

QUAKE
 QUAKE
 QUAKE
 QUAKE
 QUAKE
 QUAKE
 QUAKE
 QUAKE

313

CONTINUE
 DO 315 M=1,K
 DO 315 J=1,LMT

QUAKE
 QUAKE
 QUAKE
 QUAKE
 QUAKE
 QUAKE
 QUAKE
 QUAKE

A1(M,J)=A1(M+1,J+1)-A1(M+1,1)*A1(L,J)
 315 CONTINUE

QUAKE
 QUAKE
 QUAKE
 QUAKE
 QUAKE
 QUAKE
 QUAKE
 QUAKE

DO 317 J=1,LMT
 A1(LMT,J)=A1(L,J)

QUAKE
 QUAKE
 QUAKE
 QUAKE
 QUAKE
 QUAKE
 QUAKE
 QUAKE

317 CONTINUE

QUAKE
 QUAKE
 QUAKE
 QUAKE
 QUAKE
 QUAKE
 QUAKE
 QUAKE

C

DO 321 I=1,LMT
 DO 321 J=1,NS
 SUM=0.

QUAKE
 QUAKE
 QUAKE
 QUAKE
 QUAKE
 QUAKE
 QUAKE
 QUAKE

DO 319 K=1,LMT
 SUM=SUM+A1(I,K)*C1(K,J)
 319 CONTINUE

QUAKE
 QUAKE
 QUAKE
 QUAKE
 QUAKE
 QUAKE
 QUAKE
 QUAKE

TXK(I,J)=SUM

QUAKE
 QUAKE
 QUAKE
 QUAKE
 QUAKE
 QUAKE
 QUAKE
 QUAKE

321 CONTINUE

QUAKE
 QUAKE
 QUAKE
 QUAKE
 QUAKE
 QUAKE
 QUAKE
 QUAKE

C

	D0 325 I=1,NS	QUAKE
	D0 325 J=1,NS	QUAKE
	SUM=0.	QUAKE
	D0 323 K=1,LMT	QUAKE
	SUM=SUM+C1(K,I)*TX(K,J)	QUAKE
323	CONTINUE	QUAKE
	XK(I,J)=XK(I,J)-SUM	QUAKE
325	CONTINUE	QUAKE
	NDT=NDT+NFF(IS)+1	QUAKE
	IF (IS.EQ.NS) GOTO 361	QUAKE
	IS1=IS+1	QUAKE
	N=NTFF(IS1)/2	QUAKE
	D0 327 I=1,N	QUAKE
	L2(I)=NDT+I	QUAKE
327	CONTINUE	QUAKE
	N1=N+1	QUAKE
	MAX=NTFF(IS1)	QUAKE
	D0 329 I=N1,MAX	QUAKE
	L2(I)=NDT+I+1	QUAKE
329	CONTINUE	QUAKE
	D0 331 I=1,MAX	QUAKE
	D0 331 J=1,MAX	QUAKE
	A2(I,J)=0.	QUAKE
331	CONTINUE	QUAKE
	D0 333 I=1,MAX	QUAKE
	N1=L2(I)	QUAKE
	D0 333 J=1,I	QUAKE
	NJ=L2(J)	QUAKE
	NK=N1-NJ+1	QUAKE
	IF (NK.GT.MBAND) GOTO 333	QUAKE
	A2(I,J)=SK(NJ,NK)	QUAKE
	A2(J,I)=A2(I,J)	QUAKE
333	CONTINUE	QUAKE
	D0 335 I=1,LMT	QUAKE
	N1=L1(I)	QUAKE
	D0 335 J=1,MAX	QUAKE
	B1(I,J)=0.	QUAKE
	NJ=L2(J)	QUAKE
	N1J=N1-NJ	QUAKE
	NK=IABS(N1J)+1	QUAKE
	IF (NK.GT.MBAND) GOTO 335	QUAKE
	B1(I,J)=SK(N1,NK)	QUAKE

C

C	335	CONTINUE	IF (NIJ.GE.O) B1(I,J)=SK(NJ,NK)	QUAKE
				QUAKE
				QUAKE
				QUAKE
	D0 337	I=1,MAX		QUAKE
	D0 337	J=1,NS		QUAKE
	C2(I,J)=0.			QUAKE
	337	CONTINUE		QUAKE
	D0 339	I=1,MAX		QUAKE
	NI=L2(I)			QUAKE
	D0 339	J=1,NS		QUAKE
	NJ=NDFH(J)			QUAKE
	NIJ=NI-NJ			QUAKE
	NK=IABS(NIJ)+1			QUAKE
	IF (NK.GT.MBAND) GOTO 339			QUAKE
	C2(I,J)=SK(NI,NK)			QUAKE
	IF (NIJ.GE.O) C2(I,J)=SK(NJ,NK)			QUAKE
	339	CONTINUE		QUAKE
				QUAKE
				QUAKE
C	D0 343	I=1,LMT		QUAKE
	D0 343	J=1,MAX		QUAKE
	SUM=0.			QUAKE
	D0 341	K=1,LMT		QUAKE
	SUM=SUM+A1(I,K)*B1(K,J)			QUAKE
	341	CONTINUE		QUAKE
	TXK(I,J)=SUM			QUAKE
	343	CONTINUE		QUAKE
	D0 347	I=1,MAX		QUAKE
	D0 347	J=1,MAX		QUAKE
	SUM=0.			QUAKE
	D0 345	K=1,LMT		QUAKE
	SUM=SUM+B1(K,I)*TXK(K,J)			QUAKE
	345	CONTINUE		QUAKE
	A2(I,J)=A2(I,J)+SUM			QUAKE
	347	CONTINUE		QUAKE
	D0 351	I=1,LMT		QUAKE
	D0 351	J=1,NS		QUAKE
	SUM=0.			QUAKE
	D0 349	K=1,LMT		QUAKE
	SUM=SUM+A1(I,K)*C1(K,J)			QUAKE
	349	CONTINUE		QUAKE
	TXK(I,J)=SUM			QUAKE
	351	CONTINUE		QUAKE
	D0 355	I=1,MAX		QUAKE
				QUAKE


```

C NLP=19
C BAO.....BASE ACCELERATION AT START OF AN ACCELERATION
C RECORD.
C BAN.....BASE ACCELERATION AT END OF THE RECORD.
C DBA.....BASE ACCELERATION APPLIED OVER RESPONSE CYCLE.
C DOBA.....BASE ACCELERATION APPLIED OVER SUB CYCLE.
C ACC(I).....FLOOR ACCELERATION, I=1, NS INDF IF BETA .NE. 0.).
C VEL(I).....FLOOR VELOCITY, I=1, NS INDF IF BETA .NE. 0.).
C NRC.....NUMBER OF MAJOR RESPONSE CYCLES REQUIRED.
C NCR.....NUMBER OF CYCLES REQUIRED OVER A RECORD.
C NRS.....NUMBER OF RESPONSE SUB-CYCLES PER MAJOR CYCLE.
C NCC.....NUMBER OF CYCLES COMPLETED TO DATE IN A RECORD.
C ACT.....TIME ACCUMULATED FROM FIRST TO CURRENT CYCLE.
C LPR.....COUNTER THAT CONTROLS OUTPUT OF RESPONSE.
C
C FLAG=2
C IF (DT.EQ.DDT) FLAG=1
C NRC=EQD/DT+.01
C NCR=GAP/DT+.01
C NRS=DT/DDT+.01
C FCR=NCR
C FRS=NRS
C
C READ THE INITIAL GROUND ACCELERATION FROM A DISC FILE AND
C APPLY THE SCALE FACTOR. RECORDS ARE ASSUMED TO BE EVENLY SPACED
C IN THE TIME DOMAIN.
C
C READ (4,419) BAO
C 419 FORMAT (F14.8)
C BAO=BAO*ASF
C
C DEFINE THE INITIAL DYNAMIC KINEMATIC CONDITIONS.
C
C NOTE THAT JOINT ACCELERATIONS AND VELOCITIES ASSOCIATED WITH
C UPLIFT AND ROTATIONS ARE INITIALIZED TO ZERO. THIS IS SO BECAUSE
C STIFFNESS DEPENDENT DAMPING IS DEFINED WRT TO THE TOTAL STIFFNESS
C MATRIX AND NOT THE STIFFNESS MATRIX REDUCED TO INCLUDE SWAY
C RELATED COEFFICIENTS ONLY. IT MAY BE LIKELY THEN THAT SHOULD THE
C DAMPING BETA BE NON ZERO, NEWMARK'S BETA (BT) WOULD NEED TO BE
C SET AT 4. (1/4), BECAUSE THE NATURAL FREQUENCIES ASSOCIATED WITH
C ROTATION AND UPLIFT COULD BE MUCH HIGHER THAN THE HIGHEST
C FREQUENCY ASSOCIATED WITH SWAY.
C

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IF (BVO.EQ.0.) GOTO 422	QUAKE
IF (BETA.EQ.0.) GOTO 422	QUAKE
N1=NDFM(I)	QUAKE
IF (NS.EQ.1) GOTO 421	QUAKE
N2=NDFM(2)	QUAKE
ACC(N1)=-BAO+(ALFA+(SKKOPY(N1,1)+SKKOPY(N1,N2-N1+1))*BETA/MASS(1))	QUAKE
1 *BVO	QUAKE
N1=NDFM(NS-1)	QUAKE
N2=NDFM(NS)	QUAKE
ACC(N2)=-BAO+IALFA+(SKKOPY(N1,N2-N1+1)+SKKOPY(N2,1))*BETA/MASS(NS)	QUAKE
1 *BVO	QUAKE
IF (NS.EQ.2) GOTO 425	QUAKE
DO 420 I=2,NS1	QUAKE
N1=NDFM(I-1)	QUAKE
N2=NDFM(I)	QUAKE
N3=NDFM(I+1)	QUAKE
ACC(N2)=-BAO+(ALFA+(SKKOPY(N1,N2-N1+1)+SKKOPY(N2,1)+SKKOPY(N2,N3-N2+1))*BETA/MASS(I))*BVO	QUAKE
1	QUAKE
420 CONTINUE	QUAKE
GOTO 425	QUAKE
421 ACC(N1)=-BAO+(ALFA+SKKOPY(N1,1)*BETA/MASS(1))*BVO	QUAKE
GOTO 425	QUAKE
422 DO 423 I=1,NS	QUAKE
ACC(NDFM(I))=-BAO+ALFA*BVO	QUAKE
423 CONTINUE	QUAKE
425 DO 426 I=1,NS	QUAKE
N1=NDFM(I)	QUAKE
VEL(N1)=-BVO	QUAKE
MASS(I)=MASS(I)/F2	QUAKE
SK(N1,1)=SK(N1,1)+MASS(I)*F1	QUAKE
426 CONTINUE	QUAKE
KEY=2	QUAKE
IF (FLAG2.EQ.0) KEY=1	QUAKE
IF (FLAG2.EQ.0) FLAG6=0	QUAKE
ACT=0.	QUAKE
NCC=NCR	QUAKE
LPR=0	QUAKE
GRF=GRK*F2	QUAKE
C	QUAKE
C TASK MODULE 3-2... EVALUATE THE STRUCTURE RESPONSE OVER THE	QUAKE
C FOLLOWING CYCLES.	QUAKE
C	QUAKE
DO 485 I=1,NRC	QUAKE

C.

DO 431 L=LL,LR	QUAKE
SUM=SUM+SKKOPY(L,K=L+1)*(VEL(L)*FS+ACC(L)*F6)	QUAKE
431 CONTINUE	QUAKE
C	QUAKE
FOR ROW K, ELEMENTS AT AND TO RIGHT OF THE MAIN DIAGONAL.	
C	QUAKE
LR=K+MBAND-1	QUAKE
IF (LR.GT.NDF) LR=NDF	QUAKE
DO 432 L=K,LR	QUAKE
SUM=SUM+SKKOPY(K,L=K+1)*(VEL(L)*FS+ACC(L)*F6)	QUAKE
432 CONTINUE	QUAKE
JLD(K)=JLD(K)+SUM*F6	QUAKE
SUM=0.	QUAKE
433 CONTINUE	QUAKE
C	QUAKE
COMPUTE THE INCREMENTAL JOINT DISPLACEMENTS, AND MEMBER	
C	QUAKE
FORCES, REZERO THE JOINT LOAD VECTOR, ASSESS WHETHER THE ASSUMPTIONS	
C	QUAKE
RE SPRING STIFFNESS WERE VALID.	
C	QUAKE
434 CALL SOLVER	QUAKE
CALL FORCES	QUAKE
DO 435 J=1,NDF	QUAKE
JLD(J)=0.	QUAKE
435 CONTINUE	QUAKE
FLAGS=1	QUAKE
CALL MRHYST	QUAKE
IF (FLAG6.EQ.0) GOTO 467	QUAKE
IF (DT.EQ.DDT) GOTO 443	QUAKE
C	QUAKE
REVERT TO THE BEGINNING OF THE TIME INTERVAL, READ SK FROM	
C	QUAKE
AUXILIARY STORAGE, AND ADD THE INERTIAL TERMS, RESET THE MEMBER	
C	QUAKE
MOMENTS AND ROTATIONS TO THEIR JUST PREVIOUSLY HELD VALUES.	
C	QUAKE
LEVEL=NDFM(1)	QUAKE
CALL SKCOPY	QUAKE
DO 437 J=1,NS	QUAKE
SK(NDFM(J),1)=SK(NDFM(J),1)+MASS(J)*FF2*DF1	QUAKE
437 CONTINUE	QUAKE
DO 439 J=1,NB	QUAKE
BM(J,1)=BM0(J,1)	QUAKE
BM(J,2)=BM0(J,2)	QUAKE
BR(J,1)=BR0(J,1)	QUAKE
BR(J,2)=BR0(J,2)	QUAKE

439 CONTINUE	QUAKE
D0 441 J=1,NC	QUAKE
CM(J,1)=CM(J,1)	QUAKE
CM(J,2)=CM(J,2)	QUAKE
CR(J,1)=CR(J,1)	QUAKE
CR(J,2)=CR(J,2)	QUAKE
P(J)=P(J)	QUAKE
441 CONTINUE	QUAKE
KEY=2	QUAKE
FLAG6=0	QUAKE
C	QUAKE
C	QUAKE
C	QUAKE
443 D0 461 J=1,NRS	QUAKE
IF (DT.EQ.DDT) GOTO 453	QUAKE
IF (FLAG6.EQ.0) GOTO 445	QUAKE
C	QUAKE
D0 444 K=1,NDF	QUAKE
JLD(K)=JLD(K)/DF2	QUAKE
444 CONTINUE	QUAKE
C	QUAKE
445 D0 446 K=1,NS	QUAKE
NK=NDFM(K)	QUAKE
JLD(NK)=JLD(NK)+MASS(K)*F2*(-DDBA*ACC(NK)*DF3+VEL(NK)*DF4)	QUAKE
446 CONTINUE	QUAKE
IF (BETA.EQ.0.) GOTO 451	QUAKE
C	QUAKE
SUM=0.	QUAKE
D0 447 L=1,MBAND	QUAKE
SUM=SUM+SKKOPY(1,L)*(VEL(L)*DF5+ACC(L)*DF6)	QUAKE
447 CONTINUE	QUAKE
JLD(1)=JLD(1)+SUM*DF5	QUAKE
SUM=0.	QUAKE
D0 450 K=2,NDF	QUAKE
LL=K-MBAND+1	QUAKE
IF (LL.LT.1) LL=1	QUAKE
LR=K-1	QUAKE
D0 448 L=LL,LR	QUAKE
SUM=SUM+SKKOPY(L,K=L+1)*(VEL(L)*DF5+ACC(L)*DF6)	QUAKE
448 CONTINUE	QUAKE
LR=K-MBAND-1	QUAKE
IF (LR.GT.NDF) LR=NDF	QUAKE
D0 449 L=K,LR	QUAKE

49	SUM=SUM+SKCOPY(K,L=K+1)*(VEL(L)*DF5+ACC(L)*DF6)	QUAKE
	CONTINUE	QUAKE
	JLD(K)=JLD(K)+SUM*UFF6	QUAKE
	SUM=0.	QUAKE
50	CONTINUE	QUAKE
51	CALL SOLVER	QUAKE
	CALL FORCES	QUAKE
	DO 52 K=1,NDF	QUAKE
	JLD(K)=0.	QUAKE
52	CONTINUE	QUAKE
C		QUAKE
C	ASSUME THAT SK WILL NOT CHANGE.	QUAKE
C		QUAKE
	KEY=4	QUAKE
	FLAGS=2	QUAKE
	CALL MRHYST	QUAKE
	IF (FLAG6.EQ.0) GOTO 59	QUAKE
C		QUAKE
C	UPDATE THE MEMBER STIFFNESS COEFFICIENTS, REBUILD SK AS	QUAKE
C	REQUIRED.	QUAKE
C		QUAKE
53	LEVEL=10000	QUAKE
	KEY=2	QUAKE
	CALL FORMSK	QUAKE
	IF (J.EQ.NRS) GOTO 59	QUAKE
	CALL SKCOPY	QUAKE
	DO 58 K=1,NS	QUAKE
	IF (NDFM(K).LT.LEVEL) GOTO 58	QUAKE
	SK(NDFM(K),1)=SK(NDFM(K),1)+MASS(K)*FF2*DF1	QUAKE
58	CONTINUE	QUAKE
C		QUAKE
C	COMPUTE THE FLOOR VELOCITIES, ACCELERATIONS, SWAYS, RELATIVE	QUAKE
C	SWAYS, INERTIAL FORCES AND SHEARS. UPDATE THE MAXIMUM AND MINIMUM	QUAKE
C	RESPONSE VALUES AS REQUIRED.	QUAKE
C		QUAKE
59	CALL MOTION	QUAKE
	CALL MINMAX	QUAKE
	IF (DT.EQ.DDT) GOTO 63	QUAKE
61	CONTINUE	QUAKE
C		QUAKE
C	END OF THE REIRACE.	QUAKE
C		QUAKE
	IF (NDFM(1).LT.LEVEL) LEVEL=NDFM(1)	QUAKE

[illegible]

[illegible]

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2      , ' ) > , 5X, 1< , 6X, 1SHEAR ( 1, A3, 1 ) , 6X, 1< ,
3      7X , ' MAX, VEL, 8X, 1MAX, ACCLN, 7X, 1NO, 1, 7X, 1MAXIMUM,
4      8X , ' MINIMUM, 8X, 1MAXIMUM, 8X, 1MINIMUM, 8X, 1MAXIMUM,
5      8X , ' MINIMUM, 7X, 1< , 1, A3, 1, 4X, 1, 1, A3, 1, 1, A3,
6      1, 1, A3, 1, 1, 1 )
      NLP=19
      DO 509 I=1, NS
      IF (NLP.LT.MLP) GO TO 506
      IP=IP+1
      WRITE (IW, 273) IP, JDB, EQN
      WRITE (IW, 415)
      WRITE (IW, 505) UNITS(3), UNITS(3), UNITS(2), UNITS(3), UNITS(4),
1      UNITS(3), UNITS(4), UNITS(4)
      NLP=19
      506 WRITE (IW, 507) 1, SMYX(I), SWYN(I), RSX(I), RSMN(I), FVX(I), FVN(I),
1      VELX(I), ACCLX(I)
      507 FORMAT (6X, 13, 4X, 8(E12.5, 3X))
      NLP=NLP+1
      509 CONTINUE
      WRITE (IW, 392)
      WRITE (IW, 511) UNITS(3), UNITS(2), 0.7X, 0.7X
      511 FORMAT (48X, 1STRUCTURE OVERTURNING MOMENTS ( 1, A3, 1, 1, A3, 1 ) /
1      55X, 1MAXIMUM, 9X, 1MINIMUM, // 51X, E12.5, 5X, E12.5)
      513-STOP
      END
      SUBROUTINE SOLVER
C      THIS SUBROUTINE SOLVES THE SIMULTANEOUS LINEAR EQUATIONS FOR
C      JOINT LATERAL, VERTICAL AND ROTATIONAL DISPLACEMENTS BY REDUCING
C      THE UPPER TRIANGLE OF THE STRUCTURE STIFFNESS MATRIX STORED AS SK,
C      & JOINT LOAD VECTOR JLD TO GAUSSIAN FORM USING THE CROUT REDUCTION
C      METHOD AND THEN BACK SUBSTITUTING. COMPUTATIONAL EFFICIENCY IS
C      OBTAINED BY AVOIDING VIRTUALLY ALL NON ZERO MULTIPLICATIONS AND
C      TREATING SK AS A VECTOR. THE METHOD WAS DEVELOPED BY G.H. POWELL
C      AND D.H. MONDKAR AT THE UNIVERSITY OF CALIFORNIA CIRCA 1970. EXTRA
C      EFFICIENCY IS OBTAINED BY DECOMPOSING ONLY THAT REGION OF SK THAT
C      HAS EXPERIENCED CHANGE DURING THE PREVIOUS STEP.
C
C      NROWS.....NUMBER OF ROWS OF RECTANGULAR MATRIX IN WHICH SK IS
C      STORED (7, OR > NDF). NOTE THAT SHOULD THE DIMENSIONS
C      OF SK CHANGE, THEN THE VALUES ASSIGNED TO NROWS MUST
C      CHANGE ACCORDINGLY.
C

```

REAL	JLU,JD,MASS	QUAKE
INTEGER	FLAG4,FLAG5,FLAG6,FLAG7	QUAKE
COMMON /NUMBER/	NB,NBM,NC,NCH,NS,MS1,NDF,MBAND	QUAKE
COMMON /REGION/	LEVEL	QUAKE
COMMON /OPTION/	FLAG4,FLAG5,FLAG6,FLAG7,KEY	QUAKE
COMMON /JOINTS/	JLU(95),JD(95)	QUAKE
COMMON /STIFFS/	SK(1,25),SKKOPY(95,15),GSK(12 1,SWK(3,3),	QUAKE
1	BK(32,3),REF(32 1,BF(32,2),BSF(32 1,	QUAKE
2	CK(42,3),CEFT(42 1,CIF(42,2),CSF(42 1	QUAKE
COMMON /MISCEL/	F7,DF7,F8,DF8,F9,DF9,F10,DF10,F11,DF11,F12,DF12,E,	QUAKE
1	LSC(42 1,LVL(96 1),NDFM(12),MASS(12),ARM(12),GRF	QUAKE
	GOTO (1,11,1,23),KEY	QUAKE
1	NDOFF=NDF-1	QUAKE
	MBAA=MBAND-1	QUAKE
	NROWS=95	QUAKE
	NROWS1=NROWS-1	QUAKE
C		QUAKE
C	LOCATE THE FIRST NON ZERO ELEMENT IN EACH COLUMN OF THE UPPER	QUAKE
C	TRIANGLE OF SK (TREATED AS IF IN SQUARE FORM) FROM COLUMNS J=1,	QUAKE
C	MBAND-1.	QUAKE
C	NADR.....ROW OR COLUMN ELEMENT INDEX.	QUAKE
C	JADR.....ROW OR COLUMN ELEMENT INDEX.	QUAKE
C	LVL(*).....LOCATION OF FIRST NON ZERO ELEMENT-COLUMN **	QUAKE
C		QUAKE
	NADR=1	QUAKE
	LVL(1)=1	QUAKE
	DO 5 N=2,MBAA	QUAKE
	NADR=NADR+NROWS	QUAKE
	JADR=NADR	QUAKE
	N1=N-1	QUAKE
	DO 3 I=1,N1	QUAKE
	IF (SK(JADR).NE.0.) GOTO *	QUAKE
	JADR=JADR-NROWS1	QUAKE
	3 CONTINUE	QUAKE
	I=N1	QUAKE
	* LVL(N)=1	QUAKE
	5 CONTINUE	QUAKE
C		QUAKE
C	LOCATE THE FIRST NON ZERO ELEMENT IN COLUMNS J=MBAND TO NDF,	QUAKE
C	INCLUSIVE.	QUAKE
C		QUAKE
	M=0	QUAKE
	NADR=NROWS+MBAA	QUAKE

DO 9	N=MBAND,NDF	QUAKE
M=M+1		QUAKE
NADR=NADR+1		QUAKE
JADR=NADR		QUAKE
N1=N-1		QUAKE,
DO 7	I=M,N1	QUAKE
IF (SK(JADR).NE.0.)	GOTO 8	QUAKE
JADR=JADR-NROWS1		QUAKE
7	CONTINUE	QUAKE
I=N1		QUAKE
8	LVL(N)=I	QUAKE
9	CONTINUE	QUAKE
C		QUAKE
C		QUAKE
C		QUAKE
	REDUCE THE NON DIAGONAL ELEMENTS.	
11	IF (LEVEL.EQ.1) LEVEL=2	QUAKE
J1=(LEVEL-2)*NROWS		QUAKE
DO 21	J=LEVEL,NDP	QUAKE
IF=LVL(J)		QUAKE
IF1=IF+1		QUAKE
IL=J-1		QUAKE
J1=J1+NROWS		QUAKE
JK=J1+NROWS		QUAKE
IFN=IF+NROWS1		QUAKE
IF (IF1.GT.IL)	GOTO 17	QUAKE
JIA=J1-IFN+1		QUAKE
IKA=JK-IFN		QUAKE
NN=IF1+NROWS=JK		QUAKE
DO 15	I=IF1,IL	QUAKE
KF=LVL(I)		QUAKE
IF (LVL(I).LT.IF)	KF=IF	QUAKE
JKA=JK-KF+NROWS1		QUAKE
AA=0.		QUAKE
DO 13	K=IKA,JKA,NROWS1	QUAKE
AA=AA+SK(K)*SK(NN+K)		QUAKE
13	CONTINUE	QUAKE
SK(JIA)=SK(JIA)-AA		QUAKE
NN=NN+NROWS		QUAKE
IKA=IKA-NROWS1		QUAKE
JIA=JIA-NROWS1		QUAKE
15	CONTINUE	QUAKE
C		QUAKE
C		QUAKE
	REDUCE THE DIAGONAL ELEMENTS.	

C	17 JKA=JK-IFN	QUAKE
	AA=0.	QUAKE
	DO 19 K=IF,IL	QUAKE
	CC=SK(JKA)/SK(K)	QUAKE
	AA=AA+SK(JKA)*CC	QUAKE
	SK(JKA)=CC	QUAKE
	JKA=JKA-NROWS1	QUAKE
	19 CONTINUE	QUAKE
	SK(J)=SK(J)-AA	QUAKE
	21 CONTINUE	QUAKE
	IF (KEY.EQ.3) GOTO 39	QUAKE
C	REDUCE THE JOINT LOAD VECTOR.	QUAKE
C		QUAKE
C		QUAKE
	23 DO 25 N=1,NDF	QUAKE
	IF (JLD(N).NE.0.) GOTO 27	QUAKE
	25 CONTINUE	QUAKE
	N=NDF	QUAKE
	27 N1=N+1	QUAKE
	NADR=N1*NROWS	QUAKE
	DO 31 I=N1,NDF	QUAKE
	KF=LVL(I)	QUAKE
	IF (LVL(I).LT.N) K+=N	QUAKE
	KL=I-1	QUAKE
	JADR=NADR-KF*NROWS1	QUAKE
	BB=0.	QUAKE
	DO 29 K=KF,KL	QUAKE
	BB=BB+SK(JADR)*JLD(K)	QUAKE
	JADR=JADR-NROWS1	QUAKE
	29 CONTINUE	QUAKE
	JLD(I)=JLD(I)-BB	QUAKE
	NADR=NADR+NROWS	QUAKE
	31 CONTINUE	QUAKE
	DO 33 I=N,NDF	QUAKE
	JLD(I)=JLD(I)/SK(I)	QUAKE
	33 CONTINUE	QUAKE
C		QUAKE
C	PERFORM THE BACK SUBSTITUTION.	QUAKE
C		QUAKE
C		QUAKE
C	JD(*).....JOINT DISPLACEMENT	QUAKE
C	NN=NDF	QUAKE

QUAKE
QUAKE
QUAKE
QUAKE

GUAKE

QUAKE

QUAKE

GUAKE

QUAKE

QUAKE

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GLAXE

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GUAKE

QUAKE
QUAKE

GUAKE
GUAKE

GUANE
QUAKE

QUAKE
QUAKE

11 CONTINUE

12 RETURN

END

SUBROUTINE FORCES

THIS SUBROUTINE COMPUTES THE MEMBER MOMENTS AND ROTATIONS AT THE ENDS OF THE FLEXIBLE SECTION, AND THE COLUMN AXIAL LOADS AT THE JOINTS.

WITH RESPECT TO THE FLEXIBLE PORTION OF A MEMBER I,

DRJ.....INCREMENTAL ROTATION DUE TO JOINT TRANSLATION.

DRL.....INCREMENTAL ROTATION-LEFT END.

DRR.....INCREMENTAL ROTATION-RIGHT END.

DRL.....INCREMENTAL ROTATION-LOWER END.

DRL.....INCREMENTAL ROTATION-UPPER END.

DML.....INCREMENTAL MOMENT-LEFT END.

DMR.....INCREMENTAL MOMENT-RIGHT END.

DML.....INCREMENTAL MOMENT-LOWER END.

DMU.....INCREMENTAL MOMENT-UPPER END.

BM(1,1).....TOTAL BEAM MOMENT AT END OF STEP-LEFT END.

BM(1,2).....TOTAL BEAM MOMENT AT END OF STEP-RIGHT END.

BR(1,1).....TOTAL BEAM ROTATION AT END OF STEP-LEFT END.

BR(1,2).....TOTAL BEAM ROTATION AT END OF STEP-RIGHT END.

CM(1,1).....TOTAL COLUMN MOMENT AT END OF STEP-LOWER END.

CM(1,2).....TOTAL COLUMN MOMENT AT END OF STEP-UPPER END.

CR(1,1).....TOTAL COLUMN ROTATION AT END OF STEP-LOWER END.

CR(1,2).....TOTAL COLUMN ROTATION AT END OF TIME STEP-UPPER END

P(1).....TOTAL AXIAL LOAD AT END OF STEP.

REAL JLD,JU,MASS

COMMON /NUMBER/ NB,NBM,NC,NCM,NS,NS1,NDF,MBAND

COMMON /JOINTS/ JLD(95),JUD(95)

COMMON /STIFFS/ SKI(25,15),SKKEPY(95,15),GSK(12),SWK(3,3),

BKI(32,3),BEF(32),BTF(32,2),BSF(32),

CKI(42,3),CEF(42),CTF(42,2),CSF(42)

COMMON /ACTION/ BMI(32,2),BMO(32,2),BRI(32,2),BRO(32,2),

CM(42,2),CMO(42,2),CRO(42,2),CRO(42,2),

P(42),POI(42),OTM

COMMON /MEMBER/ NOP(32,4),BRZL(32),BCL(32),BRZR(32),LBM(32),

NOPCI(42,6),CRZL(42),CCL(42),CRZU(42),LCM(42),

CAI(42)

COMMON /MISCEL/ F7,DF7,F8,DF8,F9,DF9,F10,DF10,F11,DF11,F12,DF12,E,

LSC(42),LYLI(96),NDFMI(12),MASS(12),ARM(12),3RF

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TEST FOR COLUMNS ATTACHED TO THE FOUNDATION.

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COMMON /OPTION/	FLAG*, FLAG5, FLAG6, FLAG7, KEY	QUAKE
COMMON /KDAMPG/	BEIA	QUAKE
COMMON /JOINTS/	JLU(95), JDI(95)	QUAKE
COMMON /FLOORS/	VEL(95), ACC(95), F(12), FV(12), SWAY(12), RSWY(12)	QUAKE
COMMON /STIFFS/	SKI(95), SKKUPY(95), GSK(12), SWK(3,3), BK(32,3), BEF(32), BIF(32,2), BSF(32), CK(42,3), CEF(42), CIF(42,2), CSF(42)	QUAKE
COMMON /ACTION/	BM(32,2), BMO(32,2), BR(32,2), BRO(32,2), CM(42,2), CMO(42,2), CR(42,2), CRO(42,2), P(42), PO(42), OTH	QUAKE
COMMON /MEMBER/	NDP3(32,4), BRZL(32), BCL(32), BRZR(32), LBM(32), NDP(42,6), CRZL(42), CCL(42), CRZU(42), LCM(42), CA(42)	QUAKE
COMMON /MISCEL/	F7, DF7, F8, DF8, F9, DF9, F10, DF10, F11, DF11, F12, DF12, E, LSL(42), LVL(96), NDFM(12), MASS(12), ARM(12), GRF	QUAKE
IF (KEY.EQ.1)	GOTO 8	QUAKE
IF (FLAG5.EQ.2)	GOTO 4	QUAKE
IF (BETA.EQ.0.)	GOTO 2	QUAKE
RESPONSE FOR MAJOR CYCLE.		QUAKE
DO 1 I=1, NDF		QUAKE
DVL=ACC(I)*F7+VEL(I)*F8+JD(I)*F9		QUAKE
ACC(I)=ACC(I)+ACC(I)*F10+VEL(I)*F11+JD(I)*F12		QUAKE
VEL(I)=VEL(I)+DVL		QUAKE
1 CONTINUE		QUAKE
GOTO 8		QUAKE
DO 3 I=1, NS		QUAKE
J=NDFM(I)		QUAKE
DVL=ACC(J)*F7+VEL(J)*F8+JD(J)*F9		QUAKE
ACC(J)=ACC(J)+ACC(J)*F10+VEL(J)*F11+JD(J)*F12		QUAKE
VEL(J)=VEL(J)+DVL		QUAKE
3 CONTINUE		QUAKE
GOTO 8		QUAKE
RESPONSE FOR SUB CYCLE.		QUAKE
IF (BETA.EQ.0.)	GOTO 6	QUAKE
DO 5 I=1, NDF		QUAKE
DVL=ACC(I)*DF7+VEL(I)*DF8+JD(I)*DF9		QUAKE
ACC(I)=ACC(I)+ACC(I)*DF10+VEL(I)*DF11+JD(I)*DF12		QUAKE
VEL(I)=VEL(I)+DVL		QUAKE
5 CONTINUE		QUAKE

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      6  DO 7 I=1,NS
      7  J=NDFM(I)
      8  DVL=ACC(J)*DF7+VEL(J)*DF8+JD(J)*DF9
      9  ACC(J)=ACC(J)+ACC(J)*DF10+VEL(J)*DF11+JD(J)*DF12
      10 VEL(J)=VEL(J)+DVL
      11 CONTINUE
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      13 RESPONSE FOR EITHER CYCLE.
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C	BMN(I,1)...	MINIMUM BENDING MOMENT-LEFT END.	QUAKE
C	BMX(I,2)...	MAXIMUM BENDING MOMENT-RIGHT END.	QUAKE
C	BMN(I,2)...	MINIMUM BENDING MOMENT-RIGHT END.	QUAKE
C	BRX(I,1)...	MAXIMUM ROTATION-LEFT END.	QUAKE
C	BRN(I,1)...	MINIMUM ROTATION-LEFT END.	QUAKE
C	BRX(I,2)...	MAXIMUM ROTATION-RIGHT END.	QUAKE
C	BRN(I,2)...	MINIMUM ROTATION-RIGHT END.	QUAKE
C	FOR COLUMN 1)		
C	CMX(I,1)...	BENDING MOMENT-LOWER END.	QUAKE
C	CMN(I,1)...	BENDING MOMENT-LOWER END.	QUAKE
C	CMX(I,2)...	BENDING MOMENT-UPPER END.	QUAKE
C	CMN(I,2)...	BENDING MOMENT-UPPER END.	QUAKE
C	CRX(I,1)...	ROTATION-LOWER END.	QUAKE
C	CRN(I,1)...	ROTATION-LOWER END.	QUAKE
C	CRX(I,2)...	ROTATION-UPPER END.	QUAKE
C	CRN(I,2)...	ROTATION-UPPER END.	QUAKE
C	PMX(I)...	MAXIMUM COMPRESSIVE FORCE.	QUAKE
C	PMN(I)...	MINIMUM COMPRESSIVE FORCE (TENSION IF -).	QUAKE
C	FOR STORY 1)		
C	SWYX(I)...	MAXIMUM SWAY.	QUAKE
C	SWYN(I)...	MINIMUM SWAY.	QUAKE
C	RSWX(I)...	MAXIMUM RELATIVE SWAY.	QUAKE
C	RSWN(I)...	MINIMUM RELATIVE SWAY.	QUAKE
C	VELX(I)...	MAXIMUM VELOCITY.	QUAKE
C	ACCX(I)...	MAXIMUM ACCELERATION.	QUAKE
C	FVX(I)...	MAXIMUM SHEAR.	QUAKE
C	FVN(I)...	MINIMUM SHEAR.	QUAKE
C	OTMX...	MAXIMUM OVERTURNING MOMENT.	QUAKE
C	OTMN...	MINIMUM OVERTURNING MOMENT.	QUAKE
C	REAL MASS		
C	COMMON /NUMBER/	NB,NBM,NC,NCM,NS,NS1,NDF,MBAND	QUAKE
C	COMMON /FLOORS/	VEL(95),ACC(95),FF(12),FV(12),SWAY(12),RSWY(12)	QUAKE
C	COMMON /ACTION/	BM(32,2),BMO(32,2),BR(32,2),BRO(32,2),	QUAKE
1		CM(42,2),CMO(42,2),CR(42,2),CRO(42,2),	QUAKE
2		P(42,2),PO(42,2),OTM	QUAKE
C	COMMON /ENVLP/	BMX(32,2),BMN(32,2),BRX(32,2),BRN(32,2),	QUAKE
1		CMX(42,2),CMN(42,2),CRX(42,2),CRN(42,2),	QUAKE
2		PMX(42,2),PMN(42,2),VELX(12),ACCX(12),FVX(12),	QUAKE
3		FVN(12),SWYX(12),SWYN(12),RSWX(12),RSWN(12),	QUAKE
4		OTMX,OTMN	QUAKE

C	BEGINNING OF THE STEP AND A COMPLETE UPDATE IS APPLIED TO THE SUB	QUAKE
C	INTERVALS OF THE STEP.	QUAKE
C	THE CHOICE OF EMPLOYING SPRINGS IN ASSOCIATION WITH	QUAKE
C	ANTISYMMETRIC FLEXURE RATHER THAN OTHER COMPETING MODELS IS BASED	QUAKE
C	ON THE CONCEPTUAL SIMPLICITY, COMPUTATIONAL EFFICIENCY AND	QUAKE
C	REASONABLE REPRESENTATION OF REAL BEHAVIOUR OF MOST BEAM, AND	QUAKE
C	COLUMN COMPONENTS OF A FRAME SUBJECTED TO SEISMIC LOADING.	QUAKE
C		QUAKE
C	COMPUTE THE UNBALANCED JOINT LOADS (EQUILIBRIUM CORRECTIONS)	QUAKE
C	ARISING FROM THE CURRENT CYCLE AND ADD THEM TO THE INCREMENTAL	QUAKE
C	JOINT LOAD VECTOR. THESE IMBALANCES ARE ENTIRELY NUMERICAL	QUAKE
C	RESULTING ONLY WHEN THE EQUATION OF MOTION IS INTEGRATED	QUAKE
C	NUMERICALLY. THEY OCCUR FOR TWO REASONS:	QUAKE
C	(1) WHEN THERE IS A CHANGE OF SPRING STIFFNESS (YIELDING OR	QUAKE
C	UNLOADING) WITHIN A CYCLE, THE FORCES PREDICTED BY ASSUMING	QUAKE
C	CONSTANT STIFFNESS DIFFER FROM THE ACTUAL VALUES, ASSUMING HOWEVER	QUAKE
C	THAT THE PREDICTED DISPLACEMENTS ARE CORRECT, THEN THE FORCES HAVE	QUAKE
C	TO BE ADJUSTED, AND THEIR ALGEBRAIC SUM AT A JOINT IS NOT ZERO.	QUAKE
C	(2) IF (1) OCCURS AND IF DAMPING IS INCLUDED, THEN REVISION OF THE	QUAKE
C	STRUCTURE STIFFNESS MATRIX AT THE END OF THE INTERVAL PRODUCES A	QUAKE
C	DISCONTINUITY IN THE STIFFNESS DEPENDENT DAMPING TERM (BETA*SK)	QUAKE
C	FOR THE NEXT STEP.	QUAKE
C	IN BOTH CASES THE CORRECTIONS SHOULD BE ADDED AS NEGATIVE	QUAKE
C	JOINT-LOADS TO THE NEXT INTEGRATION STEP.	QUAKE
C	THE CORRECTIONS MANIFEST THEMSELVES, IN THREE WAYS:	QUAKE
C	(A) UNBALANCED JOINT MOMENTS,	QUAKE
C	(B) FLOOR-FLOOR SHEARS,	QUAKE
C	(C) BAY-BAY SHEARS (WHEN COLUMNS AXIALLY DEFORMABLE).	QUAKE
C	ONLY CORRECTIONS OF TYPE 1A AND 1B ARE CONSIDERED IN THIS	QUAKE
C	PROGRAM.	QUAKE
C		QUAKE
C	BEAM SPRINGS:	QUAKE
C	B16(1,1) INTERCEPT WITH ROTATION AXIS FOR STEP 6-LEFT END.	QUAKE
C	B16(1,2) INTERCEPT WITH ROTATION AXIS FOR STEP 6-RIGHT END.	QUAKE
C	B18(1,1) INTERCEPT WITH ROTATION AXIS FOR STEP 8-LEFT END.	QUAKE
C	B18(1,2) INTERCEPT WITH ROTATION AXIS FOR STEP 8-RIGHT END.	QUAKE
C	B110(1,1) INTERCEPT WITH ROTATION AXIS FOR STEP 10-LEFT END.	QUAKE
C	B110(1,2) INTERCEPT WITH ROTATION AXIS FOR STEP 10-RIGHT END.	QUAKE
C	B112(1,1) INTERCEPT WITH ROTATION AXIS FOR STEP 12-LEFT END.	QUAKE
C	B112(1,2) INTERCEPT WITH ROTATION AXIS FOR STEP 12-RIGHT END.	QUAKE
C	BUMP(1,1) UNLOADING FORCE LEVEL ON PRIMARY CURVE-LEFT END.	QUAKE
C	BUMP(1,2) UNLOADING FORCE LEVEL ON PRIMARY CURVE-LEFT END.	QUAKE
C	BUMP(1,3) UNLOADING FORCE LEVEL ON PRIMARY CURVE-RIGHT END.	QUAKE

C	BUMP(I,4):UNLOADING FORCE LEVEL ON PRIMARY CURVE-RIGHT END.	QUAKE
C	BUM7(I,1):UNLOADING FORCE LEVEL FROM STEP 6-LEFT END.	QUAKE
C	BUM7(I,2):UNLOADING FORCE LEVEL FROM STEP 6-RIGHT END.	QUAKE
C	BUM9(I,1):UNLOADING FORCE LEVEL FROM STEP 8-LEFT END.	QUAKE
C	BUM9(I,2):UNLOADING FORCE LEVEL FROM STEP 8-RIGHT END.	QUAKE
C	BUM11(I,1):UNLOADING FORCE LEVEL FROM STEP 10-LEFT END.	QUAKE
C	BUM11(I,2):UNLOADING FORCE LEVEL FROM STEP 10-RIGHT END.	QUAKE
C	BUM13(I,1):UNLOADING FORCE LEVEL FROM STEP 12-LEFT END.	QUAKE
C	BUM13(I,2):UNLOADING FORCE LEVEL FROM STEP 12-RIGHT END.	QUAKE
C	BURP(I,1):UNLOADING ROTATION ON PRIMARY CURVE-LEFT END.	QUAKE
C	BURP(I,2):UNLOADING ROTATION ON PRIMARY CURVE-LEFT END.	QUAKE
C	BURP(I,3):UNLOADING ROTATION ON PRIMARY CURVE-RIGHT END.	QUAKE
C	BURP(I,4):UNLOADING ROTATION ON PRIMARY CURVE-RIGHT END.	QUAKE
C	BUR7(I,1):UNLOADING ROTATION FROM STEP 6-LEFT END.	QUAKE
C	BUR7(I,2):UNLOADING ROTATION FROM STEP 6-RIGHT END.	QUAKE
C	BUR9(I,1):UNLOADING ROTATION FROM STEP 8-LEFT END.	QUAKE
C	BUR9(I,2):UNLOADING ROTATION FROM STEP 8-RIGHT END.	QUAKE
C	BUR11(I,1):UNLOADING ROTATION FROM STEP 10-LEFT END.	QUAKE
C	BUR11(I,2):UNLOADING ROTATION FROM STEP 10-RIGHT END.	QUAKE
C	BUR13(I,1):UNLOADING ROTATION FROM STEP 12-LEFT END.	QUAKE
C	BUR13(I,2):UNLOADING ROTATION FROM STEP 12-RIGHT END.	QUAKE
C	BF1(I,1):UNLOADING FLEXIBILITY FROM PRIMARY CURVE-LEFT END.	QUAKE
C	BF1(I,2):UNLOADING FLEXIBILITY FROM PRIMARY CURVE-RIGHT END.	QUAKE
C	BF2(I,1):REVERSING FLEXIBILITY-LEFT END.	QUAKE
C	BF2(I,2):REVERSING FLEXIBILITY-RIGHT END.	QUAKE
C	BSR(I,1):CURRENT ROTATION-LEFT END.	QUAKE
C	BSR(I,2):CURRENT ROTATION-RIGHT END.	QUAKE
C	LBR(I,1):CURRENT CONTROLLING RULE-LEFT END.	QUAKE
C	LBR(I,2):CURRENT CONTROLLING RULE-RIGHT END.	QUAKE
C	COLUMN SPRINGS:	QUAKE
C	CI6(I,1):INTERCEPT WITH ROTATION AXIS FOR STEP 6-LOWER END.	QUAKE
C	CI6(I,2):INTERCEPT WITH ROTATION AXIS FOR STEP 6-UPPER END.	QUAKE
C	CI8(I,1):INTERCEPT WITH ROTATION AXIS FOR STEP 8-LOWER END.	QUAKE
C	CI8(I,2):INTERCEPT WITH ROTATION AXIS FOR STEP 8-UPPER END.	QUAKE
C	CI10(I,1):INTERCEPT WITH ROTATION AXIS FOR STEP 10-LOWER END.	QUAKE
C	CI10(I,2):INTERCEPT WITH ROTATION AXIS FOR STEP 10-UPPER END.	QUAKE
C	CI12(I,1):INTERCEPT WITH ROTATION AXIS FOR STEP 12-LOWER END.	QUAKE
C	CI12(I,2):INTERCEPT WITH ROTATION AXIS FOR STEP 12-UPPER END.	QUAKE
C	CUMP(I,1):UNLOADING FORCE LEVEL ON PRIMARY CURVE-LOWER END.	QUAKE
C	CUMP(I,2):UNLOADING FORCE LEVEL ON PRIMARY CURVE-LOWER END.	QUAKE
C	CUMP(I,3):UNLOADING FORCE LEVEL ON PRIMARY CURVE-UPPER END.	QUAKE
C	CUMP(I,4):UNLOADING FORCE LEVEL ON PRIMARY CURVE-UPPER END.	QUAKE

C	CUM7(I,1).	UNLOADING FORCE	LEVEL FROM STEP 6-LOWER END.	QUAKE
C	CUM7(I,2).	UNLOADING FORCE	LEVEL FROM STEP 6-UPPER END.	QUAKE
C	CUM9(I,1).	UNLOADING FORCE	LEVEL FROM STEP 8-LOWER END.	QUAKE
C	CUM9(I,2).	UNLOADING FORCE	LEVEL FROM STEP 8-UPPER END.	QUAKE
C	CUM11(I,1).	UNLOADING FORCE	LEVEL FROM STEP 10-LOWER END.	QUAKE
C	CUM11(I,2).	UNLOADING FORCE	LEVEL FROM STEP 10-UPPER END.	QUAKE
C	CUM13(I,1).	UNLOADING FORCE	LEVEL FROM STEP 12-LOWER END.	QUAKE
C	CUM13(I,2).	UNLOADING FORCE	LEVEL FROM STEP 12-UPPER END.	QUAKE
C	CURP(I,1).	UNLOADING ROTATION	ON PRIMARY CURVE-LOWER END.	QUAKE
C	CURP(I,2).	UNLOADING ROTATION	ON PRIMARY CURVE-UPPER END.	QUAKE
C	CURP(I,3).	UNLOADING ROTATION	ON PRIMARY CURVE-RIGHT END.	QUAKE
C	CURP(I,4).	UNLOADING ROTATION	ON PRIMARY CURVE-RIGHT END.	QUAKE
C	CUR7(I,1).	UNLOADING ROTATION	FROM STEP 6-LOWER END.	QUAKE
C	CUR7(I,2).	UNLOADING ROTATION	FROM STEP 6-UPPER END.	QUAKE
C	CUR9(I,1).	UNLOADING ROTATION	FROM STEP 8-LOWER END.	QUAKE
C	CUR9(I,2).	UNLOADING ROTATION	FROM STEP 8-UPPER END.	QUAKE
C	CUR11(I,1).	UNLOADING ROTATION	FROM STEP 10-LOWER END.	QUAKE
C	CUR11(I,2).	UNLOADING ROTATION	FROM STEP 10-UPPER END.	QUAKE
C	CUR13(I,1).	UNLOADING ROTATION	FROM STEP 12-LOWER END.	QUAKE
C	CUR13(I,2).	UNLOADING ROTATION	FROM STEP 12-UPPER END.	QUAKE
C	CF1(I,1).	UNLOADING FLEXIBILITY	FROM PRIMARY CURVE-LOWER END.	QUAKE
C	CF1(I,2).	UNLOADING FLEXIBILITY	FROM PRIMARY CURVE-UPPER END.	QUAKE
C	CF2(I,1).	REVERSING FLEXIBILITY	-LOWER END.	QUAKE
C	CF2(I,2).	REVERSING FLEXIBILITY	-UPPER END.	QUAKE
C	CSR(I,1).	CURRENT ROTATION	-LOWER END.	QUAKE
C	CSR(I,2).	CURRENT ROTATION	-UPPER END.	QUAKE
C	LCR(I,1).	CURRENT CONTROLLING	RULE-LOWER END.	QUAKE
C	LCR(I,2).	CURRENT CONTROLLING	RULE-UPPER END.	QUAKE
C	K.....	INDEX ASSOCIATED WITH SPRING'S POSITIVE PROPERTIES.		QUAKE
C	CHANGE.....	NO CHANGE IN LINEARITY (-0.1) CHANGE (-1).		QUAKE
C	BCF.....	CURRENT TOTAL BEAM SPRING FLEXIBILITY.		QUAKE
C	CCF.....	CURRENT TOTAL COLUMN SPRING FLEXIBILITY.		QUAKE
C	BRC.....	CURRENT TOTAL BEAM SPRING ROTATION.		QUAKE
C	CRC.....	CURRENT TOTAL COLUMN SPRING ROTATION.		QUAKE
C	L.....	INDEX INDICATING SIGN OF SPRING'S FORMER PROPERTIES.		QUAKE
C	LOP.....	INDEX INDICATING OPPOSITE SIGN TO L.		QUAKE
C	SIGN.....	SIGN OF SPRING'S FORMER PROPERTIES.		QUAKE
C	FYC.....	INELASTIC FLEXIBILITY ASSOCIATED WITH RULE 2.		QUAKE
C	FUY.....	INELASTIC FLEXIBILITY ASSOCIATED WITH RULE 3.		QUAKE
C	ACOR.....	GENERALIZED MEMBER CORRECTION FORCE.		QUAKE
C	RULE.....	LOCAL VARIABLE CONTAINING STIFFNESS REGIME.		QUAKE
C	FLAG10.....	INDICATES WHETHER BEAM SPRINGS UNDER PRELIMINARY		QUAKE

C	EXAMINATION (1) OR COLUMN SPRINGS (0).	QUAKE
C	FLAG11...INDICATES WHETHER FORCE LEVEL ON BEAM SPRINGS TO BE	QUAKE
C	CORRECTED (1) OR COLUMN SPRINGS (2).	QUAKE
C	FLAG12...INDICATES YIELDING OR CRACKING (1) OR NO YIELDING OR	QUAKE
C	CRACKING (0).	QUAKE
C		QUAKE
	INTEGER CHANGE/RULE, FLAG10, FLAG11, FLAG12	QUAKE
	INTEGER FLAG4, FLAG5, FLAG6, FLAG7	QUAKE
	COMMON /NUMBER/ NB, NBM, NC, NCM, NS, NS1, NDF, MBAND	QUAKE
	COMMON /OPTION/ FLAG4, FLAG5, FLAG6, FLAG7, KEY	QUAKE
	COMMON /SWAPER/ L, LOP	QUAKE
	COMMON /PARAMS/ I, J, NU, N1, N2, N3, N4, FLAG11, BM1, BRZ, BLC, CM1, CRZ, CLC,	QUAKE
	CON	QUAKE
1	COMMON /HINGES/ IM, IP, NLP, MLP, J08(20), EQN(20), UNITS(4), ACT, DT, DOT,	QUAKE
1	JJ, NDNHGE	QUAKE
1	COMMON /STIFFS/ SKI 95, 151, SKKOPY, 95, 151, GSK112, 1, SMK1, 3, 3,	QUAKE
2	BK1 32, 31, BEF132, 1, BIF, 32, 21, BSF(32, 1),	QUAKE
2	CK1 42, 31, CEF142, 1, CIF, 42, 21, CSF(42, 1)	QUAKE
	COMMON /ACTION/ BM1 32, 21, BMQ1 32, 21, BR1 32, 21, BRO1 32, 21, 0	QUAKE
1	CM1 42, 21, CMQ1 42, 21, CR1 42, 21, CRO1 42, 21,	QUAKE
2	P1 42, 1, P01 42, 1, BTM	QUAKE
	COMMON /MEMBER/ NDPB1 32, 41, BRZL132, 1, BCL132, 1, BRZR132, 1, LBM132, 1,	QUAKE
1	NDPCL 42, 61, CRZL142, 1, CCL142, 1, CRZU142, 1, LCM142, 1,	QUAKE
2	CA142, 1	QUAKE
	COMMON /SPRING/ BCM1 32, 41, BCR1 32, 41, BYM1 32, 41, BYR1 32, 41,	QUAKE
1	BUM1 32, 41, BUR1 32, 41, BUMP1 32, 41, BURP1 32, 41,	QUAKE
2	BUM71 32, 21, BUR71 32, 21, BUM91 32, 21, BUR91 32, 21,	QUAKE
3	BUM11 32, 21, BUR11 32, 21, BUM131 32, 21,	QUAKE
4	BUM131 32, 21, B161 32, 21, B181 32, 21, B1101 32, 21,	QUAKE
5	B1121 32, 21, BFI1 32, 21, BF21 32, 21, BSR1 32, 21,	QUAKE
6	LBM1 32, 21,	QUAKE
7	CCM1 42, 41, CCR1 42, 41, CYM1 42, 41, CYR1 42, 41,	QUAKE
8	CUM1 42, 41, CUR1 42, 41, CUMP1 42, 41, CURP1 42, 41,	QUAKE
9	CUM71 42, 21, CUR71 42, 21, CUM91 42, 21, CUR91 42, 21,	QUAKE
1	CUM111 42, 21, CUR111 42, 21, CUM131 42, 21,	QUAKE
2	CUM131 42, 21, C161 42, 21, C181 42, 21, C1101 42, 21,	QUAKE
3	C1121 42, 21, CF11 42, 21, CF21 42, 21, CSR1 42, 21,	QUAKE
4	LCM1 42, 21, UF, RF	QUAKE
	FLAG6=0	QUAKE
	FLAG10=1	QUAKE
	FLAG12=0	QUAKE
	NBM=0	QUAKE
	NCM=0	QUAKE

DO 93 I=1,NB	QUAKE
K=1	QUAKE
BLC=BCL(I)	QUAKE
BRZ=BRZL(I)	QUAKE
CHANGE=0	QUAKE
N1=NDFB(I,1)	QUAKE
N2=NDFB(I,2)	QUAKE
DO 91 J=1,2	QUAKE
BM1=BM(I,J)	QUAKE
BM0=BM0(I,J)	QUAKE
DBM=BM1-BM0	QUAKE
BCF=BEF(I)+BIF(I,J)	QUAKE
BSR(I,J)=BSR(I,J)+DBM*BCF	QUAKE
BRC=BSR(I,J)	QUAKE
NJ=NDFB(I,J)	QUAKE
L=K	QUAKE
LOP=K+1	QUAKE
IF (BM1.LT.0. .AND. BM0.EQ.0.) CALL SWITCH	QUAKE
IF (BM0.LT.0.) CALL SWITCH	QUAKE
RULE=LBR(I,J)	QUAKE
GOTO (1,5,13,15,19,31,35,41,43,53,55,63,65,71,77,85),RULE	QUAKE
C	QUAKE
C	QUAKE
C	QUAKE
1 IF (BRC(I,L)=ABS(BRC(I,L)).GT.0.) GOTO 89	QUAKE
IF (FLAG.EQ.2.AND.FLAG5.EQ.1) GOTO 189	QUAKE
3 IF ((BYR(I,L)=ABS(BRC(I,L)).LE.0.) GOTO 7	QUAKE
IF (LBR(I,J).EQ.2) GOTO 89	QUAKE
IF (FLAG.EQ.2.AND.FLAG5.EQ.1) GOTO 189	QUAKE
CHANGE=1	QUAKE
LBR(I,J)=2	QUAKE
SIGN=BRC/ABS(BRC)	QUAKE
BCF=(BYR(I,L)-BRC(I,L))/(BYM(I,L)-BCM(I,L))	QUAKE
BM(I,J)=(BRC-SIGN*BCH(I,L))/BCF+SIGN*BCM(I,L)	QUAKE
BIF(I,J)=BCF-BEF(I)	QUAKE
FLAG11=1	QUAKE
CALL EGMCOR	QUAKE
GOTO 89	QUAKE
C	QUAKE
C	QUAKE
C	QUAKE
5 IF (DBM-BM0.GT.0.) GOTO 3	QUAKE
IF (FLAG.EQ.2.AND.FLAG5.EQ.1) GOTO 189	QUAKE

RULE 1: LOADING WITH UNCRACKED REAM.

RULE 2: LOADING ON PRIMARY CURVE AFTER CRACKING.

```

      GOTO 9
      IF (FLAG4.EQ.2.AND.FLAG5.EQ.1) GOTO 189
      CHANGE=1
      SIGN=BRC/ABS(BRC)
      BCF=(BUR(I,L)-BYR(I,L))/(BUM(I,L)-BYM(I,L))
      BM(I,J)=(BRC-SIGN*BYR(I,L))/BCF+SIGN*BYM(I,L)
      LBR(I,J)=3
      BIF(I,J)=BCF-BEF(I)
      FLAG11=1
      CALL EQMCOR
      IF (NOHNGE.EQ.0) GOTO 10
      IF (NLP.LT.NLP) GOTO 8
      IP=IP+1
      WRITE (IW,239) IP,JOB,EQN
      WRITE (IW,241)
      WRITE (IW,243) UNITS(2),UNITS(3),UNITS(4),UNITS(4),UNITS(4),
1      UNITS(3),UNITS(4),UNITS(4)
      NLP=19
      FLAG12=1
      TEMP=(DBM-COR)/DBM
      TIME=ACT+(JJ-1+TEMP)*DDT
      IF (OT.EQ.DOT) TIME=ACT+TEMP*DDT
      IF (J.EQ.1.AND.L.EQ.1) WRITE (IW,205) I,TIME
      IF (J.EQ.1.AND.L.EQ.2) WRITE (IW,207) I,TIME
      IF (J.EQ.2.AND.L.EQ.3) WRITE (IW,209) I,TIME
      IF (J.EQ.2.AND.L.EQ.4) WRITE (IW,211) I,TIME
      NLP=NLP+1
10  GOTO 89
      9  BUMP(I,L)=BMO
      SIGN=BMO/ABS(BMO)
      FYC=(BYR(I,L)-BCR(I,L))/(BYM(I,L)-BCM(I,L))
      BURP(I,L)=(BMO-SIGN*BCM(I,L))*FYC+SIGN*BCR(I,L)
      BF1(I,J)=(ABS(BURP(I,L))+BCR(I,LBP))/(ABS(BMO)+BCM(I,LBP))
      CHANGE=1
      LBR(I,J)=4
      11 BSR(I,J)=BURP(I,L)+(BM1-BUMP(I,L))*BF1(I,J)
      IF (RF.NE.0.AND.RULE.EQ.4) GOTO 12
      BIF(I,J)=BSR(I,J)-BEF(I)
      IF (BM1*BMO.GT.0.) GOTO 89
      GOTO 23
      12 BF1(I,J)=BEF(I)*RF
      B16(I,J)=BURP(I,L)-BUMP(I,L)*BF1(I,J)
      FUY=(BUR(I,L)-BYR(I,L))/(BUM(I,L)-BYM(I,L))

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SIGN=BMI/ABS(BM1)
BUMP(I,L)=(BI6(I,J)+BYM(I,L)*SIGN*FUY-BYR(I,L)*SIGN)/
1      (FUY-BF1(I,J))
BURP(I,L)=BUMP(I,L)+BF1(I,J)+BI6(I,J)
LBR(I,J)=5
BIF(I,J)=BF1(I,J)-DEF11
IF (BM1*BM0.GT.0.) GOTO 89
GOTO 23
C
C      RULE 3: LOADING ON PRIMARY CURVE AFTER YEILDING.
C
13 IF (DBM*BM0.GT.0.) GOTO 89
IF (FLAG4.EQ.2.AND.FLAG5.EQ.1) GOTO 189
CHANGE=1
BUMP(I,L)=BMD
SIGN=BM0/ABS(BM0)
FUY=(BUR(I,L)-BYR(I,L))/(BUM(I,L)-BYM(I,L))
BURP(I,L)=(BM0-SIGN*BYM(I,L))*FUY+SIGN*BYR(I,L)
IF (RF.NE.0.) GOTO 14
RMX=BURP(I,K)
IF ((RMX+BURP(I,K+1)).LT.0.) RMX=-BURP(I,K+1)
BF1(I,J)=SQRT(RMX/BYR(I,L))*(BYR(I,L)+BCR(I,LOP))/(BYM(I,L)+
1      BCM(I,LOP))
14 IF (RF.NE.0.) BF1(I,J)=BF1(I)*RF
LBR(I,J)=5
IF (NCHANGE.EQ.0) GOTO 18
IF (NLP.LT.MLP) GOTO 16
IP=IP+1
WRITE (IW,239) IP,JO8,EQN
WRITE (IW,241)
WRITE (IW,243) UNITS(2),UNITS(3),UNITS(4),UNITS(4),
1      UNITS(3),UNITS(4),UNITS(4)
NLP=19
16 FLAG12=1
TIME=ACT+(JJ-1)*DDI
IF (OT.EQ.DOT) TIME=ACT
IF (J.EQ.1.AND.L.EQ.1) WRITE (IW,213) I,TIME
IF (J.EQ.1.AND.L.EQ.2) WRITE (IW,215) I,TIME
IF (J.EQ.2.AND.L.EQ.3) WRITE (IW,217) I,TIME
IF (J.EQ.2.AND.L.EQ.4) WRITE (IW,219) I,TIME
NLP=NLP+1
18 GOTO 11
C

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°C

C	RULE 4: AFTER CRACKING/UNLOADING FROM POINT (BUMP,BURP) ON	QUAKE
C	THE PRIMARY CURVE.	QUAKE
C		QUAKE
	15 IF (BM1*BM0.LE.0.) GOTO 23	QUAKE
	17 IF ((BURP(I,L)-BRC)*BM1.GT.0.) GOTO 89	QUAKE
	GOTO 3	QUAKE
C		QUAKE
C	RULE 5: AFTER YIELDING/UNLOADING FROM POINT (BUMP,BURP) ON	QUAKE
C	THE PRIMARY CURVE.	QUAKE
C		QUAKE
	19 IF (BM1*BM0.LT.0.) GOTO 23	QUAKE
	21 IF ((BURP(I,L)-BRC)*BM1.GT.0.) GOTO 89	QUAKE
	GOTO 7	QUAKE
	23 IF (RF.NE.0.) GOTO 30	QUAKE
	IF (FLAG4.EQ.2.AND.FLAG5.EQ.1) GOTO 189	QUAKE
	CALL SWITCH	QUAKE
	CHANGE=1	QUAKE
	IF ((BCH(I,L)-ABS(BUMP(I,L)))/LE.0.) GOTO 25	QUAKE
	IF (LBR(I,J).NE.4) GOTO 79	QUAKE
	GOTO 73	QUAKE
	25 B16(I,J)=BURP(I,LOW)-BUMP(I,LOW)+BF1(I,J)	QUAKE
	FL1=(BURP(I,L)-B16(I,J))/BUMP(I,L)	QUAKE
	IF (LBR(I,J).EQ.4) GOTO 27	QUAKE
	IF (ABS(BURP(I,L))-GE.BYR(I,L)) GOTO 27	QUAKE
	FL2=(BYR(I,L)+ABS(B16(I,J)))/BYM(I,L)	QUAKE
	IF (FL1.LT.FL2) GOTO 27	QUAKE
	BF2(I,J)=FL2	QUAKE
	SIGN=BM1/ABS(BM1)	QUAKE
	BUMP(I,L)=SIGN*BYM(I,L)	QUAKE
	BURP(I,L)=SIGN*BYR(I,L)	QUAKE
	GOTO 29	QUAKE
	27 BF2(I,J)=FL1	QUAKE
	29 IF ((BURP(I,L)-BRC)*BM1.LE.0.) GOTO 3	QUAKE
	IF (LBR(I,J).EQ.6) GOTO 89	QUAKE
	IF (FLAG4.EQ.2.AND.FLAG5.EQ.1) GOTO 189	QUAKE
	CHANGE=1	QUAKE
	LBR(I,J)=6	QUAKE
	BM(I,J)=(BRC-B16(I,J))/BF2(I,J)	QUAKE
	FLAG11=1	QUAKE
	CALL EQMCOB	QUAKE
	BIF(I,J)=BF2(I,J)+DEF(I)	QUAKE
	GOTO 89	QUAKE
	30 B16(I,J)=BURP(I,L)-BUMP(I,L)+BF1(I,J)	QUAKE

CALL SWITCH

FUY=(BUR(I,L)-BYR(I,L))/(BUM(I,L)-BYM(I,L))

SIGN=BM1/ABS(BM1)

BUMP(I,L)=(B16(I,J)+BYM(I,L)*SIGN+FUY-BYR(I,L)*SIGN)/

1 (FUY-BF1(I,J))

BURP(I,L)=BUMP(I,L)*BF1(I,J)+B16(I,J)

LBR(I,J)=15

GOTO 84

C

C

RULE 6: LOAD REVERSED FROM STEP 4 OR 5 AND LOADING TOWARDS
POINT (BUMP,BURP) ON THE PRIMARY CURVE.

C

31 IF (DBM*BM0.GT.0.) GOTO 29

IF (FLAG4.EQ.2.AND.FLAG5.EQ.1) GOTO 189

BUM7(I,J)=BM0

BUR7(I,J)=B16(I,J)+BM0*BF2(I,J)

BCF=BF2(I,J)

33 IF (BCE.GI.BF1(I,J)*UF) BCF=BF1(I,J)*UF

CHANGE=1

LBR(I,J)=7

BIF(I,J)=BCF-BEF(I)

BSR(I,J)=BUR7(I,J)+(BM1-BUM7(I,J))*BCF

IF (BM1*BM0.GI.0.) GOTO 89

IF (FLAG4.EQ.2.AND.FLAG5.EQ.1) GOTO 189

GOTO 37

C

C

RULE 7: UNLOADING FROM POINT (BUM7,BUR7) RESULTING DURING THE
APPLICATION OF RULE 6.

C

35 IF (BM1*BM0.LE.0.) GOTO 37

IF ((BUR7(I,J)-BRC)*BM1.GI.0.) GOTO 89

GOTO 29

37 IF (FLAG4.EQ.2.AND.FLAG5.EQ.1) GOTO 189

IF (LBR(I,J).EQ.8) GOTO 89

B18(I,J)=BUR7(I,J)-BUM7(I,J)*BCF

39 IF ((BURP(I,L)-BRC)*BM1.LE.0.) GOTO 3

IF (FLAG4.EQ.2.AND.FLAG5.EQ.1) GOTO 189

CALL SWITCH

BM(I,J)=(BRC-B18(I,J))/BCF

BCF=(BURP(I,L)-B18(I,J))/BUMP(I,L)

LBR(I,J)=8

CHANGE=1

FLAG11=1

C

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C	CALL EQMCCR				QUAKE
C	BIF(I,J)=BCF-REF(I)				QUAKE
C	GOTO 89				QUAKE
C	RULE 8: LOAD REVERSED FROM RULE 7, AND LOADING TOWARDS POINT				QUAKE
C	(BUMP,BURP) ON THE PRIMARY CURVE.				QUAKE
C					QUAKE
C	41 IF (DBM*BMO.GI.O.) GOTO 39				QUAKE
	IF (FLAG4.EQ.2.AND.FLAG5.EQ.1) GOTO 189				QUAKE
	BUM9(I,J)=BMO				QUAKE
	BUR9(I,J)=BI8(I,J)+BCF*BMO				QUAKE
	IF (BCF.GT.BF1(I,J)*UF) BCF=BF1(I,J)*UF				QUAKE
	CHANGE=1				QUAKE
	LBR(I,J)=9				QUAKE
	BSR(I,J)=BUR9(I,J)+(BM1-BUM9(I,J))*BCF				QUAKE
	BIF(I,J)=BCF-REF(I)				QUAKE
	IF (BM1*BMO.GI.O.) GOTO 89				QUAKE
	IF (FLAG4.EQ.2.AND.FLAG5.EQ.1) GOTO 189				QUAKE
	GOTO 45				QUAKE
C	RULE 9: UNLOADING FROM POINT (BUM9,BUR9) RESULTING FROM				QUAKE
C	UNLOADING DURING RULE 8.				QUAKE
C					QUAKE
C	43 IF (BM1*BMO.GI.O.) GOTO 51				QUAKE
	IF (FLAG4.EQ.2.AND.FLAG5.EQ.1) GOTO 189				QUAKE
	45 BI10(I,J)=BUR9(I,J)-BUM9(I,J)*BCF				QUAKE
	47 CALL SWITCH				QUAKE
	49 IF ((BUR7(I,J)-BRC)*BM1.LE.O.) GOTO 29				QUAKE
	IF (LBR(I,J).EQ.10) GOTO 89				QUAKE
	- IF (FLAG4.EQ.2.AND.FLAG5.EQ.1) GOTO 189				QUAKE
	CHANGE=1				QUAKE
	LBR(I,J)=10				QUAKE
	BCF=(BUR7(I,J)-BI10(I,J))/BUM7(I,J)				QUAKE
	BM(I,J)=(BRC-BI10(I,J))/BCF				QUAKE
	FLAG11=1				QUAKE
	CALL EQMCCR				QUAKE
	BIF(I,J)=BCF-REF(I)				QUAKE
	GOTO 89				QUAKE
	51 IF ((BUR9(I,J)-BRC)*BM1.GI.O.) GOTO 89				QUAKE
	GOTO 39				QUAKE
C	RULE 10: LOAD REVERSED FROM RULE 9 AND LOADING TOWARDS POINT				QUAKE
C	(BUM7,BUR7) RESULTING FROM UNLOADING DURING RULE 6.				QUAKE
C					QUAKE

C	53 IF (DBM*BM0.GT.0.) GOTO 49	QUAKE
	IF (FLAG4.EQ.2.AND.FLAG5.EQ.1) GOTO 189	QUAKE
	BUM11(I,J)=BM0	QUAKE
	BUR11(I,J)=BII0(I,J)+BCF*BM0	QUAKE
	IF (BCF.GT.BF1(I,J)*UF) BCF=BF1(I,J)*UF	QUAKE
	CHANGE=1	QUAKE
	LBR(I,J)=11	QUAKE
	BSR(I,J)=BUR11(I,J)+(BM1-BUM11(I,J))*BCF	QUAKE
	BIF(I,J)=BCF-BEF(I)	QUAKE
	IF (BM1*BM0.GT.0.) GOTO 89	QUAKE
	IF (FLAG4.EQ.2.AND.FLAG5.EQ.1) GOTO 189	QUAKE
	GOTO 57	QUAKE
C		QUAKE
C	RULE 11: UNLOADING FROM POINT (RUM11,BUR11) RESULTING FROM	QUAKE
C	UNLOADING DURING RULE 10.	QUAKE
C		QUAKE
	55 IF (BM1*BM0.GT.0.) GOTO 61	QUAKE
	IF (FLAG4.EQ.2.AND.FLAG5.EQ.1) GOTO 189	QUAKE
	57 BII2(I,J)=BUR11(I,J)-BUM11(I,J)*BCF	QUAKE
	CALL SWITCH	QUAKE
	59 IF ((BUR9(I,J)-BRC)*BM1.LE.0.) GOTO 19	QUAKE
	IF (LBR(I,J).EQ.12) GOTO 89	QUAKE
	IF (FLAG4.EQ.2.AND.FLAG5.EQ.1) GOTO 189	QUAKE
	CHANGE=1	QUAKE
	LBR(I,J)=12	QUAKE
	BCF=(BUR9(I,J)-BII2(I,J))/BUM9(I,J)	QUAKE
	BM(I,J)=(BRC-BII2(I,J))/BCF	QUAKE
	FLAG11=1	QUAKE
	CALL EQMCOB	QUAKE
	BIF(I,J)=BCF-BEF(I)	QUAKE
	GOTO 89	QUAKE
	61 IF ((BUR11(I,J)-BRC)*BM1.GT.0.) GOTO 89.	QUAKE
	GOTO 49	QUAKE
C		QUAKE
C	RULE 12: LOAD REVERSED FROM RULE 11 AND LOADING TOWARDS POINT	QUAKE
C	(BUM9,BUR9) RESULTING FROM UNLOADING DURING RULE 8.	QUAKE
C		QUAKE
	63 IF (DBM*BM0.GT.0.) GOTO 59	QUAKE
	IF (FLAG4.EQ.2.AND.FLAG5.EQ.1) GOTO 189	QUAKE
	BUM13(I,J)=BM0	QUAKE
	BUR13(I,J)=BII2(I,J)+BCF*BM0	QUAKE
	IF (BCF.GT.BF1(I,J)*UF) BCF=BF1(I,J)*UF	QUAKE

LBM(NBM)=I
FLAG6=1
93 CONTINUE
FLAG10=0

COLUMNS.

DO 187 I=1,NC

K=1

CLC=CCCL(I)

CRZ=CRZL(I)

CHANGE=0

N1=NDFC(I,1)

N2=NDFC(I,2)

N3=NDFC(I,3)

N4=NDFC(I,4)

DO 185 J=1,2

TEST FOR COLUMNS HINGED TO THE FOUNDATIONS.

IF (NDFC(I,J).EQ.99999) GOTO 183

NJ=NDFC(I,J)

CM1=CM(I,J)

CM0=CM0(I,J)

DBM=CM1-CM0

CCF=CEF(I)+CIF(I,J)

CSR(I,J)=CSR(I,J)+UBM*CCF

CRC=CSR(I,J)

L=K

LBP=K+1

IF (CM1.LT.0. .AND. CM0.EQ.0.) CALL SWITCH

IF (CM0.LT.0.) CALL SWITCH

RULE=LCR(I,J)

GOTO (95,99,107,109,113,125,129,135,137,147,149,157,159,165,171,
179),RULE

RULE 1:

95 IF (CCR(I,L)-ABS(CRC)).GT.0.) GOTO 183

IF (FLAG4.EQ.2 .AND. FLAG5.EQ.1) GOTO 189

97 IF (VCYR(I,L)-ABS(CRC)).LE.0.) GOTO 101

IF (UCR(I,J).EQ.2) GOTO 183

IF (FLAG4.EQ.2 .AND. FLAG5.EQ.1) GOTO 189

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CHANGE=1	QUAKE
LCR(I,J)=2	QUAKE
SIGN=CRC/ABS(CRC)	QUAKE
CCF=(CYR(I,L)-CCR(I,L))/(LYM(I,L)-CCM(I,L))	QUAKE
CM(I,J)=(CRC-SIGN*CCR(I,L))/CCF+SIGN*CCM(I,L)	QUAKE
FLAG11=2	QUAKE
CALL EQMCCR	QUAKE
CIF(I,J)=CCF-CEF(I)	QUAKE
GOTO 183	QUAKE
C	QUAKE
C	QUAKE
C	QUAKE
99 IF (DBM*CMO.GT.O.) GOTO 97	QUAKE
IF (FLAG4.EQ.2.AND.FLAG5.EQ.1) GOTO 189	QUAKE
GOTO 103	QUAKE
101 IF (FLAG4.EQ.2.AND.FLAG5.EQ.1) GOTO 189	QUAKE
CHANGE=1	QUAKE
SIGN=CRC/ABS(CRC)	QUAKE
CCF=(CUR(I,L)-CYR(I,L))/(LUM(I,L)-CYM(I,L))	QUAKE
CM(I,J)=(CRC-SIGN*CYR(I,L))/CCF+SIGN*CYM(I,L)	QUAKE
LCR(I,J)=3	QUAKE
FLAG11=2	QUAKE
CALL EQMCCR	QUAKE
CIF(I,J)=CCF-CEF(I)	QUAKE
IF (NOMNGE.EQ.O) GOTO 104	QUAKE
IF (NLP.LT.MLP) GOTO 102	QUAKE
IP=IP+1	QUAKE
WRITE (IW,239) IP,JBB,EQN	QUAKE
WRITE (IW,241)	QUAKE
WRITE (IW,243) UNITS(2),UNITS(3),UNITS(4),UNITS(4),	QUAKE
UNITS(3),UNITS(4),UNITS(4),	QUAKE
1	QUAKE
NLP=19	QUAKE
102 FLAG12=1	QUAKE
TEMP=(DBM-COR)/DBM	QUAKE
TIME=ACT+(JJ-1+TEMP)*DDT	QUAKE
IF (OT.EQ.DDT) TIME=ACT+TEMP*DDT	QUAKE
IF (J.EQ.1.AND.L.EQ.1) WRITE (IW,221) I,TIME	QUAKE
IF (J.EQ.1.AND.L.EQ.2) WRITE (IW,223) I,TIME	QUAKE
IF (J.EQ.2.AND.L.EQ.3) WRITE (IW,225) I,TIME	QUAKE
IF (J.EQ.2.AND.L.EQ.4) WRITE (IW,227) I,TIME	QUAKE
NLP=NLP+1	QUAKE
104 GOTO 183	QUAKE
103 CUM(I,L)=CMO	QUAKE

121 CF2(I,J)=FL1	QUAKE
123 IF ((CURP(I,L)=CRC)*CM1.LE.0.) GOTO 97	QUAKE
IF (LCR(I,J).EQ.6) GOTO 183	QUAKE
IF (FLAG4.EQ.2.AND.FLAG5.EQ.1) GOTO 189	QUAKE
CHANGE=1	QUAKE
LCR(I,J)=6	QUAKE
CM1,J)=(CRC-CI6(I,J))/CF2(I,J)	QUAKE
FLAG11=2	QUAKE
CALL EQMCDR	QUAKE
CF(I,J)=CF2(I,J)-LEF(I)	QUAKE
GOTO 183	QUAKE
124 CI6(I,J)=CURP(I,L)=CUMP(I,L)*CF1(I,J)	QUAKE
CALL SWITCH	QUAKE
FUY=(CUR(I,L)=CYR(I,L))/(CUM(I,L)-CYM(I,L))	QUAKE
SIGN=CM1/ABS(CM1)	QUAKE
CUMP(I,L)=(CI6(I,J)+CYM(I,L))*SIGN+FUY=CYP(I,L)*SIGN/	QUAKE
1 (FUY=CF1(I,J))	QUAKE
CURP(I,L)=CUMP(I,L)*CF1(I,J)+CI6(I,J)	QUAKE
LCR(I,J)=15	QUAKE
GOTO 178	QUAKE
C	QUAKE
C	QUAKE
C	QUAKE
125 IF (DBM*CM0.GT.0.) GOTO 123	QUAKE
IF (FLAG4.EQ.2.AND.FLAG5.EQ.1) GOTO 189	QUAKE
CUM7(I,J)=CM0	QUAKE
CUR7(I,J)=CI6(I,J)+CUM7(I,J)	QUAKE
CCF=CF2(I,J)	QUAKE
127 IF (CCF.GT.CF1(I,J)*UF) CCF=CF1(I,J)*UF	QUAKE
CHANGE=1	QUAKE
LCR(I,J)=7	QUAKE
CF(I,J)=CCF=CF1(I)	QUAKE
CSR(I,J)=CUR7(I,J)+(CM1-CUM7(I,J))*CCF	QUAKE
IF (CM1*CM0.GT.0.) GOTO 183	QUAKE
IF (FLAG4.EQ.2.AND.FLAG5.EQ.1) GOTO 189	QUAKE
GOTO 131	QUAKE
C	QUAKE
C	QUAKE
C	QUAKE
129 IF (CM1*CM0.LE.0.) GOTO 131	QUAKE
IF ((CUR7(I,J)=CRC)*CM1.GT.0.) GOTO 183	QUAKE
GOTO 123	QUAKE
131 IF (FLAG4.EQ.2.AND.FLAG5.EQ.1) GOTO 189	QUAKE

Line	Code	Text	Comments
133	C	CALL SWITCH	
134	C	CI8(I,J)=CUR7(I,J)-CUM7(I,J)*CCF	
135	C	IF (ICUMP(I,L)-CRC)*CM1.LE.O.) GOTO 97	
136	C	IF (ICR(I,J).EQ.8) GOTO 183	
137	C	IF (FLAG4.EQ.2.AND.FLAG5.EQ.1) GOTO 189.	
138	C	CHANGE=1	
139	C	LCR(I,J)=2	
140	C	CCF=(CURP(I,L)-CI8(I,J))/CUMP(I,L)	
141	C	CH(I,J)=(CRC-CI8(I,J))/CCF	
142	C	FLAG11=2	
143	C	CALL EQMCCR	
144	C	CIF(I,J)=CCF-CEF(I)	
145	C	GOTO 183	
146	C	RULE 8.	
147	C	135. IF (OBH*CMO.GT.O.) GOTO 133.	
148	C	IF (FLAG4.EQ.2.AND.FLAG5.EQ.1) GOTO 189	
149	C	CUM9(I,J)=CMO	
150	C	CUR9(I,J)=CI8(I,J)+CCF*CMO	
151	C	IF (CCF.GT.CF1(I,J)*UF) CCF=CF1(I,J)*UF	
152	C	CHANGE=1	
153	C	LCR(I,J)=9	
154	C	CSR(I,J)=CUR9(I,J)+(CM1-CUM9(I,J))*CCF	
155	C	CIF(I,J)=CCF-CEF(I)	
156	C	IF (CM1*CMO.GT.O.) GOTO 183	
157	C	IF (FLAG4.EQ.2.AND.FLAG5.EQ.1) GOTO 189	
158	C	GOTO 139	
159	C	RULE 9.	
160	C	137 IF (CM1*CMO.GT.O.) GOTO 145	
161	C	IF (FLAG4.EQ.2.AND.FLAG5.EQ.1) GOTO 189	
162	C	CI10(I,J)=CUR9(I,J)-CUM9(I,J)*CCF	
163	C	CALL SWITCH	
164	C	IF (CUR7(I,J)-CRC)*CM1.LE.O.) GOTO 123	
165	C	IF (LCR(I,J).EQ.10) GOTO 183	
166	C	IF (FLAG4.EQ.2.AND.FLAG5.EQ.1) GOTO 189	
167	C	CHANGE=1	
168	C	LCR(I,J)=10	
169	C	CCF=(CUR7(I,J)-CI10(I,J))/CUM7(I,J)	
170	C	CH(I,J)=(CRC-CI10(I,J))/CCF	
171	C	FLAG11=2	

CALL EQMCOR	QUAKE
CIF(I,J)=CCF-CEF(I)	QUAKE
GOTO 183	QUAKE
145 IF ((CUR9(I,J)-CRC)*CM1*GT.0.) GOTO 183	QUAKE
GOTO 133	QUAKE
C	QUAKE
C	QUAKE
C	QUAKE
RULE 10.	QUAKE
147 IF (DBM*CM0*GT.0.) GOTO 143	QUAKE
IF (FLAG4*EQ.2.AND.FLAG5*EQ.1) GOTO 189	QUAKE
CUM1(I,J)=CM0	QUAKE
CUR1(I,J)=C110(I,J)+CCF*CM0	QUAKE
IF (CCF*GT.CF1(I,J)*UF) CCF=CF1(I,J)*UF	QUAKE
CHANGE=1	QUAKE
LCR(I,J)=11	QUAKE
CSR(I,J)=CUR1(I,J)+(CM1-CUM1(I,J))*CCF	QUAKE
CIF(I,J)=CCF-CEF(I)	QUAKE
IF (CM1*CM0*GT.0.) GOTO 183	QUAKE
IF (FLAG4*EQ.2.AND.FLAG5*EQ.1) GOTO 189	QUAKE
GOTO 151	QUAKE
C	QUAKE
C	QUAKE
C	QUAKE
RULE 11.	QUAKE
149 IF (CM1*CM0*GT.0.) GOTO 195	QUAKE
IF (FLAG4*EQ.2.AND.FLAG5*EQ.1) GOTO 189	QUAKE
151 C112(I,J)=CUR1(I,J)-CUM1(I,J)*CCF	QUAKE
CALL SWITCH	QUAKE
153 IF ((CUR9(I,J)-CRC)*CM1*LE.0.) GOTO 133	QUAKE
IF (LCR(I,J)*EQ.12) GOTO 183	QUAKE
IF (FLAG4*EQ.2.AND.FLAG5*EQ.1) GOTO 189	QUAKE
CHANGE=1	QUAKE
LCR(I,J)=12	QUAKE
CCF=(CUR9(I,J)-C112(I,J))/CUM9(I,J)	QUAKE
CM1(I,J)=LCRC-C112(I,J)/CCF	QUAKE
FLAG11=2	QUAKE
CALL EQMCOR	QUAKE
CIF(I,J)=CCF-CEF(I)	QUAKE
GOTO 183	QUAKE
155 IF ((CUR11(I,J)-CRC)*CM1*GT.0.) GOTO 183	QUAKE
GOTO 143	QUAKE
C	QUAKE
C	QUAKE
C	QUAKE
RULE 12.	QUAKE


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207 FORMAT (22X,'FLASH** A NEGATIVE HINGE HAS JUST FORMED AT THE LEFT',
1      , 'END OF BEAM ',13,' AT TIME ',F9.6)
209 FORMAT (22X,'FLASH** A POSITIVE HINGE HAS JUST FORMED AT THE',
1      , 'RIGHT END OF BEAM ',13,' AT TIME ',F9.6)
211 FORMAT (22X,'FLASH** A NEGATIVE HINGE HAS JUST FORMED AT THE',
1      , 'RIGHT END OF BEAM ',13,' AT TIME ',F9.6)
C
213 FORMAT (22X,'FLASH** THE POSITIVE HINGE AT THE LEFT END OF BEAM ',
1      , 13,' HAS JUST BEEN REMOVED AT TIME ',F9.6)
215 FORMAT (22X,'FLASH** THE NEGATIVE HINGE AT THE LEFT END OF BEAM ',
1      , 13,' HAS JUST BEEN REMOVED AT TIME ',F9.6)
217 FORMAT (22X,'FLASH** THE POSITIVE HINGE AT THE RIGHT END OF BEAM ',
1      , 13,' HAS JUST BEEN REMOVED AT TIME ',F9.6)
219 FORMAT (22X,'FLASH** THE NEGATIVE HINGE AT THE RIGHT END OF BEAM ',
1      , 13,' HAS JUST BEEN REMOVED AT TIME ',F9.6)
C
221 FORMAT (22X,'FLASH** A POSITIVE HINGE HAS JUST FORMED AT THE',
1      , 'LOWER END OF COLUMN ',13,' AT TIME ',F9.6)
223 FORMAT (22X,'FLASH** A NEGATIVE HINGE HAS JUST FORMED AT THE',
1      , 'LOWER END OF COLUMN ',13,' AT TIME ',F9.6)
225 FORMAT (22X,'FLASH** A POSITIVE HINGE HAS JUST FORMED AT THE',
1      , 'UPPER END OF COLUMN ',13,' AT TIME ',F9.6)
227 FORMAT (22X,'FLASH** A NEGATIVE HINGE HAS JUST FORMED AT THE',
1      , 'UPPER END OF COLUMN ',13,' AT TIME ',F9.6)
C
229 FORMAT (22X,'FLASH** THE POSITIVE HINGE AT THE LOWER END OF',
1      , 'COLUMN ',13,' HAS JUST BEEN REMOVED AT TIME ',F9.6)
231 FORMAT (22X,'FLASH** THE NEGATIVE HINGE AT THE LOWER END OF',
1      , 'COLUMN ',13,' HAS JUST BEEN REMOVED AT TIME ',F9.6)
233 FORMAT (22X,'FLASH** THE POSITIVE HINGE AT THE UPPER END OF',
1      , 'COLUMN ',13,' HAS JUST BEEN REMOVED AT TIME ',F9.6)
235 FORMAT (22X,'FLASH** THE NEGATIVE HINGE AT THE UPPER END OF',
1      , 'COLUMN ',13,' HAS JUST BEEN REMOVED AT TIME ',F9.6)
C
237 FORMAT (/)
239 FORMAT (1H1, 120X,'PAGE ',13/27X,20A4/27X,20A4//
1      , 59X,'** OUTPUT DATA **')
241 FORMAT (53X,'RESULTS OF DYNAMIC ANALYSIS.////////')
243 FORMAT (61X,'STORY RESPONSE',7X,'STORY',3X,'<',6X,'FORCES ',1,A3
1      , ',6X,12,X,<',7X,'SWAY ',1,A3,1,7X,12,
2      , 'VELOCITY',5X,<' ACCELN ',1,A3,1,1,A3,1,1,A3,
3      , ') > ',4X,'TIME',7X,'INM.',7X,'INERTIAL',9X,'SHEAR',
4      , 'ABSOLUTE',7X,'RELATIVE',6X,1,1,A3,1,1,A3,1,1,

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END

2000 79

FIN